STORMWATER MANAGEMENT AND DRAINAGE MANUAL

March 2025

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1. Submittal Procedures

1.1 General

Recognizing that properly designed stormwater management systems are essential to the general public's health and welfare within a metropolitan area as the City of Little Rock, the City hereby adopts the following criteria for standard procedures in stormwater management. This manual is intended to serve as a guide for the design of all stormwater infrastructure, including the design of new facilities and the upgrading of existing facilities.

Any company, agency, or person proposing to alter current land use within the City's Planning Jurisdiction shall submit drainage plans to the Department of Planning and Development for approval of a stormwater management and drainage plan before grading permits are issued, or subdivisions are approved. No land shall be developed except upon approval by the Department of Planning and Development, in coordination with appropriate departments.

Exceptions where no stormwater management and drainage plan are required according to Section 29-61 of the City of Little Rock Municipal Code:

- One (1) new or existing single-family structure.
- One (1) new or existing duplex family structure.
- One (1) new commercial or industrial structure located on less than a one-acre individual lot.
- One (1) existing commercial or industrial structure where additional structural improvements are less than five hundred (500) square feet.

Development not requiring a stormwater management and drainage plan still must comply with any applicable floodplain or building codes regarding drainage, stormwater, flooding, and finished floor elevation criteria.

The application for a Grading and Land Alteration Permit shall be prepared by the Engineer of Record, who is a licensed Professional Engineer of the State of Arkansas and shall be submitted in accordance with the submittal procedures described in this chapter. The Grading and Land Alteration Permit application shall consist of the Final Drainage Report and the Grading and Drainage Design Plans and Specifications (Plans and Specifications).

The updated Stormwater Management and Drainage Manual shall be in full force and effect on March 26, 2025, after passage of this Resolution, unless DEQ agrees that the updated, proposed version of the Stormwater Management and Drainage Manual fulfills the City's requirement to present such a manual after March, 26, 2025, then the date of approval shall be the effective date this includes any revisions or updates to the manual after the adoption of this Resolution.

Projects which have completed review under Little Rock City Code §§ 31-36, et seq. prior to the updated Stormwater Management and Drainage Manual being in full force and effect are to comply with the September 2016 Update version of the Little Rock Stormwater Management and Drainage Manual. The September 2016 Update version of the Stormwater Drainage Manual shall be applied in instances where a sketch preliminary plat under Little Rock City Code § 31-36(1) has been submitted on or before September 26, 2025, and a completed, prepared preliminary plat for consideration for approval by the Little Rock Planning Commission under Little Rock City Code § 31-36(2) is submitted on or before March 26, 2026.

1.1.1 Additional Regulatory Requirements

Arkansas Department of Environmental Quality

A NPDES Construction Stormwater General Permit (Permit No. ARR150000) is required for discharges from large and small construction activities that result in a total land disturbance of equal to or greater than one acre, where those discharges enter waters of the State or a municipal separate storm sewer system (MS4).

- Small construction sites (disturbing one acre or more and less than five acres) have automatic coverage under the Construction Stormwater General Permit. Under automatic coverage for small sites, it is not necessary to submit any documents to ADEQ and there is no fee. However, the automatic Notice of Coverage (NOC) must be posted at the site prior to commencing construction and a Stormwater Pollution Prevention Plan (SWPPP) must be prepared and made available at the site prior to commencing construction.
- Large Construction Sites (disturbing five acres or more) must submit a Notice of Intent (NOI), a Stormwater Pollution Prevention Plan (SWPPP) and pay a fee to the Arkansas Department of Environmental Quality (ADEQ) in order to obtain coverage for discharges of stormwater associated with construction activity at any site or common plan of development or sale that will result in the disturbance of five (5) or more acres of total land area. Additional information may be found at: https://www.adeq.state.ar.us/water

U.S. Army Corps of Engineers

Section 404 of the Clean Water Act requires a permit from the U.S. Army Corps of Engineers (USACE) prior to discharging dredged or fill material into waters of the United States, including wetlands. Activities in waters of the United States regulated under this program include fill for development, water resources projects (such as dams and levees), infrastructure development (such as highways and airports), streambank restoration, and mining projects.

Floodplain Development Permits

Any development within or bordering a Special Flood Hazard Area, as portrayed on FEMA Flood Insurance Rate Maps (FIRMs) is required to obtain a Floodplain Development Permit. Permit requirements and application procedures can be found in Section 13-56 of the City of Little Rock Municipal Code.

1.2 Submittal Procedures

1.2.1 Conceptual Drainage Review

A conceptual drainage plan review with staff is suggested before preliminary platting for the purpose of overall general drainage concept review.

1.2.2 Preliminary Review

A preliminary stormwater and drainage plan, and accompanying information, shall be submitted at the time of preliminary plat submittal. If needed, a review meeting will be scheduled by the Department of Planning and Development with representatives of the developer, including the Engineer of Record, to review the overall concepts included in the preliminary stormwater and drainage plan. The purpose of this review shall be to jointly agree upon an overall stormwater management concept for all phases of the proposed development and to review criteria and design parameters which shall apply to final design of the project. Preliminary drainage plan/study must be approved prior to Preliminary Platting.

1.2.3 Final Drainage Review

Following the preliminary stormwater management and drainage plan review, the final stormwater management and drainage plan shall be prepared for each phase of the proposed project as each phase is developed. The final plan shall constitute a refinement of the concepts approved in the preliminary

stormwater and drainage plan with preparation and submittal of detailed information as required in the drainage manual. This plan shall be submitted at the time construction drawings are submitted for approval. Final drainage plan must be approved prior to approval of construction plans. No final plat is to be approved until the drainage structures approved on the construction plans are constructed, inspected, and approved by the Department of Planning and Development.

1.2.4 Online Submittal Procedures

The grading and land alteration permit application process is through the Planning and Development Department's online portal (https://permitpayment.littlerock.gov/ips_PD/Views/Login.aspx) New users must register for an online account. The applicant is required to fill out the online application for a grading & land alteration permit and submit the following documents through the portal: grading & drainage plans, sediment and erosion control plans, land survey, drainage report, and soil loss estimate calculations. When filling out the online application, no fields should be left blank. If a field is not applicable to the project, enter "Not Applicable" in those fields on the application. For required numerical value fields, use only whole numbers (no commas or decimal points).

1.3 Plans and Specifications Requirements

Plans and Specifications for plans including stormwater drainage are to be signed by a Professional Engineer registered in the State of Arkansas in accordance with applicable state statutes and State PELS board licensure requirements. Because all Plans, Specifications, and Calculations will be retained by the City for use as permanent records, neatness, clarity, and completeness are very important, and lack of these qualities will be considered sufficient basis for submittal rejection.

Plan sheet size must be 11 inches x 17 inches with all sheets in each set of plans the same size. Plan and profile drawings shall be prepared according to the following:

Plan Drawings

- Maximum horizontal scale 1 inch = 100 feet

Profile Drawings for drainage ditches and storm systems

- Suggested horizontal scale 1 inch = 20 feet
- Maximum horizontal scale 1 inch = 50 feet
- Minimum vertical scale 1 inch = 5 feet

Each sheet in a set of Plans shall contain a sheet number, the total number of sheets in the plans, proper project identification, and the date. Revised sheets submitted must contain a revision block with identifying notations and dates for revisions. A complete legend shall be included in the set of Plans.

To ensure reviews are completed in a timely manner, Plans and Specifications for all proposed improvements must be submitted in the following format during the project application process, where pertinent, and shall include at a minimum: (1) Title Sheet, (2) General Layout Sheet, (3) Grading and Drainage Plan (4) Paving, and/or Building Plans, (5) Three Phase Erosion and Sedimentation Control Plan, (6) Plan and Profile Sheets, (7) Cross Sections, (8) Standard and Special Detail Sheets, (9) Mapping, and (10) Calculations. A detailed checklist of requirements is included in Appendix A.

1.3.1 Title Sheet

The Title Sheet shall include:

1. The designation of the project which includes the nature of the project, the name of the development, city, and state.
2. Project number.

3. Index of Sheets.
4. Vicinity maps showing project location in relation to streets, railroads, and physical features. The location map shall have a north arrow and appropriate scale.
5. A project control benchmark identified as to the location and elevation with notation referencing City monument(s) used to establish the Project Benchmark.
6. Reference to the horizontal and vertical datum for the project.
    a. Prior to the release of the new Low Distortion Projection Zones (LDPZs) and 2022 Datums, planned to be released in 2025.
        i. The horizontal datum shall be NAD83, Arkansas State Plane, North Zone.
        ii. The vertical datum shall be North American Vertical Datum of 1988 (NAVD88).
    b. After the release of the new Low Distortion Projection Zones (LDPZs) and 2022 Datums, planned to be released in 2025.
        i. Any projects started in a legacy zone and datum shall stay in that legacy zone and datum. Any projects being started shall be started utilizing the new zones and datums.
        ii. The unit of measurement for NAD83 Horizontal Datum shall be the US Survey Foot.
        iii. The unit of measurement for NAVD88 Vertical Datum shall be the US Survey Foot.
        iv. The unit of measurement for NATRF2022 Horizontal Datum shall be International Foot.
        v. The unit of measurement for NAPGD2022 Vertical Datum shall be the International Foot.
7. Survey metadata shall be noted and include the following:
    a. Horizontal Datum and/or Horizontal published control relied upon.
    b. Vertical Datum and/or Vertical published benchmark relied upon.
    c. Horizontal epoch adjustment:
        i. Any new adjustments that may supersede the 2011 adjustment.
        ii. 2011 Adjustment Epoch 2010.00.
        iii. 2007 NSRS Adjustment (Superseded).
        iv. 1990 HARN Adjustment (Superseded).
        v. 1986 Adjustment (Superseded).
    d. Geoid Model.
    e. State Plane Coordinate Zone or Low Distortion Projection Zone.
    f. The time and date the data was collected.
    g. Noting if legacy data was transformed via “on ground” survey observations.
    h. Unit of measure of Metadata (US Survey Foot or International Foot)
8. The name and address of the owner of the project and the name and address of the engineer preparing the plans.
9. Floodplain statement identifying the FIRM panel, date, and flood zone; and,

10. Engineer's seal (every sheet).

1.3.2 General Layout Sheet

The general layout sheet shall include:

1. North arrow and scale.
2. Legend of symbols that will apply to all sheets.
3. Name of subdivision, if applicable, and all street names. Unplatted tracts should have an accurate tie to at least one quarter section corner.
4. Boundary line or project area.
5. Benchmark location and benchmark calls.
6. Location and description of existing major drainage facilities within or adjacent to the project area.
7. Location of major proposed drainage facilities.
8. Floodplain Boundaries with mapped BFE, if available
9. Name of each utility within or adjacent to the project area.
10. Standard notes.
11. If more than one general layout sheet is required, a match line should be used to show continuation of coverage from one sheet to the next.

1.3.3 Other Requirements for Plans and Specifications

Other requirements for Plans and Specifications include:

1. The registration seal of the Engineer of Record shall be placed in a convenient place in the lower right-hand corner of each sheet of plans.
2. Elevations on profiles of sections or as indicated on plans shall have survey data or best available topographic data. At least one permanent benchmark in the vicinity of each project shall be noted on the first drawing of each project, and their location and elevation shall be clearly defined.
3. Convention for stationing shall be West to East or South to North from the left to the right side of the sheet respectively.
4. Each project shall show at least 20' of topography on each side. At least 50' of topography shall be shown in areas of channel flow at the property boundary. All existing topography and any proposed changes, including utilities, telephone installations, etc., shall be shown on the plans, profiles, and cross-sections.
5. Revisions to drawing shall be indicated above the title block in a revision box and shall show the nature of the revision and the date made.
6. Utilizing the standard symbols for engineering plans, all existing utilities, telephone installations, sanitary and storm sewers, pavements, curbs, inlets, and culverts, etc., shall be shown with a dithered/grayed out linework. Proposed facilities shall be shown with a solid line and land, lot, easement, and property lines shall be shown with bold and black linework.
7. Lot lines and dimensions shall be shown where applicable.
8. Show and label FEMA floodplains and/or the 100-year floodplain.
9. Minimum floor elevation shall be shown on each lot when located in a designated floodplain and in areas where flooding is known to occur. For all occupied structures within a designated floodplain, the top surface of the lowest floor must have an elevation at least one (1) foot or more above the published base flood elevation (BFE). All occupied buildings, whether in or out of a designated floodplain shall have the finished floor elevation (FFE) a minimum of 12" above the land immediately surrounding the building. Reference Section 13-60 of the City of Little Rock Municipal Code for additional information.

10. It shall be understood that the requirements outlined in these sections are only minimum requirements and shall only be applied when conditions, design criteria, and materials conform to the City specifications and are normal and acceptable to the Design Review Engineer. When unusual subsoil or drainage conditions are suspected, an investigation should be made, and a special design prepared in line with good engineering practice.

1.4 Grading and Drainage Report Requirements

1.4.1 Preliminary Grading and Drainage Report

A Preliminary Grading and Drainage Report will be required at the time of the Technical Plat Review for site development projects. The Preliminary Grading and Drainage Report shall follow the Final Grading and Drainage Report Template provided in Appendix A of this Drainage Manual. In addition to the Preliminary Grading and Drainage Report, also submit preliminary grading and drainage drawings.

1.4.2 Final Grading and Drainage Report

A Final Grading and Drainage Report shall be included in the Final Grading and Drainage Permit Application. Computer model input summary tables and output result tables shall also be provided as part of the Final Grading and Drainage Report. An example of input and output data to report may be found in Appendix B.

A Grading and Drainage Permit will not be issued until the Final Report has been submitted, reviewed, and approved. The Design Review Engineer may request a more detailed drainage study prior to the approval of the Final Grading and Drainage Permit application and issuance of the permit.

If hydrologic and hydraulic studies reveal that the proposed development would cause increased frequency of flooding, increased depth of inundation of structures, or inundation of unprotected structures not previously subject to inundation, then the permit application shall be denied unless one or more of the following mitigation measures are used to remove increases: (1) onsite storage, (2) offsite storage, or (3) offsite drainage systems improvements.

If it is determined by the Design Review Engineer that offsite drainage improvements are a feasible option to mitigate development impacts, then cost sharing will be in accordance with City ordinances. If the City is unable to contribute its share of the offsite costs, the developer shall have the option of: a) building the offsite improvements at his own expense, b) providing detention so as to match pre-development downstream capacities, or c) delaying the project until the City is able to share in the offsite costs.

1.5 Electronic As-Builts

Per Section 31-117(b)(18) and Section 29-5 of the City of Little Rock's Municipal Code, the City of Little Rock Public Work's Department requires as-built plans and information submitted from the Engineer of Record with final plats; request for certificates of occupancies on building permits; and following street and drainage infrastructure construction projects. Plans and information should be provided on public and private stormwater drainage systems installed and/or modified.

Final approval of final plat shall not be given until the Department of Planning and Development receives an electronic copy of the Stormwater Drainage Features As-Built, in either a compatible ArcGIS file format (Esri shapefile or Esri geodatabase), or AutoCAD .dwg file format. The As-Built Plan drawings shall be in State Plane Arkansas North Zone coordinates, with the North American Datum 1983 with units as survey feet. The As-Built drawings shall have the stormwater features drawn in a separate layer in AutoCAD so the features can be easily separated from other layers in the drawing. The associated attribute data table will conform to the approved specifications contained in the "SW Attribute Data Entry

Template.xlsx" as provided by the City's Public Works Department. On the As-Built Plans, all Control, Linear and Junction map features will be annotated by a unique identifier that will correspond to the same unique identifier in the "SW Attribute Data Entry Template.xlsx" or GIS attribute table. All required attribute information for each Linear and Junction feature will be completed in the "SW Attribute Data Entry Template.xlsx" or GIS attribute table as follows, or as indicted by bold column headings in the "SW Attribute Data Entry Template.xlsx", using the domain values specified in Appendix A of this Drainage Manual.

2. Stormwater Criteria, Planning, and Regulation

2.1 Stormwater Sizing Criteria

Development projects applying for a Grading and Drainage Permit shall meet the following criteria related to stormwater runoff and protection of existing water bodies and properties. For the purposes of this Drainage Manual, pre-development is defined as the existing conditions of the site at the time of development. The primary goal of this manual is to promote the health, safety, and welfare of the public. To achieve this goal, the objective of this manual is to:

- Reduce stormwater runoff pollutants and protect water quality.
- Reduce downstream overbank flooding and channel erosion.
- Safely convey the design storm and extreme storm events.

For the objectives outlined above, the following stormwater sizing criteria have been developed which are used to size and design structural stormwater controls. Table 2.1 briefly summarizes the criteria.

Table 2.1 Summary of Stormwater Sizing Criteria for Stormwater Control and Mitigation.

Sizing Criteria	Description
Water Quality	Provide post-construction water quality treatment as required by ADEE.
Downstream Flood Protection	Provide peak discharge control of the 2-year, 25-year, and 100-year storm event such that the post-development peak rate does not exceed the pre-development rate. Extend analysis through the zone of influence based on the 10% rule for projects greater than 40 acres.
Level of Service	Provide a level of service for the storm drainage system based on the 25-year storm. Analyze and provide a safe 100-year overflow path for all developments. Based on the Master Street Plan, provide a level of service for cross drainage of 100-year storm for Principal Arterials, as well as for Major and Minor Arterials, 25-year storm for all other streets.

Each stormwater sizing criterion is intended to be used in common conjunction with the others to address the overall stormwater impacts from a development site. Used as a set, the criteria control a range of hydrologic events from the smallest runoff-producing rainfalls to the 100-year storm.

2.1.1 Water Quality

In accordance with the City's Municipal Separate Storm Sewer System (MS4) general permit under NPDES Permit No. ARS000002, the stormwater management system should be designed to reduce, to the Maximum Extent Practicable (MEP), the discharge of pollutants from stormwater flows which exceed pre-development levels and be able to meet any other additional watershed- or site-specific water quality requirements. Any onsite impervious area (draining to a common outlet point) may be treated to meet this requirement – parking areas are preferred due to highest pollutant removal opportunity.

The engineer is required to provide a description of the stormwater control measures (SCMs) that will be installed during the construction process to control pollutants in stormwater discharges that will occur after construction operations have been completed. Structural measures shall be placed on upland soils to the degree attainable. A goal of at least eighty percent 80% removal of total suspended solids from these flows which exceed predevelopment levels should be used in designing and installing stormwater management controls (where practicable).

2.1.2 Downstream Flood Protection

Downstream overbank flood protection shall be provided by controlling the post-development peak discharge rate to not exceed the pre-development rate for the 2-, 25-, and 100-year, 24-hour return frequency storm event. The NOAA Atlas 14 rainfall depths that correspond to these events are 4.19, 5.12, 5.94, 7.14, and 9.17 inches, respectively. Please note that these depths are subject to change once NOAA releases their Atlas 15 rainfall depths.

Existing floodplain areas should be preserved to the extent possible. At the discretion of the Design Review Engineer, analysis of floodplain impacts and additional detention or reduction in post-development peak discharge rates may be required for developments. Downstream analysis, when required, shall extend through the zone of influence based on the 10% rule.

Determining the Overbank Flood Protection Volume

- *Peak-Discharge and Hydrograph Generation:* The hydrograph methods provided in Chapter 3 of this Drainage Manual shall be used to compute the peak discharge rate and runoff for the 2-, 25-, and 100-year, 24-hour storm. The runoff hydrographs shall be routed through the proposed detention/retention structures using appropriate software or methodology as outlined in Chapter 7.

2.1.3 Level of Service

The stormwater system (inlets, storm pipes) shall be designed to meet the 25-year Level of Service and cross drainage (cross culverts, bridges) shall be designed to meet the Level of Service based on the Master Street Plan. The Design Storm is based on the fully developed watershed conditions. The Level of Service for the roadway classifications are as follows:

Table 2.2 Design Storm Street Classification.

Street Classification	Design Storm	
	Cross Drainage	Storm System
Principal Arterials	100-year	25-year
Major and Minor Arterials	100-year	25-year
All other streets	25-year	25-year

The hydraulic grade line (HGL) shall be calculated throughout the storm system to ensure the maximum HGL for the design storm is 1-foot below the throat of each roadway inlet. The starting HGL shall be based on the known or calculated tailwater elevation of the receiving channel or waterbody for the design event. The fully developed 100-year storm must be contained within the designated right of way.

Determining the Adequacy of the Stormwater Management System

- *On-site Storm System Sizing:* The Rational method or SCS TR-55 hydrograph method provided in Chapter 3 shall be used to compute peak discharge rate and runoff for the design storm.
- *Downstream Analysis:* Peak discharges at downstream locations shall be checked and evaluated for any increase in peak flow above pre-development conditions. The downstream check shall extend to the point where the developed site area comprises at least 10% of the total drainage area checked. If the post-developed discharges at the downstream checkpoints exceed pre-development conditions, additional mitigation measures shall be required.
- *System Check:* As a final check, the 100-year, 24-hour storm event shall be routed through the drainage system and stormwater management facilities to determine the effects on the facilities, adjacent property, and downstream, and to confirm adequacy of finished floor elevations for structures. Emergency spillways for structural stormwater controls should be designed to safely pass the fully developed 100-year flow. A positive overflow pathway shall be provided that conveys the flow without damaging structures, utilities, or infrastructure.

2.1.4 Pertinent Ordinances and Regulations

Refer to the following chapters of the City of Little Rock Municipal Code and additional documents for information related stormwater management within the City of Little Rock.

Ordinance No.

Chapter 13 – Floods

Chapter 29 – Stormwater Management and Drainage

Chapter 30 – Streets and Sidewalks

Chapter 31 – Subdivisions

NPDES Permit Number ARS000002

Kaylin Hills et al v. City of Little Rock Planning Commission, 60CV-18-8041, 2019

3. Determination of Storm Runoff

3.1 General

An accurate estimation of storm runoff is critical for planning and development. Atlas 14 rainfall provided by the National Oceanic and Atmospheric Administration (NOAA) will be used in drainage calculations and can be found in section 3.2. Please note that the rainfall depths found in Table 3.2 are subject to change once NOAA releases Atlas 15 rainfall data. Section 3.3 discusses how to estimate flows using the Rational Method. Section 3.4 discusses the Unit Hydrograph method using the Soil Conservation Service (SCS) Technical Release 55 (TR-55) method for hydrograph generation. Section 3.5 details the use of the Runoff Routing Method using Hydrologic Engineering Center Hydrologic Modeling System (HEC-HMS) software for runoff estimation. Table 3.1 summarizes the accepted approaches of runoff determination; however, the Director of Planning and Development may approve other engineering methods when they are shown to be comparable to the following methods.

Table 3.1 Hydrology Methodology.

Methodology (Section)	Watershed Size
Rational Method (3.3)	Less than 40 acres.
Unit Hydrograph (SCS) Method (3.4)	Up to 200 acres.
Runoff Routing (HEC-HMS) Method (3.5)	Greater than 200 acres.

Section 3.7 provides a summary of acceptable runoff analysis software. Other software that utilizes the recommended methodology per Table 3.1 may be used at the City Design Review Engineer's discretion if results are shown to be comparable. The runoff analysis must show the results for the fully developed 2-, 25-, and 100-year, 24-hour storm events.

3.2 Precipitation Data

Rainfall data from the latest, fully released version of the NOAA Precipitation Frequency Atlas will be used in drainage calculations. The current version, NOAA Atlas 14, is expected to be replaced by Atlas 15 which will be fully released in 2027. The NOAA Atlas 14 rainfall depths for the Clinton National Airport Gauge in Little Rock, Arkansas are provided in Table 3.2. Intensity-Duration-Frequency curve equation coefficients can be found in Table 3.3. The following sections 3.3-3.6 provide more detail on how to use these values according to the appropriate runoff methodology.

Once the drainage basin is defined, the next step in the hydrologic analysis is an estimation of the rainfall that will fall on the basin for a given time period. The duration, depth, and intensity of the rainfall are defined below:

- ***Duration (hours)*** – Length of time over which rainfall (storm event) occurs.
- ***Depth (inches)*** – Total amount of rainfall occurring during the storm duration.
- ***Intensity (inches per hour)*** – Depth divided by the duration.

The frequency of a rainfall event is the recurrence interval of storms having the same duration and volume (depth). This can be expressed in terms of annual chance or return period.

- ***Annual Chance*** – *Percent chance that a storm event having the specified duration and volume will be exceeded in one year/years (e.g., a "25-year" storm has a 4-percent-annual-chance of occurring in any given year).*

- ***Return Period*** *– Average length of time between events that have the same duration and volume* (e.g., 10-year event).

Thus, if a storm event with a specified duration and volume has a 1 percent chance of occurring in any given year, it may be termed a 1-percent-annual chance event. The use of the phrase “return period” is discouraged because it gives a false impression that storm events cannot occur more frequently than the corresponding return periods.

Rainfall depths for the 24-hour duration storm for the City of Little Rock are provided in Table 3.2 below. If rainfall is required for other than 24-hour duration, it can be taken from the NOAA Atlas 14 Precipitation Frequency Data Server at https://hdsc.nws.noaa.gov/pfds/.

Table 3.2 Atlas 14 Rainfall Depths for 24-hour Duration Storm.

Storm Event	2-year	5-year	10-year	25-year	50-year	100-year
Depth (in)	4.19	5.12	5.94	7.14	8.13	9.17

Rainfall intensity is selected based on the design rainfall duration and frequency of occurrence. The design duration is equal to the time of concentration for a drainage area under consideration. Once the time of concentration is known, the design intensity of rainfall may be determined from the rainfall intensity equation below.

The rainfall intensity is calculated using the formula below:

$$I = \frac{b}{(t_c + d)^e} \qquad \textbf{Eq. 3.1}$$

Where: I = precipitation intensity (inches per hour)

t_c = time of concentration (minutes)

e,b,d = variable coefficients (see table below)

Table 3.3 includes the coefficients used in the IDF curve equation. The variable coefficients *e, b*, and *d* in the IDF curve equation are derived by fitting the equation to data on the frequency and intensity of storm events from Atlas 14 for the City of Little Rock.

The majority of drainage sub-basins within the City of Little Rock have a relatively short time of concentration. In general, basins with computed times of concentration in excess of 60 minutes (maximum) should be subdivided to create smaller sub-basins for more accurate computation of peak discharge. Where a design time of concentration for a watershed sub-basin exceeds 30 minutes, the applicability of the Rational Method shall be justified with documentation if it is used. In sub- basins with significant channel or overland storage, errors may be introduced by the use of the Rational Method.

Table 3.3 Atlas 14 e,b,d Variable Coefficients.

Return Period	Variable		
	e	***b***	***d***
2yr	0.625	22.461	3.488
5yr	0.628	27.071	3.592
10yr	0.626	30.302	3.550
25yr	0.620	34.032	3.395
50yr	0.619	37.392	3.507
100yr	0.612	39.450	3.253
500yr	0.598	44.067	2.969

3.3 Rational Method

The Rational Method can be used to estimate stormwater runoff peak flows for the design of gutter flows, drainage inlets, storm pipes, culverts, and roadside ditches. It is most applicable to small, highly impervious areas. The Rational Method was not developed for storage design or any application where a more detailed routing procedure is required. However, for the design of small detention facilities, the Modified Rational Method may be used for sites up to 40 acres. Detention design methodology for the Modified Rational Method is included in Chapter 7 of this manual.

Basic assumptions associated with use of the Rational Method are as follows:

1. The computed peak rate of runoff to the design point is a function of the average rainfall intensity during the time of concentration to that point.
2. The time of concentration (see Section 3.4.1) is the critical value in determining the design rainfall intensity and is equal to the time required for water to flow from the hydraulically most distant point in the watershed to the point of design.
3. Infiltration, represented by the runoff coefficient (C), is uniform during the entire duration of the storm event.
4. The rainfall intensity, I, is assumed to be uniform for the entire duration of the storm event and is uniformly distributed over the entire watershed area.

The Rational Method Equation, shown below, estimates the peak flow based on the runoff coefficient, drainage area, and rainfall intensity associated with the watershed time of concentration.

$$\mathbf{Q = CiA} \qquad \textbf{Eq. 3.2}$$

Where: *Q* – Peak flow (cfs).

C – Dimensionless runoff coefficient.

i – Rainfall intensity (in/hr).

A – Drainage area (acres).

The value of C assigned for the drainage area should be an area-weighted average accounting for all the proposed land uses within the project area. Typical values of C are provided in Table 3.4 for various land use conditions, slopes, and hydrologic soil types.

Table 3.4 Runoff coefficients (C-values) for various land uses.

Land Use Description	Slope, %	Hydrologic Soil Group		
		A/B	C	D
Lawns				
	0-2	0.15	0.25	0.35
	2-7	0.25	0.35	0.40
	> 7	0.30	0.40	0.45
Forest				
	0-2	0.23	0.28	0.33
	2-7	0.31	0.36	0.41
	> 7	0.36	0.41	0.46
Unimproved areas				
Meadow		0.20-0.4	0.25-0.45	0.30-0.55
Row crops		0.25-0.6	0.35-0.75	0.40-0.80
Business				
Downtown areas		0.7	0.8	0.9
Neighborhood areas		0.5	0.6	0.7
Residential				
8 lots / acre		0.67	0.71	0.76
4 lots / acre		0.46	0.52	0.61
3 lots / acre		0.40	0.47	0.57
2 lots / acre		0.35	0.43	0.54
Suburban (1 lot / acre)		0.30	0.38	0.50
Multi-units, detached		0.70	0.75	0.80
Multi-units, attached		0.75	0.80	0.85
Apartments		0.65	0.70	0.75
Industrial				
Light areas		0.60	0.75	0.85
Heavy areas		0.80	0.85	0.90
Parks, cemeteries		0.25	0.35	0.45
Schools, Churches		0.70	0.75	0.80
Railroad yard areas		0.20	0.35	0.50
Asphalt & Concrete Pavements, Roofs.			0.95	
Brick Pavement or Gravel (compacted subgrade)			0.85	
Graded or no plant cover				
	0-2	0.25	0.30	0.35
	2-7	0.40	0.50	0.60
	> 7	0.50	0.65	0.80

1. State of Georgia (2001).

2. Oregon Dept. of Transportation (2005).
3. Arkansas Highway and Transportation Department (1982).
4. Virginia Department of Transportation (2002).

- Selection of design rainfall intensity in the Rational Method shall be based on the required design frequency and calculated using the IDF equation provided in Section 3.2.
- Drainage area computations for runoff estimation should be based on the best available data. Where more recent and more detailed site-specific topographic data is not available, the most recent publicly available topographic contour data should be used (www.pagis.org).

The coefficients given in Table 3.4 are applicable for storm events up to the 10-year frequency. Less frequent, higher intensity storms require modification of the coefficient because infiltration and other losses have a proportionally smaller effect on runoff. The adjustment of the Rational Method for use with major storms can be made by multiplying by a frequency factor, C_f. The Rational Formula now becomes:

$$\mathbf{Q = C_f CiA} \qquad \textbf{Eq. 3.3}$$

The C_f values are provided in Table 3.5. The product of C_f times C shall not exceed 1.0.

Table 3.5 Frequency Factors (C_f) for Rational Equation.

Recurrence Interval	C_f
10-year or less	1.0
25-year	1.1
50-year	1.2
100-year	1.25

Note: Cf * C shall not exceed 1.0

Source: Georgia Stormwater Manual

3.4 Unit Hydrograph (SCS) Method

The Soil Conservation Service (SCS) hydrologic method is based on a synthetic unit hydrograph. The SCS Technical Release 55 (TR-55) approach for runoff determination was developed specifically for use in urbanized and urbanizing areas. Multiple software programs are available that utilize the SCS hydrologic method. A detailed examination of the capabilities and limitations of various software is required to ensure that the appropriate software is used.

In general, the SCS approach considers time distribution of rainfall, initial rainfall losses (infiltration and depression storage), and allows for varying infiltration throughout the storm interval. Further details are provided in the National Engineering Handbook (NRCS, 2004). The SCS method directly relates runoff to rainfall amounts through use of curve numbers (CNs) based on Hydrologic Soil Group (HSG) soil type and on land use.

A typical application of the SCS method includes the following basic steps:

- Determine curve numbers for different land uses and soil types within the drainage area.
- Calculate time of concentration to the drainage area outlet point.
- Use the Type II rainfall distribution to determine excess rainfall.
- Develop the direct runoff hydrograph for the drainage basin.

This method can be used both to estimate stormwater runoff peak discharges and to generate hydrographs for routing stormwater flows. This method may be used for design applications including open channels, small drainage ditches, energy dissipation, storm drain systems, storm sewer networks, inlet and outlet structures, and storage facilities.

Design rainfall may be input into various programs that use the SCS method. For the purpose of pre- and post-development runoff comparisons, the following design storm data shall be used:

Rainfall amounts for 24-hour storm durations with recurrence intervals of 2-, 25-, and 100- years. The appropriate rainfall distribution for the City of Little Rock is Type II.

3.4.1 Time of Concentration and Travel Time

Time of Concentration must be calculated using SCS TR-55 Method only, other methods shall not be allowed. Time of Concentration (T_C) is the sum of Travel Time values for the various consecutive flow segments: sheet flow, shallow concentrated flow, and channel flow.

$$T_c = T_s + T_{sc} + T_{ch} \quad \textbf{Eq. 3.4}$$

Where: Tc = Time of Concentration

Ts = Sheet Flow Time

Tsc = Shallow Concentrated Flow Time

Tch = Channel Flow Time

The time of concentration longest flow path shall be provided for each basin and shall be updated as required between pre- and post-development computations. The flow path should be the flow path that best represents the basin which may not be the longest (in length) flow path. The minimum time of concentration is 5 minutes.

a. **Sheet Flow**
The maximum length of sheet flow is 100 feet. For sheet flow of less than 100 feet, use Manning's Kinematic solution (Overtop and Meadows 1976) to compute sheet flow travel time:

$$T_s = \frac{0.007(nL)^{0.8}}{(P_2)^{0.5} s^{0.4}} \quad \textbf{Eq. 3.5}$$

Where: n = Manning's Roughness Coefficient (see TR-55 for roughness coefficients)
L = flow length (ft)

P2 = 2-year, 24-hour rainfall (in)

s = slope of hydraulic grade line (land slope, ft/ft)

b. **Shallow Concentrated Flow**
After a maximum of 100 feet, sheet flow usually becomes shallow concentrated flow. Shallow Concentrated flow transitions to channel flow when the flow reaches the curb or a defined swale or channel. Shallow Concentrated flow shall be calculated based on the average velocity along the flow path using the following equation:

$$Tsc = \frac{L_f}{3600\,V} \quad \textbf{Eq. 3.6}$$

Where: L_f = Flow length (ft)

V = Velocity (ft/sec)

The average velocity for shallow concentrated flow may be determined using the equations below:

Paved $$V = 20.33(S)^{0.5}$$ **Eq. 3.7**

Unpaved $$V = 16.13\ (S)^{0.5}$$ **Eq. 3.8**

Where: V = Velocity (ft/sec)

S = watercourse slope (ft/ft)

c. **Channel Flow Time**

The channel flow time can be calculated using the following formula:

$$Tch = \frac{L_f}{3600\,V}$$ **Eq. 3.9**

Where: L_f = Flow length (ft)

V = Velocity (ft/sec)

The velocity in the channel can be calculated using the Manning's equation.

$$V = \frac{1.486}{n} R^{2/3} S^{1/2}$$ **Eq. 3.10**

Where: Q = total discharge in cubic feet per second

n = coefficient roughness

A = cross-sectional area of conduit in feet

R = hydraulic radius of channel in feet

S = slope of energy line in feet per foot

V = velocity in feet per second

3.4.2 Runoff Factor

The principal physical watershed characteristics affecting the relationship between rainfall and runoff are land use, land treatment, soil types, and land slope. The SCS method uses a combination of soil conditions and land uses (ground cover) to assign a runoff factor to an area. These runoff factors, called runoff curve numbers (CN), indicate the runoff potential of an area. Soil properties influence the relationship between runoff and rainfall since soils have differing rates of infiltration. Based on infiltration rates, the SCS has divided soils into four hydrologic soil groups (HSGs).

Group A Soils having a low runoff potential due to high infiltration rates. These soils consist primarily of deep, well-drained sands and gravels.

Group B Soils having a moderately low runoff potential due to moderate infiltration rates. These soils consist primarily of moderately deep to deep, moderately well to well drained soils with moderately fine to moderately coarse textures.

Group C Soils having a moderately high runoff potential due to slow infiltration rates. These soils consist primarily of soils in which a layer exists near the surface that impedes the downward movement of water or soils with moderately fine to fine texture.

Group D Soils having a high runoff potential due to very slow infiltration rates. These soils consist primarily of clays with high swelling potential, soils with permanently high-water tables, soils with a claypan or clay layer at or near the surface, and shallow soils over nearly impervious parent material. Embankments designated or identified as "hillside" in the City shall be classified as Hydrologic Soil Group D.

For use in hydrologic computations, the most recent soil distribution data can be viewed online and downloaded from the NRCS Web Soil Survey (USDA NRCS).

The effects of urbanization on the natural hydrologic soil group should be accounted for in design. Runoff curve numbers for different land uses are provided in Table 3.6. In all areas disturbed by heavy equipment used during construction or where grading will mix the surface and subsurface soils, the curve numbers shall be shifted to the next higher HSG for design.

Area-weighted composite curve numbers shall be calculated for each drainage area and used in the analysis based on variations in soil type and land use. It should be noted that when composite curve numbers are used, the analysis does not take into account the location of the specific land uses. The drainage area is assigned a composite uniform land use represented by the composite curve number. However, if the spatial distribution of land use is important to the hydrologic analysis, then sub-basins corresponding to the distribution (to the extent possible) should be developed and separate sub-basin hydrographs developed and routed to the study point.

The curve numbers in Table 3.6 are based on directly connected impervious area. An impervious area is considered directly connected if runoff from it flows directly into the drainage system or occurs as concentrated shallow flow that runs over pervious areas then into a drainage system.

It is possible that curve number values from urban areas could be reduced by disconnecting impervious areas and allowing such runoff to sheet flow over additional significant pervious areas prior to entering the drainage system. Additional information on this approach is described in Chapter 8. The CNs provided for various land cover types were developed for typical land use relationships based on specific assumed percentages of impervious area. These CN values were developed on the assumptions that:

- Pervious areas that are not disturbed by construction equipment are equivalent to pasture in good hydrologic condition, and
- Impervious areas have a CN of 98 and are directly connected to the drainage system.

If Low Impact Development, or Green Infrastructure, stormwater controls or practices are implemented in design, the impact of such features in reducing overall stormwater runoff may be accounted for. Practices resulting in increased infiltration will decrease overall runoff and this can be addressed by modifying the Curve Number.

If the actual impervious area percentage for the proposed design exceeds the proportion assumed for land uses in Table 3.6, a composite CN shall be computed based on actual percentage rather than using the table values.

3.5 Runoff Routing (HEC-HMS) Method

For larger watersheds, runoff routing methodology is required to sub-divide the watershed into small sub-catchments to model the runoff generation and flow routing. These models account for the areal distribution of rainfall, land use, catchment, and stream characteristics. The Hydrologic Modeling system (HEC-HMS) is a free hydrologic modeling software available from the USACE. It accommodates significant complexity, and a wide variety of options are available; therefore, it is included as an available method. This method may be applied for developing peak discharge and hydrograph information to use in drainage infrastructure and detention design. For runoff computations, the model provides several options for the following components:

i. Various precipitation models – observed conditions, frequency-based, upper limit event.
ii. Runoff volume estimation models.
iii. Direct runoff models that account for overland flow, storage, and energy losses.
iv. Hydrologic routing models.
v. Modeling of natural confluences and bifurcations.
vi. Water-control measures including diversions and storage facilities.

In HEC-HMS a runoff hydrograph for a subbasin is computed from meteorologic data by subtracting losses and transforming excess precipitation. The preferred method for both the Loss Method and the Transform Method is the SCS Curve Number method. However, other methods or software may be accepted at the discretion of the Director of Planning and Development. Complex systems with multiple basins may also require Reaches to connect Subbasins. Reach elements represent segments of a stream or river through which flow is routed. The selection of the reach routing method should consider channel slope, the influence of backwater and whether there is a need for the model method to account for in-line channel storage. The HEC-MHS user's manual provides detailed descriptions and parameters for the various loss, transform, and routing methods.

3.6 Regression Equations

Regression equations are used to determine annual exceedance probability discharges for un-gaged streams in Arkansas based on upon the physical, climatic, and land use of characteristics of a drainage basin. The US Geological Survey (USGS) periodically updates the regional regression equations based on annual peak-discharge data through the latest available water year. There are four flood regions in Arkansas. Pulaski County is split between Region A and Region D, with the majority of the City of Little Rock being within Region A. The current regional regression equations can be found on their website: https://pubs.usgs.gov/publication/sir20165081.

USGS produces a web application, StreamStats, that can be used to delineate drainage areas and estimate design flows based on the regression equations for the flood region. However, regression equations and StreamStats flows should only be used for preliminary estimates and should not be used for final design.

3.7 Offsite Watershed Basins

For offsite basins that pass through the proposed development, the Rational Method may be used for calculation of design flows for basins up to 200 acres. The Rational Method flows can be used to design conveyance of offsite flows through the site and connecting into existing downstream systems. Onsite and detention flows shall be calculated according to Table 3.1. If the Rational method shows the downstream system is undersized, the Director of Planning and Development may allow onsite over-detention, offsite improvements, or reduced level-of-service for offsite conveyance. For over-detention and offsite improvements, cost sharing will be in accordance with City ordinances.

Table 3.6 TR-55 Runoff Curve Numbers[1] (CN)

Cover type and hydrologic condition[2]	Average percent impervious area[3]	Curve numbers for hydrologic soil groups[1]			
		A	B	C	D
Cultivated land					
Without conservation treatment		72	81	88	91
With conservation treatment		62	71	78	81
Pasture or range land					
Poor condition		68	79	86	89
Good condition		39	61	74	80
Meadow					
Good condition		30	58	71	78
Wood or forest land					
Thin stand, poor cover		45	66	77	83
Good cover		30	55	70	77
Open space (lawns, parks, golf courses, cemeteries, etc.)[4]					
Poor condition (grass cover <50%)		68	79	86	89
Fair condition (grass cover 50% to 75%)		49	69	79	84
Good condition (grass cover > 75%)		39	61	74	80
Impervious areas:					
Paved parking lots, roofs, driveways, etc. (excluding right-of-way)		98	98	98	98
Streets and roads					
Paved; curbs and storm drains (excluding right-of-way)		98	98	98	98
Paved; open ditches (including right-of-way)		83	89	92	93
Gravel (including right-of-way)		76	85	89	91
Dirt (including right-of-way)		72	82	87	89
Urban districts					
Commercial and business	85%	89	92	94	95
Industrial	72%	81	88	91	93
Residential districts by average lot size					
1/8 acre or less (town houses)	65%	77	85	90	92
1/4 acre	38%	61	75	83	87
1/3 acre	30%	57	72	81	86
1/2 acre	25%	54	70	80	85
1 acre	20%	51	68	79	84
2 acres	12%	46	65	77	82
Developing urban areas and newly graded areas (pervious areas only, no vegetation).		77	86	91	94

1. Antecedent Moisture Condition II, and Ia = 0.2S.

2. The average percent impervious area shown was used to develop the composite CNs. Other assumptions are as follows: impervious areas are directly connected to the drainage system, impervious areas have a CN of 98, and pervious areas are considered equivalent to open space in good hydrologic condition.
3. CNs shown are equivalent to those of pasture. Composite CNs may be computed for other combinations of open space cover type.

Source: USDA Technical Release 55 (TR-55)

3.8 Runoff Analysis Software

Computer software shall be used for stormwater runoff analyses in conformance with design criteria to meet the design standards of the City of Little Rock and this Drainage Criteria Manual. Software such as HEC-HMS, HEC-RAS, EPA-SWMM, XPSWMM, Infoworks ICM, Bentley StormCAD, HydroCAD, Bentley Pond-Pack, Autodesk Storm and Sanitary Analysis or comparable may be used for runoff analyses. Use of two-dimensional (2-D) models, with approval by the Director of Planning and Development, may be used when necessitated by site conditions with complicated overland flow paths or other special circumstances. Within Special Flood Hazard Areas, FEMA-approved hydrologic models should be used. A list of FEMA- approved software can be found on their website: https://www.fema.gov/flood-maps/products-tools/numerical-models

For all submittals, the model input and output data provided shall be clearly, concisely, and consistently organized and labeled based on percent-annual-chance events or design storms. A spatial file or schematic that identifies and references subbasins shall be provided that identifies drainage areas for which data are computed. Minimum output data required shall correspond to the Drainage Report requirements detailed in Appendix A. Example tables depicting required input/output data to be reported are provided within Appendix A.

3.9 Rain-on-Mesh

Traditionally, hydrologic and hydraulic (H&H) models have been developed separately. The hydrologic model estimates inflow boundary conditions (rainfall runoff, inflow hydrographs, etc.) and the hydraulic model routes overland and stream flow to estimate water surface elevations, flow velocities, and flood extents. Recent software developments allow both the hydrology and hydraulics to be conducted within one model framework. Rather than delineating basins and calculating runoff at discreet points, rain-on-mesh or rain-on-grid methodology is where precipitation is applied directly to the 2-D grid or mesh of the model. Generally, there are two approaches to this methodology: excess precipitation and 2-D infiltration. Use of a 2-D rain-on-mesh model must be approved by the Director of Planning and Development.

3.8.1 Excess Precipitation

For the excess precipitation method, infiltration losses based on soil and land use data are calculated for the entire basin in HEC-HMS. Then the excess precipitation hyetograph from the HEC-HMS model is applied to the 2-D mesh in the hydraulic model. This method is not recommended as it does not account for the spatial variations in infiltration because the losses are averaged over the entire basin. It also does not account for the timing variation in runoff from directly connected impervious area.

3.8.2 Two-Dimensional Infiltration

For the two-dimensional infiltration method, infiltration losses are calculated within the hydraulic model. A direct rainfall hyetograph is applied to the 2-D mesh and infiltration is subtracted based on the spatial infiltration layer within the model. The infiltration method used within the 2-D model should be the SCS Curve Number method. Other methods such as Deficit & Constant or Green-Ampt may be used with

approval by the Director of Planning and Development. The infiltration layer is developed by combining the land use layer and the soils layer. To properly account for directly connected impervious area, the land use layer should not use a composite curve number. Instead, the land use layer should include the impervious percentage and the runoff Curve Number for the pervious area only. This is especially important in highly developed, urban areas where runoff will occur at the very beginning of storms due to impervious areas that are directly connected to the storm runoff system.

4. Storm Drainage System Design

4.1 General

The purpose of this section is to focus on the proper hydraulic design of storm drains, the collection system, and appurtenances. The storm drainage system consists of inlets, grates, parking lots, street gutters, roadside ditches, small channels and swales, and underground pipe systems which collect stormwater runoff and transport it to structural control facilities and/or the major drainage system (i.e., natural waterways, large man-made conduits, large water impoundments).

This section provides criteria and guidance for the design of minor drainage system components including:

- Street and roadway gutters
- Stormwater inlets and grates
- Storm drainpipe systems

Roadside ditch, channel and swale design criteria and guidance are covered in Chapter 5, *Open Channel Design*.

Procedures for performing gutter flow calculations are based on a modification of Manning's Equation. Inlet capacity calculations for grate, curb and combination inlets are based on information contained in HEC-22 (USDOT, FHWA, 2009). Storm drainage system design may be based on either the use of the Rational Formula for gutters and inlets, subject to the area limitations provided in Chapter 3, or the SCS or TR-55 methodologies.

4.2 Storm System Design Requirements

In preparation of storm sewer design, the following list of minimum requirements must be prepared for a storm sewer design:

1. Engineering report displaying relevant calculations and design. See Example Report in Appendix A.
2. A drainage area map at a scale of 1” = 200’ with 2-foot minimum contour intervals using USGS datum for areas less than 100 acres or a plan of the drainage area at a scale of 1” = 500’ with 5-foot minimum contour intervals for larger areas.
3. This plan shall include all existing and proposed street, drainage, and grading improvements with flow quantities and direction at all critical points.
4. All areas and subareas for drainage calculations shall be clearly distinguished.
5. Complete hydraulic data showing all calculations, including a copy of all graphs used for your calculations shall be submitted.
6. A plan and profile of all proposed improvements at a scale of 1” = 50’ horizontal and 1” = 5’ vertical shall be submitted. This plan shall include the following:
    a. Locations, sizes, flowline elevations and grades of pipes, channels.
    b. Boxes, manholes, and other structures drawn on standard plan-profile sheets.
    c. Existing and proposed ground line profiles.
    d. List of the kind and quantities of materials.
    e. Typical sections of all boxes and channels.
    f. Location of property lines, street paving, sanitary sewers, and other utilities.

7. A field study of the downstream capacity is required of all drainage facilities if flows are increased. The effect of additional flow from the area to be improved shall be submitted. If effects endanger property or life, the problem must be resolved before the plan will be given approval. Downstream effects shall be evaluated to the point where the drainage area of the site comprises 10% of the total drainage area.
8. Stormwater flow quantities in the street shall be shown at all street intersections and all inlet openings and locations where flow is removed from the streets.

Stormwater system design shall include the hydraulic calculations for all inlet openings and street capacities. The gutter flow shall be limited according to Section 4.3, Street and Roadway Gutters. Any additional information deemed necessary by the City Design Review Engineer for an adequate consideration of the storm drainage effect on the City of Little Rock and surrounding areas must be submitted.

4.2.1 Design Storm

The design storm for the storm drainage system is the 25-year event. The storm drainage system refers to the infrastructure that collects local runoff which includes inlets, gutters, roadside ditches, swales, and underground pipe. The design storm for cross drainage is based on the road classification in the Master Street Plan. However, if a street is the only means of ingress/egress then the cross drainage shall be sized for the 100-year storm regardless of street classification Cross drainage refers to a pipe, culvert, or bridge that conveys water from one side of a roadway to another. The current version of the Master Street Plan can be found on the City of Little Rock's website: https://www.littlerock.gov/business/planning-and-development/2023-master-street-plan-update-transportation-development/

Design storm requirements are provided in Table 4.1. The fully developed conditions shall be used to calculate flows for the appropriate design storm frequencies. Reasonable assumptions must be made for off-site flows. The 100-year design storm event shall be used as the check storm to estimate runoff for routing to evaluate effects on the facilities, adjacent property, floodplain encroachment and downstream areas. For the 100-year event, ensure that storm pipe systems will safely convey flows that are in excess of pipe design flows without damaging structures or flooding major roadways. The 100-year storm shall not be conveyed through driveway cuts or across private property but shall remain within the ROW and/or a drainage easement. No fences, portable storage buildings, large landscaping features (i.e., boulders, decorative rock), or other obstructions may be placed within drainage easements.

Table 4.1 Design Storm for Street Classification

Street Classification	Design Storm	
	Cross Drainage	Storm System
Principal Arterials	100-year	25-year
Major and Minor Arterials	100-year	25-year
All other streets	25-year	25-year

Note: Fully developed 100-yr flow must be contained within the right of way (ROW)

4.3 Street and Roadway Gutters

The location of inlets and permissible flow of water in the streets should be related to the extent and frequency of interference to traffic and the likelihood of flood damage to surrounding property. Effective drainage of street and roadway pavements is essential to pavement longevity and traffic safety. Surface drainage is a function of transverse and longitudinal pavement slope, pavement roughness, inlet spacing, inlet capacity, and adequate subsurface drainage. The design of these elements is dependent on storm frequency and the allowable spread of stormwater on the pavement surface. To compute gutter flow, the

Manning's equation is integrated for an increment of width across the section. The resulting gutter equation is:

$$Q = (K_u/n) S_x^{1.67} S_L^{0.5} T^{2.67} \quad \text{Eq. 4.1}$$

Where: Q = flow rate

K_u = 0.56

n = Manning's coefficient (Table 4.2)

S_x = Cross slope (ft/ft)

S_L = Longitudinal slope (ft/ft)

T = Width of flow (spread) (ft)

Table 4.2 Manning's n for Street and Pavement Gutters.

Type of Gutter or Pavement	Manning's n
Concrete gutter, troweled finish	0.019
Asphalt Pavement:	
Smooth texture	0.013
Rough texture	0.016
Concrete gutter-asphalt pavement:	
Smooth	0.013
Rough	0.015
Concrete pavement:	
Float finish	0.014
Broom finish	0.016
For gutters with small slope, where sediment may accumulate, increase above values of "n" by	0.002

Source: HEC-22

4.3.1 Permissible Spread of Water

Inlets shall be installed at low points and at such intervals to provide the appropriate clear traffic lane per street classification in each direction based upon peak discharges from the 25-year design storm. Minimum lane clearance requirements are provided in Table 4.3. All computations for the 25-year design storm and 100-year, 24-hour storm shall be provided.

Table 4.3 Flow Spread Limits for Inlets

Street Classification	Minimum clear space
Principal and Arterial Streets	Two 12-feet traffic lanes, one in each direction, independent of curb and gutter
Collector Streets	One 12-feet traffic lane within 6 feet of roadway centerline
All other streets	One 10-feet traffic lane within 4 feet of roadway centerline

4.3.2 Flow Bypass

Bypass flow occurs when storm sewer inlets do not capture 100% of the flow upstream of their location. A variety of factors, including organic debris, gutter flow rate, longitudinal slope, and inlet type/geometry, play a role in the capture efficiency of an individual inlet. Flow bypassing each inlet must be included in the total gutter flow to the next inlet downstream. A bypass of 10 to 20% per inlet will result in a more economical drainage system.

4.3.3 Minimum and Maximum Velocities

To ensure cleaning velocities at very low flows, the gutter shall have a minimum longitudinal slope of 0.005 feet per foot (0.5%).

The maximum velocity of gutter flow shall be 10 feet per second. Along sharp horizontal curves, peak flows tend to jump behind the curb line at driveways and other curb breaks. Water running behind the curb line can result in considerable damage due to erosion and flooding. Inlets should be placed upstream of horizontal curves to capture gutter flow and limit flow along the curve.

4.4 Storm Drain Inlets

The primary purpose of storm drain inlets is to intercept excess surface runoff and deposit it in a drainage system, thereby reducing the possibility of surface flooding.

The most common location for inlets is in streets which collect and channelize surface flow making it convenient to intercept. Because the primary purpose of streets is to carry vehicular traffic, inlets must be designed so as not to conflict with that purpose.

The following guidelines shall be used in the design of inlets to be located in streets:

1. Grate inlets shall not be used in a roadway.
2. Inlets shall not be placed on the radius of a curve.
3. Placing inlets downstream of a radius should be avoided.
4. Design and location of inlets shall take into consideration pedestrian and bicycle traffic.
5. Inlet design and location must be compatible with the spread limitations presented in Table 4.3.

4.4.1 Classification

Inlets are classified into three major groups, mainly: inlets in sumps (Type A), inlets on grade without gutter depression (Type B), and inlets on grade with gutter depression (Type C). Each of the three major classes include several varieties, shown in Table 4.4. Recessed inlets are identified by the suffix (R, i.e.: A-1 (R)). The term "continuous grade" or "on grade" refers to an inlet located on the street with a continuous slope past the inlet with water entering from one direction. The "sump" condition exists when street grade is less than 1% or the inlet is located at a low point allowing water to enter from both directions.

The Department of Planning and Development review of a proposed drainage plan shall include examination of the supporting inlet computations. Inlet calculations must be submitted on separate tabulations sheets convenient for review and use of a permanent record in order to speed review.

Table 4.4 Stormwater Inlet Types

Inlets in Sumps	
Type A-1	Curb Opening
Type A-2	Grate
Type A-3	Combination (Grate and Curb Opening)
Type A-4	Drop
Type A-5	Drop (Grate Covering)
Inlets on Grade Without Gutter Depression	
Type B-1	Curb Opening
Type B-2	Grate
Type B-3	Combination (Grate and Curb Opening)
Inlets on Grade with Gutter Depression	
Type C-1	Curb Opening
Type C-2	Grate
Type C-3	Combination (Grate and Curb Opening)

4.4.2 Inlets in Sumps

Inlets in sumps are inlets placed in low points of surface drainage to relieve ponding. Inlets with a 5-inch depression located in streets of less than one percent (1.0%) grade, shall be considered inlets in sumps. The capacity of inlets in sumps must be known in order to determine the depth and width of ponding for a given discharge. Capacity calculations should be based on HEC-22 methodology or manufactures capacity curves.

Inlets in sumps function like weirs for shallow depths. The hydraulic capacity of a curb opening inlet or a vaned grate inlet operating as a weir is expressed as:

$$Q_i = C_w L_w d^{1.5} \qquad \textbf{Eq. 4.2}$$

Where: Q_i = inlet capacity (CFS)

C_w = weir discharge coefficient

Lw = weir length (ft), length of inlet opening acting as a weir

d = flow depth (ft)

Curb opening inlets and drop inlets in sumps have a tendency to collect debris at their entrances. For this reason, the calculated inlet capacity shall be reduced by 20 percent to allow for clogging. Grate inlets have a tendency to clog when flows carry debris such as leaves and papers. For this reason, the calculated inlet capacity of a grate inlet shall be reduced by 50 percent to allow for clogging.

Table 4.5 Sump Inlet Discharge Variables and Coefficients for weir inlets.

Weir Inlet Types	C_w	L_w	Weir equation valid for
Curb opening inlet	3.00	L	d < h
Recessed curb opening inlet	2.30	L+1.8W*	d < h + a
Vane Grate Inlet	3.00	L+2W	d < 1.79 (A_o/L_w)

Definition of terms:

L = length of curb opening

a = depth of curb depression

h = height of curb opening

Ao = clear opening area

d = depth of water at curb opening

W = width of grate

W* = lateral width of recessed section

As the depth of stormwater increases, inlet sumps begin to function like an orifice. HEC-22 provides guidance on the transition region based on significant testing. At depths above 1.4 times the opening height, the inlet operates as an orifice and between these depths, transition between weir and orifice flow occurs. The hydraulic capacity of a curb opening inlet or a vane grate inlet operating as an orifice is expressed as:

$$Q_i = C_o A_o \sqrt{2gd} \qquad \textbf{Eq. 4.3}$$

Where: Q_i = inlet capacity (cfs)

C_o = orifice coefficient

A_o = orifice area (ft^2)

g = gravitational acceleration (32.2 ft/sec^2)

d = characteristic depth (ft) defined in Table 4.6

Table 4.6 Sump Inlet Discharge Variables and Coefficients for orifice inlets.

Orifice Inlet Types	C_o	A_o	Orifice equation valid for
Curb opening inlet or recessed curb opening inlet	0.67	hL	di* > 1.4h
Vane Grate Inlet	0.67	Clear opening area	d** > 1.79(Ao/Lw)

* di = depth of water at curb opening

** d = depth of water over grate

Note: The orifice area (A_o) should be reduced where clogging is expected.

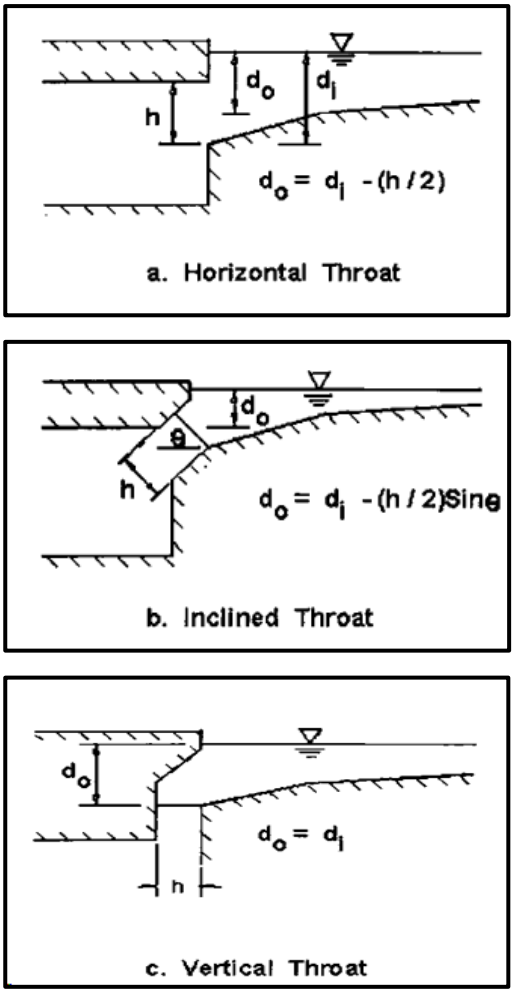


Figure 4.1. Curb-Opening Inlets

4.4.3 Inlets on Grade

Curb opening inlets are effective in the drainage of roadway pavements and in parking lots where flow depth at the curb is sufficient for the inlet to perform efficiently. Curb openings are relatively free of clogging tendencies and offer little interference to traffic operation. Street inlets shall be depressed 4 inches with a 12-feet transition upstream and 4-foot transition downstream. Where stormwater flow approaches an arterial street or tee intersection, an inlet is required.

Inlet dimensional requirements: clear throat opening shall be 6 inches in height and 4-feet minimum length. For all throat extensions, clear dimensions shall be 6 inches in height and 3 feet, 6 inches in length. City of Little Rock standard drawings and details shall be used.

Inlets with extensions shall have a maximum clear opening dimension that complies with the City of Little Rock standard drawings. For inlets with extensions, the spacing of 4-inch stools shall not exceed 4 feet in length. If additional length is needed to accommodate City spread and ponding depth requirements, additional inlets shall be added upstream. No clogging factor is required to be applied for curb inlets on grade. For calculation of the interception capacity of inlets on grade, refer to HEC-22.

Flow bypassing each inlet must be included in the total gutter flow to the inlet downstream. A bypass of 10 to 20 percent per inlet will result in a more economical drainage system.

4.4.4 Combination Inlets

The capacity of a combined inlet Type A-3 consisting of a grate and curb opening inlet in a sump shall be considered to be the sum of the capacities. When the capacity of the gutter is not exceeded, the grate inlet accepts the major portion of the flow. Under severe flooding conditions the curb inlet will accept most of the flow.

Combination inlets and sumps have a tendency to clog and collect debris at their entrance. For this reason, the calculated grate capacity of the inlet shall be reduced by 50 percent to allow for this clogging.

4.4.5 Plan and Calculation Submittals

It is important to carefully track runoff flow rates, bypassing flow rates, flow spreads, and other parameters related to gutter flow and inlet capacity for all design storms. Details of grate inlets, net opening, and ratings curves are required to be turned into the Department of Planning and Development.

4.5 Flow in Storm Drains

Storm drainpipe systems, also known as storm sewers, are pipe conveyances used in the stormwater drainage system for transporting runoff from roadway and other inlets to outfalls at structural stormwater controls and receiving waters. Pipe drain systems are suitable mainly for medium to high-density residential and commercial/industrial development where the use of natural drainageways and/or vegetated open channels is not feasible.

There are several general guidelines to be observed when designing storm sewer systems. When followed, they will tend to alleviate or eliminate the common issues made in storm sewer design. These guidelines are as follows:

1. Select pipe size and slope so that the velocity of flow will increase progressively, or at least will not appreciably decrease at inlets, bends or other changes in geometry or configuration. A 15” pipe diameter is the minimum acceptable pipe diameter for maintenance purposes.
2. Do not discharge the contents of a larger pipe into a smaller one, even though the capacity of the smaller pipe may be greater due to steeper slope. This guideline does not apply to when the larger pipe is specifically designed to provide detention.
3. At changes in pipe sizes, match the soffits of the two pipes at the same level rather than matching the flow lines to reduce backwater effects in the system. Matching inverts rather than soffits may be appropriate based on-site constraints, conflicts with other utilities, etc.
4. Conduits are to be checked at the time of their design with reference to critical slope. If the slope of the line is greater than critical slope, the unit will likely be operating under entrance control instead of the originally assumed normal flow. Conduit slopes should be kept below critical slope if at all possible. This also removes the possibility of a hydraulic jump within the line.

4.5.1 Hydraulic Grade Line

The water surface elevation, or hydraulic grade line, shall be at least 1 foot below the inlet throat elevation for the design flow. Where required, adjustments shall be made in the system to reduce the elevation of the hydraulic grade line to meet this requirement. All head losses in a storm sewer system including minor losses are considered in computing the hydraulic grade line to determine the water surface elevations, under design conditions, in the various inlets, catch basins, manholes, junction boxes, etc. The starting elevation of the hydraulic grade line shall be set to the tailwater elevation of the receiving stream or waterbody matching the design storm. For example, if the storm pipe system is

designed for the 25-year storm, then the tailwater elevation shall be based on the 25-year storm elevation in the receiving channel.

4.5.2 Roughness Coefficients

Any storm drainpipe located in a right-of-way or drainage easement shall be reinforced concrete pipe (RCP) except for side drains which are allowed to be high density polyethylene (HDPE). Table 4.7 below should be used for Manning's n-values for conduits.

4.5.3 Manhole Location

Manholes shall be located at intervals not to exceed 500 feet. Manholes shall preferably be located at street intersections, conduit junctions, changes of grade, changes of horizontal alignment and all changes of pipe sizes. For manholes and junction boxes deeper than 3 feet, steps shall be added.

4.5.4 Minor Head Losses at Structures

The following total energy head losses at structures shall be determined for inlets, manholes, wye branches or bends and the design of closed conduits. Minimum head loss used at any structure shall be 0.10 foot, unless otherwise approved. The basic equation for most cases, where there is both upstream and downstream velocity, takes the form as seen below with the various conditions of the coefficient of K_j shown in Tables 4.8 and 4.9.

$$h_j = K_j \frac{\left|V_2{}^2 - V_1{}^2\right|}{2g} \qquad \textbf{Eq. 4.4}$$

Where: h_j = Junction or structure head loss and feet.

V_1 = Velocity in upstream pipe and feet per second.

V2 = Velocity in downstream pipe and feet per second.

K_j = Junction or structure coefficient of loss.

In the case where the initial velocity is negligible, the equation for head loss becomes:

$$h_j = K_j \left(\frac{V^2}{2g}\right) \qquad \textbf{Eq. 4.5}$$

Short radius bends may be used on 24 inch or larger pipes where flow must undergo a direction change at a junction or bend. Reductions in head loss at manholes may be realized in this way. A manhole shall always be located at the end of such short radius bends.

Table 4.7 Manning's n-values for conduits.

Conduit Material	Manning's n-value
Steel:	
Lockbar and welded	0.012
Riveted and spiral	0.016
Cast Iron:	
Coated	0.013
Uncoated	0.014
Wrought Iron:	
Black	0.014
Galvanized	0.016
HDPE:	
Smooth	0.012
Corrugated	0.024
Corrugated Metal:	
Storm Drain	0.024
Cement:	
Neat, surface	0.011
Mortar	0.013
Concrete:	
Culvert, straight and free of debris	0.011
Culvert with bends, connections, and some debris	0.013
Sewer with manholes, inlet, etc., straight	0.015
Unfinished, steel form	0.013
Unfinished, smooth wood form	0.014
Unfinished, rough wood form	0.017
Wood:	
Stave	0.012
Laminated, treated	0.017
Brickwork:	
Lined with cement mortar	0.015
Sanitary sewers coated with sewage slime with bends and connections	0.013
Paved invert, sewer, smooth bottom	0.019
Rubble masonry, cemented	0.025

Source: Chow, 1959

The values of the coefficient K_j for determining the loss of head due to obstructions in pipe are shown in Table 4.8 and the coefficients are used in the previous equation to calculate the head loss at the obstruction. The values of the coefficient K_j for determining the loss of head due to sudden enlargements and sudden contractions in pipes are shown in Table 4.9 and the coefficients are used in the previous equation to calculate the head loss at the change.

Table 4.8 Minor Loss Coefficients.

Type of Obstruction	Coefficient (K_j)
22.5-Degree Bend	0.20
45-Degree Bend	0.35
60-Degree Bend	0.43
90-Degree Bend	0.50
Straight through Manhole	0.05
Inlet on main line	0.50
Inlet on main line with a lateral branch	0.25

Table 4.9 Minor Loss Coefficients for Junctions.

Type of Junction	Coefficient (K_j)
Junction or manhole on main line with a 22.5-degree lateral branch	0.75
Junction or manhole on main Line with a 45-degree lateral branch	0.50
Junction or manhole on main line with 60-degree lateral branch	0.35
Junction or manhole on main line with 90-degree lateral branch	0.25
Inlet entrance	1.25
Conduit Projecting from Fill, Socket End (Groove End)	0.20
Projecting from Fill, Square Cut End	0.50
Socket End of Pipe (Groove-End)	0.20
Square-Edge	0.50
Rounded	0.20
Mitered to Conform to Fill Slope	0.70

Source: Brazoria County, Texas Drainage Manual

4.5.5 Minimum Grades

Storm drains should operate with velocities of flow sufficient to prevent excessive deposition of solid material; otherwise, objectionable clogging may result. The controlling velocity is near the bottom of conduits and considerably less than the mean velocity. Storm drains shall be designed to have a minimum velocity flowing full of 2.5 feet per second (fps). Table 4.10 indicates the grades for both concrete pipe (n = 0.013) and for HDPE pipe (n = 0.024) to produce a velocity of 2.5 fps, which is considered to be the lower limit of scouring velocity. Grades for closed storm sewers and open paved channels shall be designed so that the velocity shall not be less than 2.5 fps nor exceed 12 fps. All other structures such as junction boxes or inlets shall be in accordance with City standard drawings. The minimum slope for standard construction procedures shall be 0.40 percent when possible. Any variance must be approved by the Director of Planning and Development. Closed storm sewers extending to furthest downstream point of development shall consider velocities and discharge energy dissipators to prevent erosion and scouring along downstream properties.

Table 4.10 Minimum slope required to produce scouring velocity.

Pipe Size (Inches)	Concrete Pipe Slope ft/ft	Corrugated HDPE Pipe ft/ft
15	0.0023	0.0076
18	0.0018	0.0060
21	0.0015	0.0049
24	0.0013	0.0041
27	0.0011	0.0035
30	0.0009	0.0031
36	0.0007	0.0024
42	0.0006	0.0020
48	0.0005	0.0016
54	0.0004	0.0014
60	0.0004	0.0012
66	0.0004	0.0011
72	0.0003	0.0010
78	0.0003	0.0009
84	0.0003	0.0008

4.5.6 Utilities

In the design of a storm drainage system, the engineer is frequently confronted with the problem of grade conflict between the proposed storm drain and existing utilities, such as communications, water, gas, and sanitary sewer lines. When conflicts arise between a proposed drainage system and utility system, the owner of the utility system shall be contacted and made aware of the conflict. Any adjustments necessary to the drainage system or the utility can then be determined.

Due to the difficulty and expense to the public with regard to hand cleaning, clearing, and other ditch maintenance, the following ditch requirements are specified to expedite small equipment cleaning and access to drainage easements and ditches:

- Manholes are not allowed in drainage ditches.
- Access Easements shall be required every 500 feet.
- Utility Crossings (above the channel flowline) shall be limited to one per block.
- Utilities shall not be located beneath a concrete bottom except at crossings.

4.5.7 Easements

Drainage easements shall be provided in accordance with the following requirements:

- Drainage easements shall be a minimum of 15 feet.
- For pipe or culverts less than 36-inch in diameter or width, the pipe shall be centered within the easement. For pipes or culverts greater than 36-inch diameter or width, the easement shall provide a minimum of 10 feet from the outside edges of the pipe or culvert on each side.

Minimum widths given above are for installations with depths of cover of 10-feet or less (measured at the top of pipe). For each additional 5-feet of cover over 10-feet (rounded up), the minimum easement width shall be increased by 10-feet.

4.6 References

- Flood Control and Water Quality Manual, Springfield 2022.

5. Open Channel Design

5.1 General

Open channel systems are an integral part of stormwater drainage design. In addition to natural channels, dry and wet swales, drainage ditches, riprap channels, concrete lined channels, and grass channels, encompass open channels. This section provides an overview of open channel design criteria and methods.

5.1.1 Considerations for Use of Open Channels

Open channels for use in the major drainage system have significant impact to stormwater drainage design in the aspects of cost, capacity, and aesthetic purposes. Disadvantages of open channels include right-of-way needs and maintenance costs.

Open channels may be used in lieu of storm sewers to convey storm runoff where:

- Sufficient right-of-way is available.
- Sufficient cover for the storm sewers is not available.
- It is important to maintain compatibility with existing or proposed developments.
- Economy of construction can be shown without long-term public maintenance expenditures.

Intermittent alternating reaches of opened and closed systems should be avoided.

The ideal channel is a natural one carved by nature over a long period of time and protected through comprehensive stormwater management techniques. The benefits of a natural channel include:

- Lower velocities and increased stability in channel bottom and banks.
- Channel storage tends to decrease peak flows.
- Decrease in maintenance associated with stability.
- Increase in ecological habitat.
- Channel provides desirable green belt.

In general, the man-made channel that most nearly conforms to the character of the natural channel is the most efficient and the most desirable.

In many areas undergoing development, the runoff has been so minimal that natural channels do not exist. However, a small trickle path nearly always exists which provides an excellent basis for location and construction of channels. Utilization of these trickle paths reduces development costs and minimize drainage problems by following grade and working with gravity.

Channel stability is a well-recognized problem in urban settings because of the significant increase in low flows and peak storm discharges.

Sufficient right-of-way or permanent easements should be provided adjacent to open channels to allow entry of city maintenance vehicles.

5.1.2 Open Channel Types

The three main classifications of open channel types according to channel linings are vegetated, flexible, and rigid. Vegetated linings include natural, grass-lined, grass with mulch, sod and lapped sod, and wetland channels. Riprap and gabions are examples of flexible linings, while rigid linings are generally concrete or rigid block.

Vegetative Linings – Vegetation, where practical, is the preferred lining for man-made channels. It stabilizes the channel body and bed, reduces erosion on the channel surface, and provides habitat and water quality benefits.

Conditions under which vegetation may not be acceptable include but are not limited to:

- High velocities.
- Lack of maintenance required to prevent growth of taller or woody vegetation and invasive species.
- Lack of nutrients and inadequate topsoil.
- Severe lack of access for maintenance.

Proper seeding, mulching and soil preparation are required during construction to assure establishment of healthy vegetation. Also, erosion control matting or other geofabrics may be required to be placed along the base and / or side slopes of these channels to allow establishment of vegetation. Post construction care of vegetation is critical to successful establishment.

Flexible Linings – Rock riprap, including rubble, is the most common type of flexible lining for channels. It presents a rough surface that can dissipate energy. These linings are usually less expensive than rigid linings. However, they may require the use of a filter fabric depending on the erosive characteristics of the underlying soils, and the growth of grass and weeds may present maintenance problems. Silty sand or silty loam soils typically require the use of a filter fabric. The US Army Corps of Engineers provides detailed design approach for riprap in Engineer Manual No. 1110-2-1601, Hydraulic Design of Flood Control Channels.

Rigid Linings – Rigid linings are generally constructed of articulated block or concrete and used where high flow capacity is required. Higher velocities, however, create the potential for scour at channel lining transitions and may lead to channel head cutting.

5.2 Design Criteria

Open channels shall be designed to the following criteria:

- In all cases for open channels, the design engineer shall calculate the 100-year flow and show the 100-year flow boundary and water surface elevation in the Plans and Specifications.
- Channel or adjacent public drainage/floodplain easement, etc., shall be capable of containing the fully developed 100-year storm with a minimum one-foot freeboard. Public drainage easements should encompass the width of the flow channel, floodplain, etc., with an additional 15 feet on each side of the specified design. For example, if the channel, or floodplain width is 50 feet wide, the drainage easement width at the same point will be 80 feet.
- Trapezoidal or parabolic cross sections are preferred.
- Channel side slopes shall be designed to have a maximum slope of 3:1 unless otherwise justified and designed with proper slope stabilization practices. Roadside ditches should have a maximum side slope of 3:1.
- Channel design shall consider effects of channel lining.
- If a stream channel must be relocated, the cross-sectional shape, meander, pattern, roughness, sediment transport capacity, and slope should conform to the existing conditions to the extent practicable. Some means of energy dissipation may be necessary when existing conditions cannot be duplicated.
- Streambank stabilization should be provided as a result of any stream disturbance such as encroachment and should include both upstream and downstream banks as well as the local site.

5.2.1 Velocity Limitations

The final design of engineered open channels should be consistent with the velocity limitations for the selected channel lining. Maximum velocity values for earthen materials categories are presented in Table 5.1. Seeding and mulch should only be used when the design value does not exceed the allowable value for bare soil. Velocity limitations for vegetative linings are reported in Table 5.1. Erosion Control Matting may be used if designed and constructed in accordance with manufacturer's specifications subject to the limitations provided in this manual.

Table 5.1 Maximum velocities for comparing lining materials.

Material	Maximum Velocity (ft/s)
Sand	2.0
Silt	3.5
Firm Loam	3.5
Fine Gravel	5.0
Stiff Clay	5.0
Graded Loam or Silt to Cobbles	5.0
Coarse Gravel	6.0
Shales and Hard Pans	6.0
Erosion control matting	*

Source: AASHTO Model Drainage Manual, 1991.

* Based on manufacturer specifications and subject to approval by City Design Review Engineer.

Table 5.2 Maximum velocities for vegetative channel linings.

Vegetation Type	Slope Range (%)[1]	Maximum Velocity[2] (ft/s)
Bermuda grass	0-10	5
Bahia		4
Tall fescue grass mixtures[3]	0-10	4
Kentucky bluegrass	0-5	6
Buffalo grass	0-10	5
	>10	4
Grass mixture	0-5[1]	4
	5-10	3
Annuals[4]	0-5	3
Sod		4
Staked sod		5

1. Do not use on slopes steeper than 10% except for side-slope in combination channel.
2. Use velocities exceeding 5 ft/s only where good stands can be maintained.
3. Mixtures of Tall Fescue, Bahia, and/or Bermuda.
4. Annuals – use on mild slopes or as temporary protection until permanent covers are established.
5. Source: Manual for Erosion and Sediment Control in Georgia, 1996.

5.2.2 Channel Cross Sections

The channel shape may be almost any type suitable to the location and to the environmental conditions. The shape may be able to be designed to promote open space, recreational needs and to create additional benefits.

1. Bend Radius: 25 feet or 10 times the bottom width, whichever is larger, is the minimum bend radius required for open channels.
2. Freeboard: Freeboard to top of bank shall be based on velocities associated with the design storm and shall be a minimum of 1 foot for channel velocities up to 8 ft/s and 2 feet for velocities exceeding 8 ft/s at the design storm. For deep flows with high velocities, greater freeboard shall be required, calculated in accordance with the following formula:

$$\boldsymbol{Freeboard\ (ft)\ =\ 1.0\ +\ 0.025\ vD^{1/3}} \qquad \textbf{Eq. 5.1}$$

Where: v = velocity of flow (ft/s)

D = depth of flow (ft)

For freeboard of a channel on a sharp curve less than the minimum bend radius, additional freeboard, to account for superelevation of the water surface, shall be computed as:

$$\boldsymbol{H\ =\ v^2\ ((T\ +\ b)/2gR_c)} \qquad \textbf{Eq. 5.2}$$

Where: H = additional height on the outside edge of channel (ft)

v = velocity of flow (ft/s)

T = top width of water surface (ft)

b = bottom width of channel (ft)

g = acceleration of gravity (32.2 ft/s2)

Rc = mean radius of bend (ft)

3. Connections: Connections at the junction of two or more open channels shall be designed to minimize transition loss for both vertical and horizontal transitions. Pipe and box culverts or sewers entering an open channel shall not project into the normal channel section. Nor will they be permitted to discharge into an open channel at an angle that directs flow upstream.

5.2.3 Channel Drops

Sloped drops shall have roughened faces and shall be no steeper than 2:1. They shall be adequately protected from scour and shall not cause an upstream water surface drop that will result in high velocities upstream. Downcutting and lateral cutting just downstream from the drops is a common problem which must be protected against. FHWA's HEC-14 manual and programs like HY-8 can be used to calculate scour potential and design energy dissipators.

5.2.4 Baffle Chutes

Baffle chutes are used to dissipate the energy in the flow at a larger drop. They require no tailwater to be effective. They are partially useful where the water surface upstream is held at a higher elevation to provide head for filling a side storage pond during peak flows.

Baffle chutes may be used in channels where water is to be lowered from one level to another. The baffle piers prevent undue acceleration of the flow as it passes down the chute. The baffled apron shall be designed for the full discharge design flow and shall be protected from scouring at the lower end. A stilling basin shall be added where appropriate based on velocities. Refer to FHWA's HEC-14 manual for baffle design.

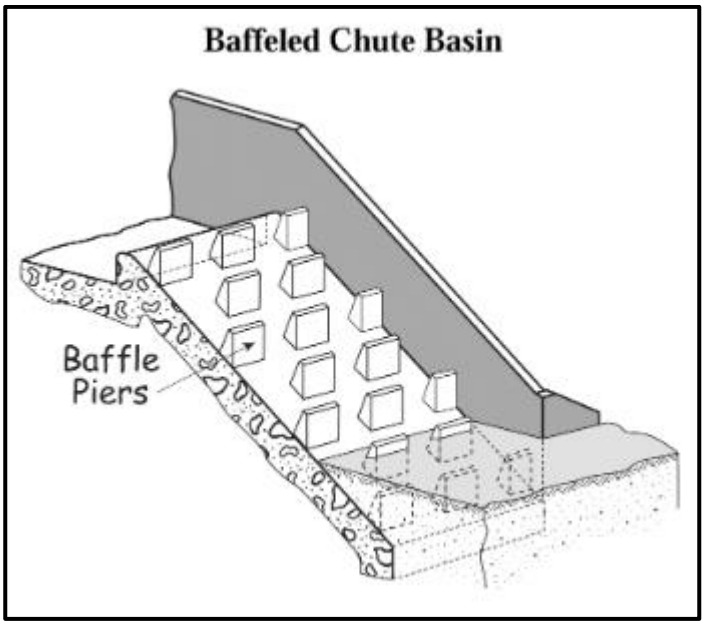


Figure 5.1. Example of Baffle Chute Basin

5.3 Computation and Software

Computer programs that utilize the Manning's equation shall be used for open channel design. Computer programs such as Hydraflow Express or the FHWA Hydraulic Toolbox may be used for Uniform Flow conditions; however, for more complex reaches or streams with higher flows, a backwater model such as HEC-RAS should be used. The general information to be provided in an open channel design is:

- Plan View - Existing and proposed topography.
- Profile - Left and right top of bank, 100-yr HGL, slope, invert flowlines.
- Cross-section - Dimensions, 100-yr hydraulic parameters.

Design flow and applicable design standards, design geometry required based on operational characteristics – freeboard, velocity, minimum standard capacity and site requirements, and flow regime – subcritical or supercritical – shall be reported and taken into consideration as part of design. Table 5.3 is a sample output file using Hydraflow Express computer software; FHWA Hydraulic Toolbox software provides a similar output report.

Table 5.3 Channel report output file.

Channel Section			
Channel Section Data:		**Highlighted:**	
Bottom Width (ft)	2.00	Depth (ft)	0.80
Side Slopes (z:1)	3.00, 3.00	Q (cfs)	13.00
Total Depth (ft)	2.00	Area (sq ft)	3.52
Invert Elevation (ft)	100.00	Velocity (ft/s)	3.69
Slope (%)	1.00	Wetted Perimeter (ft)	7.06
N-Value	0.025	Critical Depth, Yc (ft)	0.77
		Top Width (ft)	6.80
Calculations:		EGL (ft)	1.01
Compute by:	Known Q		
Known Q (cfs)	13.00		

5.3.1 Manning's n Values

Recommended Manning's n values for artificial channel linings are given in Table 5.4. The Values in Table 5.4 are based off the flow depth of the channel. For natural channels, earthen channels, and various types of vegetation, Manning's n values should be estimated using experienced judgment and based on the information in Table 5.5. Additional details are provided in the *Guide for Selecting Manning's Roughness Coefficients for Natural Channels and Flood Plains*, FHWA-TS 84-204, 1984.

Table 5.4 Manning's roughness coefficients (n) for artificial lined channels.

Category	Lining Type	Depth Ranges		
		0-0.5 ft	0.5-2.0 ft	>2.0 ft
Rigid	Concrete	0.015	0.013	0.013
	Grouted Riprap	0.040	0.030	0.028
	Stone Masonry	0.042	0.032	0.030
	Soil Cement	0.025	0.022	0.020
	Asphalt	0.018	0.016	0.016
Unlined	Bare Soil	0.023	0.020	0.020
	Rock Cut	0.045	0.035	0.025
Temporary*	Woven Paper Net	0.016	0.015	0.015
	Jute Net	0.028	0.022	0.019
	Fiberglass Roving	0.028	0.022	0.019
	Straw with Net	0.065	0.033	0.025
	Curled Wood Mat	0.066	0.035	0.028
	Synthetic Mat	0.036	0.025	0.021
Gravel Riprap	1-inch D50	0.044	0.033	0.03
	2-inch D50	0.066	0.041	0.034
Rock Riprap	6-inch D50	0.104	0.069	0.035
	12-inch D50	----	0.078	0.040

Note: Values listed are representative values for the respective depth ranges. Manning's roughness coefficients, n, vary with the flow depth.

*Some "temporary" linings become permanent when buried.
Source: HEC-15, 1988.

Table 5.5 Uniform flow values of roughness coefficient (n) for natural channels.

Type of Channel and Description	Minimum	Normal	Maximum
Natural Streams - Minor streams (top width at flood stage < 100 ft)			
1. Main Channels			
a. Clean, straight, full stage	0.025	0.030	0.033
b. Same as above, but some stones and weeds	0.030	0.035	0.040
c. Clean, winding, some pools and shoals	0.033	0.040	0.045
d. Clean, winding, but some weeds and some stones	0.035	0.045	0.050
e. Same as 4, lower stages, more ineffective slopes and sections	0.040	0.048	0.055
f. Same as 4, but more stones	0.045	0.050	0.060
g. Sluggish reaches, weedy, deep pools	0.050	0.070	0.080
h. Very weedy reaches, deep pools, or floodways with heavy stand of timber and underbrush	0.075	0.100	0.150
2. Mountain streams, no vegetation in channel, banks usually steep, trees and brush along banks submerged at high stages			
a. Bottom: gravels, cobbles, few boulders	0.030	0.040	0.050
b. Bottom: cobbles with large boulders	0.040	0.050	0.070
3. Floodplains			
a. Pasture, no brush			
1. Short grass	0.025	0.030	0.035
2. High grass	0.030	0.035	0.050
b. Cultivated area			
1. No crop	0.020	0.030	0.040
2. Mature row crops	0.025	0.035	0.045
3. Mature field crops	0.030	0.040	0.050
c. Brush			
1. Scattered brush, heavy weeds	0.035	0.050	0.070
2. Light brush and trees in winter	0.035	0.050	0.060
3. Light brush and trees, in summer	0.040	0.060	0.080
4. Medium to dense brush, in winter	0.045	0.070	0.110
5. Medium to dense brush, in summer	0.070	0.100	0.160
d. Trees			
1. Dense willows, summer, straight	0.110	0.150	0.200
2. Cleared land, tree stumps, no sprouts	0.030	0.040	0.050
3. Same as above, but with heavy growth of sprouts	0.050	0.060	0.080
4. Heavy stand of timber, a few down trees, little undergrowth, flood stage below branches	0.080	0.100	0.120
5. Same as above, but with flood stage reaching branches	0.100	0.120	0.160

Table 5.5 Uniform flow values of roughness coefficient n for natural channels.

Type of Channel and Description	Minimum	Normal	Maximum
5. EXCAVATED OR DREDGED			
a. Earth, straight and uniform			
1. Clean, recently completed	0.016	0.018	0.020
2. Clean, after weathering	0.018	0.022	0.025
3. Gravel, uniform section, clean	0.022	0.025	0.030
4. With short grass, few weeds	0.022	0.027	0.033
b. Earth, winding and sluggish			
1. No vegetation	0.023	0.025	0.030
2. Grass, some weeds	0.025	0.030	0.033
3. Dense weeds/plants in deep channels	0.030	0.035	0.040
4. Earth bottom and rubble sides	0.025	0.030	0.035
5. Stony bottom and weedy sides	0.025	0.035	0.045
6. Cobble bottom and clean sides	0.030	0.040	0.050
c. Dragline-excavated or dredged			
1. No vegetation	0.025	0.028	0.033
2. Light brush on banks	0.035	0.050	0.060
d. Rock cuts			
1. Smooth and uniform	0.025	0.035	0.040
2. Jagged and irregular	0.035	0.040	0.050
e. Channels not maintained, weeds and brush uncut			
1. Dense weeds, high as flow depth	0.050	0.080	0.120
2. Clean bottom, brush on sides	0.040	0.050	0.080
3. Same, highest stage of flow	0.045	0.070	0.110
4. Dense brush, high stage	0.080	0.100	0.140

5.3.2 Channel Discharge - Manning's Equation

Manning's equation, presented in three forms below, shall be used for evaluating uniform flow conditions in open channels:

$$V = (1.49/n)\, R^{2/3}\, S^{1/2} \qquad \text{Eq. 5.3}$$

$$Q = (1.49/n)\, AR^{2/3}\, S^{1/2} \qquad \text{Eq. 5.4}$$

$$S = \left[Qn \,/\, (1.49\, AR^{2/3})\right]^2 \qquad \text{Eq. 5.5}$$

Where: V = average channel velocity (ft/s)

Q = discharge rate for design conditions (cfs)

n = Manning's roughness coefficient

A = cross-sectional area (ft2)

R = hydraulic radius A/P (ft)

P = wetted perimeter (ft)

S = slope of the energy grade line (ft/ft)

If the channel is uniform in resistance and gravity forces are in exact balance, the water surface will be parallel to the bottom of the channel. This is the condition of uniform flow.

Open channel flow in urban drainage systems is complicated by bridge openings, curbs, and structures. Typically backwater computations will be required for channel design work; however, a check should also be performed for velocity based on headwater-controlled conditions.

A water surface profile shall be computed for all channels and shown on all final drawings. Computation of the water surface profile should utilize standard backwater methods or acceptable calculation procedures, taking into consideration all losses due to the changes in velocity, drops, bridge openings, and other obstructions.

Where practical, unlined channels should have sufficient gradient, depending upon the type of soil, to provide velocities that will be self-cleaning but will not cause erosion. Lined channels, drop structures, check dams, or concrete spillways may be required to control erosion that results from the high velocities of large volumes of water. Unless approved otherwise by the Director of Planning and Development or the Public Works Director, channel velocities in man-made channels shall not exceed those specified in Tables 5.1 and 5.2. Where velocities exceed specified velocities, riprap, pavement, or other approved erosion protection measures shall be required. As minimum protection to reduce erosion, all open channel slopes shall be seeded or sodded expeditiously after grading has been completed.

5.4 Channel Lining Design

5.4.1 Vegetative Design

For channels with vegetative and temporary lining, design stability shall be determined using Manning's n based upon poor vegetation conditions and for design capacity better conditions should be used. Channel velocities shall not exceed the maximum permissible velocities given in Tables 5.1 and 5.2. For more details on vegetative design refer to HEC-15.

5.4.2 Riprap Design

Where the use of riprap is allowed by the City Design Review Engineer, riprap sizing shall be determined based on maximum anticipated channel velocities. Adequate erosion protection shall be provided for the design configurations. For example, if riprap will extend into a stream with higher water surface elevations and/or velocities, i.e., at a pipe outfall going into a creek, then the riprap must be sized to resist the forces of the higher flow in the creek. When rock riprap is used, the need for an underlying filter material must be evaluated. The filter material may be either a granular blanket or plastic filter cloth.

Isbash Equation

The Isbash formula (Isbash 1936) was developed for the construction of dams by depositing rocks into moving water. The Isbash curve should only be used for quick estimates or for comparisons. A coefficient is provided to target high- and low-turbulence flow conditions, so this method can be a high- or low-energy application. The equation is:

$$V_c = C\left[2g\left(\frac{\gamma_S - \gamma_W}{\gamma_W}\right)\right]^{0.50}(D_{50})^{0.50} \qquad \textbf{Eq. 5.6}$$

Where: V_C = critical velocity (ft/s)

C = 0.86 for high turbulence

C = 1.20 for low turbulence

g = 32.2 ft/s^2

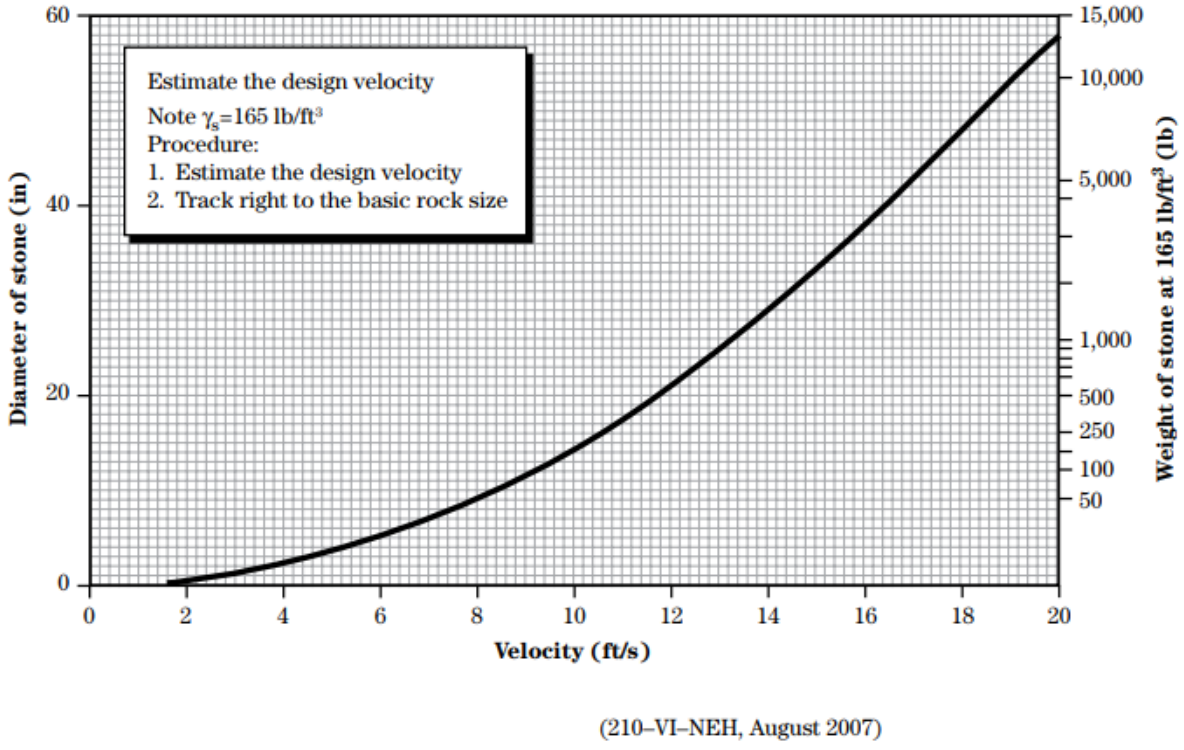
γ_S = stone density (lb/ft^3)

γ_W = water density (lb/ft^3)

D_{50} = median stone diameter (ft)

Figure 5.0 provides general riprap sizing criteria. For more detailed design, reference the US Army Corps of Engineers *Hydraulic Design of Flood Control Channels* manual. The design velocity should be based on the highest velocity of the design storm events, include velocities in receiving stream, if applicable. Extend a vertical line from the x-axis of the figure at the appropriate velocity until the curve is intersected, then extend a horizontal line to intersect the y-axis at the corresponding D_{50}, or median stone diameter for which no more than 50% of the stone by weight is smaller.

Figure TS14C–5 Rock size based on Isbash curve



(210–VI–NEH, August 2007)

Figure 5.3. Riprap sizing curve.

5.5 References

- Drainage Criteria Manual, Fayetteville 2014.
- National Engineering Handbook, Part 654, Natural Resource Conservation Service 2007.
- Open-channel hydraulic, Chow, V.T., 1959
- Outlet Erosion Control Structures (Stilling Basins), Association of State Dam Safety 2024.

6. Culvert Hydraulics

6.1 General

A culvert is defined as a short conduit used to convey stormwater runoff under an embankment such as a roadway or driveway whose primary purpose is to convey surface water. Alongside the hydraulic capabilities of culverts, a culvert must support the embankment and/or roadway while also protecting traffic and adjacent property owners from flood hazards to the extent practicable.

6.1.1 Criteria for Use of Culverts

Culvert design shall be based upon peak discharges for the appropriate design storm based on roadway type. Requirements are provided in Table 6.1. All computations, hydraulic profiles, and energy transition to channel shall be provided for the design event and the 100-year storm check.

Table 6.1 Culvert and Bridge Sizing Requirements

Roadway Classification	Design Storm Event	Minimum Freeboard (Culvert)	Minimum Freeboard (Bridges)
Principal Arterial Streets	100-year	1 foot	1 foot
Major and Minor Arterial Streets	100-year	1 foot	1 foot
All other streets	25-year	1 foot	1 foot

Note: Freeboard for culverts shall be from top of low point in road. Freeboard for Bridges shall be measured from the low chord.

Route the fully developed conditions 100-year frequency storm through all culverts to be sure building structures (i.e., houses, commercial buildings) are not flooded or increased damage does not occur to the roadway or adjacent property for the 100-year storm event. The runoff generated from the 100-year event shall be safely conveyed through drainage easements and/or the Right of Way. Appropriate tailwater conditions shall be used from receiving waters. See section 6.2.5, tailwater considerations.

6.2 Design Criteria

6.2.1 Velocity Requirements

The final design of culverts should consider the minimum and maximum velocities. A minimum velocity of 3 ft/s is required when a culvert is flowing partially full to ensure no siltation occurs. There is no maximum velocity constraint, however; if velocities exceed 10 ft/s, chances of abrasion due to bedload movement and erosion downstream increase significantly. When velocities exceed the permissible velocity for the receiving channel type, energy dissipators are necessary and should be included in the culvert design. Energy dissipators are discussed in Section 6.2.2.

6.2.2 Energy Dissipators

To prevent scour at stormwater outlets, protect the outlet structure, and minimize the potential of downstream erosion, energy dissipators are required to reduce the flow to a non-erosive velocity. Some common types of energy dissipators include:

- **Rock-Protected Outlets**

  Rock is often placed around the outlet of culverts to protect against the erosive action of the water. Typical placement of rock protection is shown in Figure 6.1. The material size used is dependent on the velocity and should be determined using a full flow analysis as noted in Table 6.2. Riprap is required to have a minimum depth of 12 inches.

- **Other Energy-Dissipating Structures**

  Other structures include baffled outlets, plunge pools, internal dissipators, impact basins, and stilling basins designed according to the FHWA's HEC-14, "Hydraulic Design of Energy Dissipators for Culverts and Channels."

Energy dissipators should be analyzed and designed using HY-8 Culvert Hydraulic Analysis Program or an approved equivalent.

Energy dissipators are known to collect debris so the possibility of debris collection should be considered when choosing a dissipator design. Dissipators should be kept open and easily accessible to maintenance crews and provisions should be made to allow water to overtop without causing excessive damage.

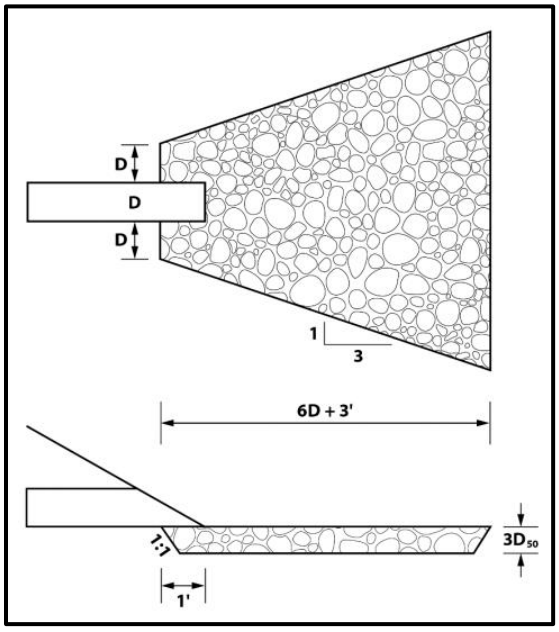


Figure 6.1. Typical Rock Protection Placement

Table 6.2 Outlet Protection Material Size

Outlet Velocity (ft/s)	Material
Up to 10	Dumped Riprap
>10	Foundation Protection Riprap

Note: All rock protection shall be designed in accordance with ARDOT Standard Specifications for Highway Construction Section 816.

6.2.3 Length and Slope

The maximum culvert slope using a reinforced concrete pipe shall be 10%. For culverts with a slope greater than 10%, the culvert must be approved by the Director of Planning and Development or Design Review Engineer to ensure proper pipe restraints. while the minimum slope for standard construction procedures shall be 0.5% when possible. Maximum drop in a drainage structure or junction box is 10 feet unless approved by the Director of Planning and Development or Design Review Engineer.

6.2.4 Headwater Limitations

Headwater is the water above the culvert invert at the entrance of the culvert. Headwater will be non-damaging to adjacent property and/or roadways. The maximum permissible headwater is determined based on hydraulic evaluation and the proposed or existing roadway elevation and is the primary basis for sizing a culvert.

The following headwater criteria apply to culvert design:

- The allowable headwater is the depth of water that can be ponded at the upstream end of the culvert during the design flood.
- Headwater shall have no adverse impact on upstream property.
- Maximum headwater depth for the design storm shall be 1 foot lower than the lowest top of road or curb elevation.
- Ponding depth shall be no greater than the elevation where flow diverts around the culvert.
- For drainage facilities with a cross-sectional area equal to or less than 30 sq.ft., headwater to depth ratio (HW/D) should be equal to or less than 1.5.
- For drainage facilities with a cross-sectional area greater than 30 sq.ft., HW/D should be equal to or less than 1.2.
- The headwater should be checked against the 100-year flood (base flood) elevation to ensure compliance with floodplain management criteria.
- The culvert should be sized to maintain flood-free conditions on principal and minor arterials with 1-foot freeboard from the low point of the road.
- Identify the maximum acceptable outlet velocity, based on receiving channel conditions. Reference Section 5.2.1 to determine acceptable velocities based on channel type.
- The constraint that gives the lowest allowable headwater elevation establishes the criteria for the hydraulic calculations.
- Bridges require 1 foot of freeboard from the low chord.

6.2.5 Tailwater Considerations

The hydraulic conditions downstream of the culvert site must be evaluated to determine tailwater depth for a range of discharge for the appropriate design storm and the 100-yr storm. At times, there may be a need for calculating backwater curves to establish the tailwater conditions. When evaluating the tailwater, the following must be considered:

- If the culvert outlet is operating with a freefall outfall, the critical depth and hydraulic grade line shall be determined.
- For culverts that discharge into an open channel, the water surface elevation in the open channel for the relevant design storm events. Tailwater conditions for all required flood events should be evaluated as a part of the culvert capacity and velocity computations. See Chapter 5, Open Channel Design.

- If an upstream culvert outlet is located near a downstream culvert inlet, the headwater elevation of the downstream culvert may establish the design tailwater depth for the upstream culvert.
- If the culvert discharges to a lake, pond, or other major water body, the expected high-water elevation for the design storm of the water body may establish the culvert tailwater.

6.2.6 Culvert End Treatments

The culvert inlet often has a significant impact on the culvert's hydraulic capacity, efficiency, and cost. The inlet coefficient, K_e, is a measure of the hydraulic efficiency of the inlet, with lower values indicating greater efficiency. Table 6.3 provides recommended inlet coefficients.

Culvert end treatments are required for all culverts installed in public right of ways or drainage easements. Some common end treatments include:

Headwalls
Headwalls shall be constructed with reinforced concrete. Straight, flared, and warped headwalls are all permissible depending on site conditions. Headwalls are required to be included in the culvert design when culverts cross the embankments at angle of 15-degrees or greater.

Wingwalls
Wingwalls are required when the side slopes of the channel adjacent to the entrance are unstable or where the culvert is skewed to the normal channel flow.

Aprons
If the approach velocity in the channel will cause scour, channel aprons at the toe are required to be included in the culvert design. Aprons shall extend a minimum of one pipe diameter upstream from the culvert entrance. The top of apron elevation shall not protrude above the normal streambed elevation.

Flared End Sections
Flared End Sections are a common end treatment for arch and round pipes suitable for many site applications. In some cases, additional erosion control or energy dissipators may be required depending on velocities.

6.2.7 Size and Material Selection

Reinforced concrete pipe (RCP) is to be used in roadway areas including under curbs. Sections where other material is used must be approved by the City Design Review Engineer. Polyvinyl chloride pipe (PVC) and high-density polyethylene pipe (HDPE) may be used in non-roadway areas. Galvanized CMP is not permissible and shall not be used in any culvert design. Coated CMP and HDPE flared end sections are prohibited within right of ways and drainage easements. All pipes shall be installed according to the standard details provided by the City of Little Rock including bedding, backfill, and compaction. Details for bedding, backfill, and compaction must be included in the Plans and Specifications.

The minimum allowable circular pipe diameter shall be 18 inches for culverts.

6.3 Design Procedure

6.3.1 Flow Type

Inlet and outlet control are the two basic types of flow control defined by the FHWA. The characterization of pressure, subcritical, and supercritical flow regimes play an important role in determining the control type. The control type also plays a significant role in determining the hydraulic capacity of a culvert. Proper culvert design requires checking for both inlet and outlet control to determine which will govern culvert designs.

Inlet Control

Inlet control occurs when the culvert barrel can convey more flow than the inlet will accept. In this control, critical depth occurs just inside the entrance of the culvert and the flow regime immediately downstream is supercritical. The upstream water surface elevation and the inlet geometry represent the major flow controls. Figure 6.2 depicts a typical inlet control flow section.

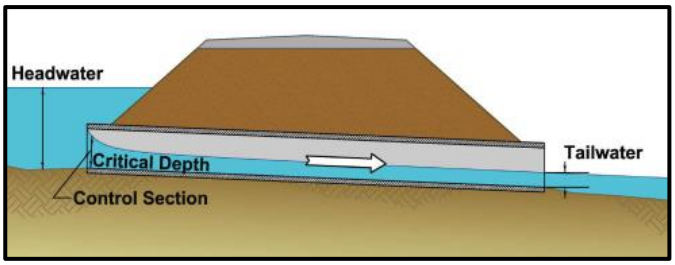


Figure 6.2. Typical Inlet Control Flow Section

Outlet Control

Outlet control flow occurs when the culvert barrel is not capable of conveying as much flow as the inlet opening will accept. The control section for outlet control flow is located at the barrel exit or further downstream. Either subcritical or pressure flow exists in the culvert barrel under these conditions. All geometric and hydraulic control characteristics, including all factors governing inlet control, water surface elevation at the outlet, and barrel characteristics, play a role in determining the culvert's capacity. Figure 6.3 depicts two typical outlet control flow sections.

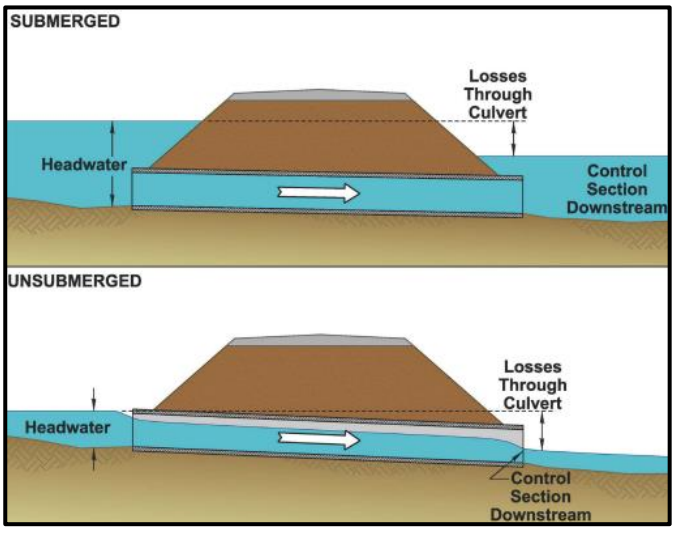


Figure 6.3. Typical Outlet Control Flow Sections

6.4 Design Software

It is recommended to use HY-8 Culvert Hydraulic Analysis Program developed by the Federal Highway Administration for all culvert design and analysis. Additional software may be accepted for use by the City Design Review Engineer provided it is shown to be equivalent to HY-8. For culvert crossings along creeks, include the culvert crossing in the HEC-RAS analysis.

6.4.1 Design Procedure

The following design procedure should be conducted using HY-8 or an approved equivalent.

Step 1. – List Design Input Data.

Q = discharge (cfs)	L = culvert length (ft)
S = culvert slope (ft/ft)	TW = tailwater depth (ft)
V = velocity for trial diameter (ft/s)	K_e = inlet loss coefficient
Material Type	HW = allowable headwater depth (ft)

Step 2. – Determine Trial Size.

Assume a trial velocity of 3-5 ft/s and compute the culvert area using A = Q/V. Determine the culvert shape, open size (diameter or span and rise), and number of barrels.

Step 3. – Calculate HW for Inlet and Outlet Control

For inlet control, enter inlet control data into the software with D and Q and find HW/D for the entrance type. If HW is too large, adjust the opening size and recompute until the HW is acceptable.

For outlet control, enter the outlet control data into the software with the culvert length, entrance loss coefficient, and trial culvert diameter. Use Equation 6.1 to compute the HW elevation.

$$HW = H + h_0 - LS \qquad \textbf{Eq. 6.1}$$

Where: h_0 = ½ (critical depth + D) or tailwater depth, whichever is greater.

Step 4. – Determine if the culvert is under Inlet or outlet control.

Compare the computed headwaters and use the higher HW to determine if the culvert is under inlet or outlet control.

If inlet control governs, then the design is complete, and no further analysis is required.

If outlet control governs and the HW is unacceptable, select a larger trial size and repeat the steps. Unless material or entrance conditions change, the inlet control conditions for the larger pipe does not need to be rechecked.

Step 5. – Check Potential Scour.

Calculate the exit velocity and if erosion problems are expected. Modify the culvert size to eliminate the erosion problems. If erosion problems cannot be eliminated, refer to section 6.2.2 for appropriate energy dissipation design.

6.4.2 Multi-barrel Installations

For culverts installations with multiple barrels exceeding a 3:1 width to depth ratio, the side bevels become excessively large when proportioned on the basis of the total clear width. For these structures, it is recommended that the side bevel be sized in proportion to the total clear width or three times the height, whichever is smaller.

The top bevel dimension should always be based on the culvert height.

The shape of the upstream edge of the intermediate walls of a multi-barrel culvert is not as important to the hydraulic performance of a culvert as the edge condition of the top and sides. Therefore, the edges of these walls may be square, rounded with a radius of one-half their thickness, chamfered, or beveled. The intermediate walls may also project from the face and slope downward to the channel bottom to help direct debris through the culvert. Multi-barrel pipe culverts should be designed as a series of single barrel installations since each pipe requires a separate bevel.

For multi-barrel culvert installations, one culvert in the middle of the channel should be set at the elevation of the existing flowline and the remaining culverts should be set at the elevation of the bank full channel. Figure 6.4 provides an example of a multi-barrel culvert installation.

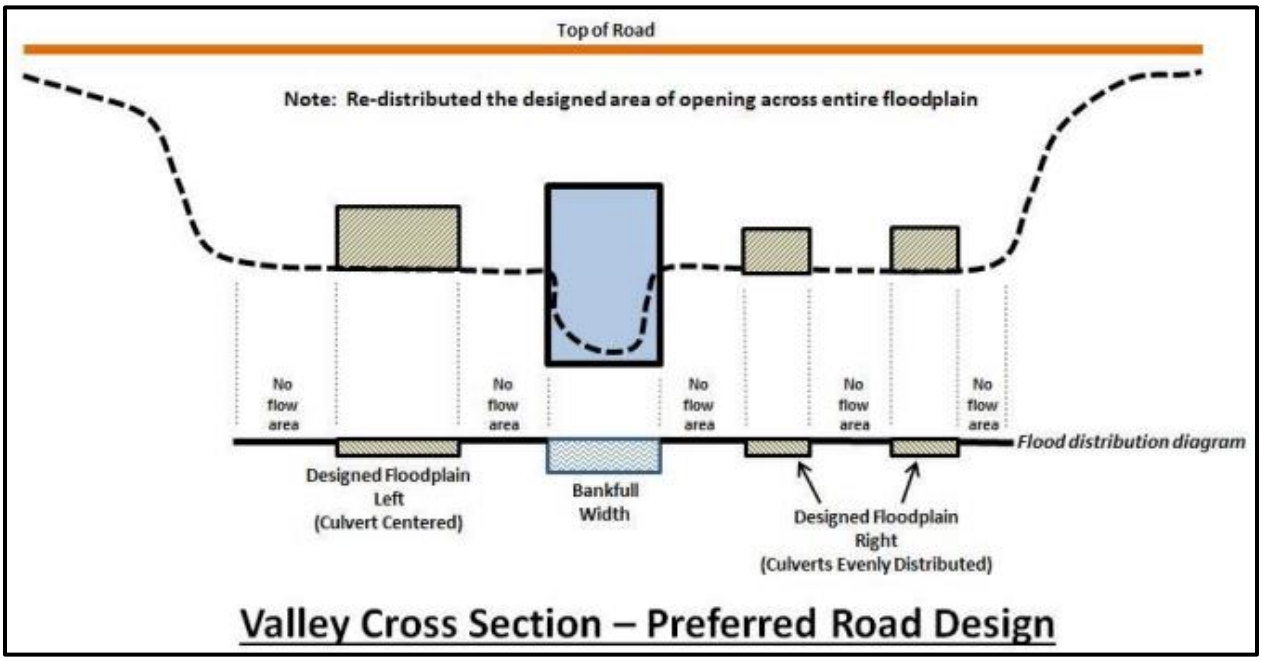


Figure 6.4. Road bisecting the floodplain with floodplain culverts.

Table 6.3 Inlet Coefficients

Type of Structure and Design of Entrance	Coefficient K_e[1]
Pipe, Concrete	
Projecting from fill, socket end (grove-end)	0.2
Projecting from fill, square cut end	0.5
Headwall or headwall and wingwalls	
Socket end of pipe (groove-end)	0.2
Square-edge	0.5
Rounded [radius = 1/12(D)]	0.2
Mitered to conform to fill slope	0.7
Flared End-Section conforming to fill slope	0.5
Beveled edges, 33.7o or 45 o bevels	0.2
Side- or slope-tapered inlet	0.2
Pipe, or Pipe-Arch, Corrugated Metal[1]	
Projecting from fill (no headwall)	0.9
Headwall or headwall and wingwalls square-edge	0.5
Mitered to fill slope, paved or unpaved slope	0.7
Flared End-Section conforming to fill slope	0.5
Beveled edges, 33.7 o or 45 o bevels	0.2
Side- or slope-tapered inlet	0.2
Box, Reinforced Concrete	
Headwall parallel to embankment (no wingwalls)	
Square-edged on 3 edges	0.5
Rounded on 3 edges to radius of [1/12(D)] or beveled edges on 3 sides	0.2
Wingwalls at 30° to 75° to barrel	
Square-edged at crown	0.4
Crown edge rounded to radius of [1/12(D)] or beveled top edge	0.2
Wingwalls at 10° or 25° to barrel	
Square-edged at crown	0.5
Wingwalls parallel (extension of sides) Square-edged at crown	0.7
Side- or slope-tapered inlet	0.2

1. The K_e values for corrugated metal pipes are also recommended for HDPE pipes.

Source: HDS No. 5, 1985.

6.5 References

- Hydraulic Design of Highway Culverts, 3rd Edition, USDOT 2012.
- WSDOT Hydraulics Manual, WSDOT 2024.

7. Stormwater Detention

7.1 General

This section provides guidance on stormwater runoff storage for meeting stormwater management control requirements (i.e., water quality treatment, downstream channel protection, overbank flood protection, and extreme flood protection).

Stormwater detention within a stormwater management system is essential to provide the required flow reduction for water quality treatment and downstream channel protection, as well as for peak flow attenuation of larger flows for overbank and extreme flood protection. Runoff storage can be provided within an on-site system through the use of structural stormwater controls and/or nonstructural features and landscaped areas.

7.2 Method of Evaluation

Pre-development and post-development runoff shall be calculated to evaluate the use of stormwater detention. Runoff shall be evaluated for the 2-, 25-, and 100-year 24-hour storm events. For areas less than 40 acres, the Modified Rational Method may be used to evaluate pre-development and post-development runoff conditions. Areas greater than 40 acres shall be evaluated using the SCS TR-55 Method.

If another method is used, the Owner's Engineer shall submit the proposed method of evaluation for the sizing of the retention basin or detention basin to the City Design Review Engineer. The method will be evaluated for professional acceptance, applicability, and reliability by the City Design Review Engineer. No detailed review will be rendered before the method of evaluation of the retention or detention basin is approved.

7.2.1. The Modified Rational Method

The Modified Rational Method uses the peak flow calculating capability of the Rational Method paired with assumptions about the inflow and outflow hydrographs to compute an approximation of storage volumes for simple detention calculations. This approach has many variations. Figure 7.0 illustrates one application. The rising and falling limbs of the inflow hydrograph have a duration equal to the time of concentration (t_c). An allowable peak outflow is set (Q_a) based on pre-development conditions. The storm duration is t_d and is varied until the storage volume (shaded gray area) is maximized.

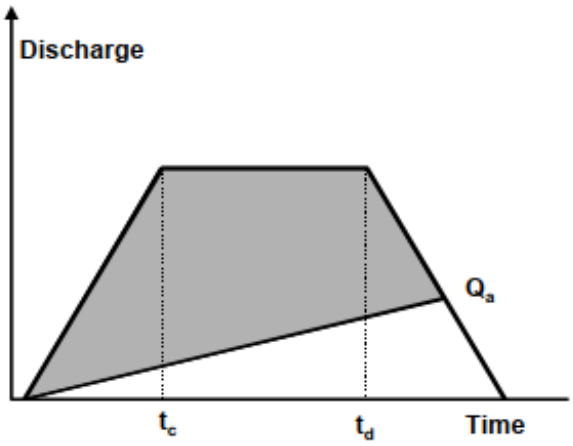


Figure 7.0. Modified Rational Definitions

The peak runoff rate can be determined from:

$$q_{pi} = C_a\, i\, A \qquad \text{Eq. 7.0}$$

Where: q_{pi} = peak runoff from site (cfs)

C_a = post-development Rational Method runoff coefficient

i = rainfall intensity for the corresponding time of concentration (in/hr)

A = area (acres)

It is assumed the peak of the outflow hydrograph falls on the recession limb of the inflow hydrograph and the rising limb of the outflow hydrograph can be approximated by a straight line.

The storage volume is determined by the critical (inflow) duration and using a constant outfall release rate. With these assumptions:

$$S_d = q_{pi} t_d - \frac{Q_a(t_d + t_c)}{2} \qquad \text{Eq. 7.1}$$

Where: S_d = detention volume required (ft^3)

Qa = allowable peak (pre-development) outflow rate (cfs)

td = design storm duration (sec)

tc = time of concentration for the watershed (sec)

The design storm duration is the duration that maximizes the detention storage volume, S_d, for a given return period. The storm duration can be found by trial and error using rainfall data from NOAA Atlas 14. This is normally an iterative process done by hand or with a spreadsheet. Software such as HydraFlow Hydrographs will compute storm durations and determine the estimated volume for a storm event. Downstream analysis is not possible with this method, as only approximate graphical routing takes place.

7.2.2. Flood Routing

The most commonly used method for calculating detention basin volume is to route an inflow hydrograph through a detention pond utilizing the Storage Indication or modified Puls method. This method compares the difference in the average values of two closely spaced inflows and outflows, yielding the change in storage over a given time period. By continuing this process for the duration of the storm and beyond, the total required storage for the basin can be determined. This is the methodology utilized by HEC-HMS, WinTR-55, and other hydrology software and can also be completed through the use of a spreadsheet. A detailed description of the manual process for routing a storm through a detention basin is presented in Chapter 8 of FHWA's HEC-22.

7.3 Detention Volume

The detention volume calculated using the Modified Rational method should not be used for final design. The final design should be verified by using an appropriate software to route the inflow hydrograph and determine if the proposed volume is adequate. Software such as HEC-HMS, Hydraflow Hydrographs, and others have capabilities to route hydrographs through detention basins.

The volume of the basin is determined by developing a hydrograph and routing the design storm through the basin. If the design storm can be routed through the basin without overtopping or exceeding the freeboard requirements, the basin volume is adequate. If the routing procedure indicates the storage elevation of the basin exceeds the freeboard requirements or overtops the basin, additional volume in the basin is required.

The final design of a detention facility requires three items:

- an inflow hydrograph
- a stage vs. storage curve
- a stage vs. discharge curve

1. To check the capacity of a basin with a known volume, use the methods described in the previous sections.
   a. Develop an inflow hydrograph for the storm in question.
   b. Develop the stage-storage and stage-discharge curves for the basin.
   c. Route the storm through the basin to determine the outflow hydrograph. Check the peak of the outflow hydrograph to ensure that it does not exceed the allowable value. Also, check the peak storage volume to ensure that it does not exceed the capacity of the basin.

2. Analyzing a known basin utilizing the methods developed in the previous sections is relatively straightforward. However, determining the required size of a proposed basin is an iterative process, and can be quite time consuming without a method to develop a preliminary volume estimate. TR-55 provides a method for determining quick estimates of detention basin volumes.

   a. Figure 7.1 relates two ratios: peak outflow to peak inflow (q_o/q_i) and storage volume to runoff volume (V_s/V_r). The value for q_i is determined by the peak of the inflow hydrograph. The value for q_o is normally dictated by the allowable release rate. The volume of runoff can be calculated by the SCS method or tabular hydrograph method. The relationships in Figure 7.1 were determined on the basis of single stage outflow devices. Some were controlled by pipe flow, others by weir flow. Verification runs were made using multiple stage outflow devices, and the variance was similar to that in the base data.

   b. The method can therefore be used for both single- and multiple-stage outflow devices. The only constraints are that:
      1) Each stage requires a design storm and a computation of the storage required for it.
      2) The discharge of the upper stage(s) includes the discharge of the lower stage(s).

   c. The brevity of the procedure allows the designer to examine many combinations of detention basins. When combined with the Tabular Hydrograph Method, the procedure's usefulness is increased. Its principal use is to develop preliminary indications of storage adequacy.

      This estimating technique becomes less accurate as the q_o/q_i ratio approaches the limits shown in Figure 7.1. The curves in Figure 7.1 depend on the relationship among available storage, outflow device, inflow volume, and shape of the inflow hydrograph. When the storage volume (V_s) required is small, the shape of the outflow hydrograph is sensitive to the rate of the inflow hydrograph. Conversely, when V_s is large, the inflow hydrograph shape has little effect on the outflow hydrograph. In such instances, the outflow hydrograph is controlled by the hydraulics of the outflow device and the procedure therefore yields consistent results. When the peak outflow discharge (q_o) approaches the peak inflow (q_i),

the parameters that affect the rate of rise of a hydrograph, such as rainfall volume, curve number, and time of concentration, become especially significant.

The procedure should not be used to perform final design if an error in storage of 25% cannot be tolerated. Figure 7.1 is biased to prevent under sizing of outflow devices, but it may significantly overestimate the required storage capacity. More detailed hydrograph development and routing will often pay for itself through reduced construction costs.

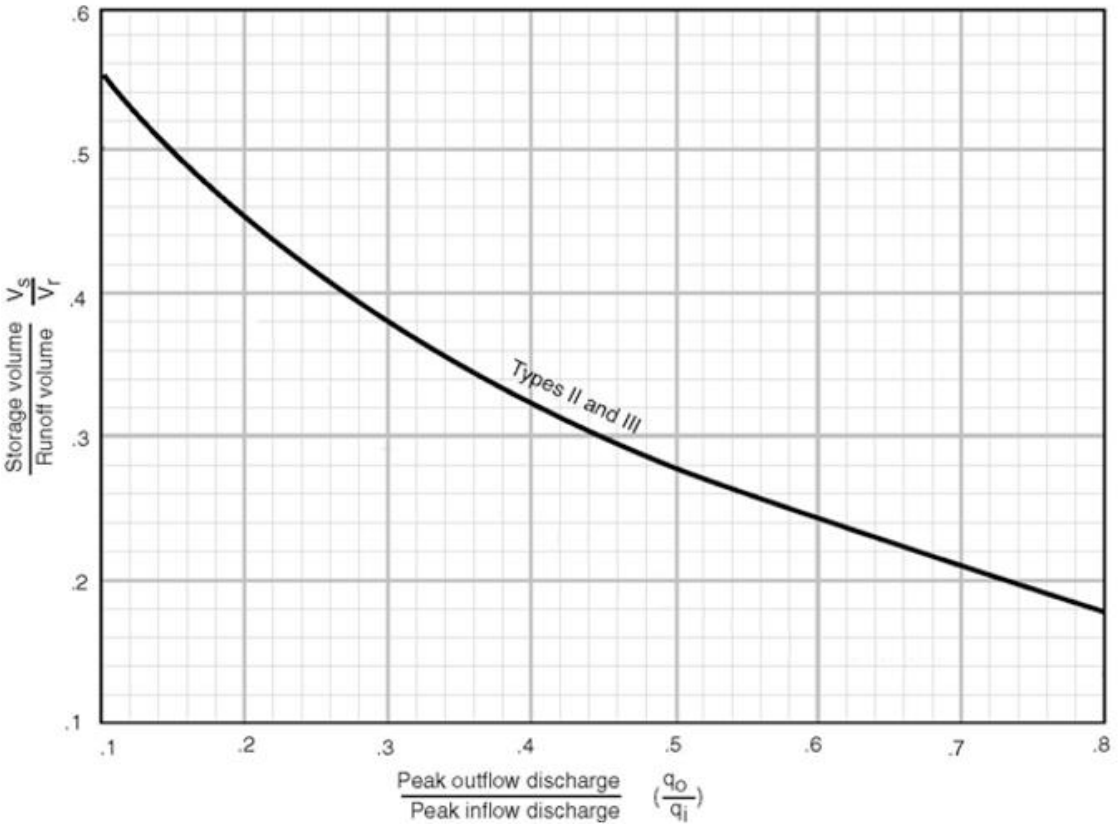


Figure 7.1. Approximate Detention Basin Routing for Type II and III

d. The purpose of Figure 7.1 is to provide a starting point for the size of the basin. The process may have to be repeated several times to achieve a basin that has sufficient volume and meets specific inlet and outlet controls.
e. Little Rock falls within the Type II rainfall category.

7.4 Methods of detention

Detention storage may be categorized as inline or offline. The City of Little Rock allows inline storage however offline storage is preferred. The engineer should submit a written plan that details the site constraints with regards to detention, the proposed site design, and the planned detention alignment for review and use of inline detention. Figure 7.2 illustrates inline versus offline storage.

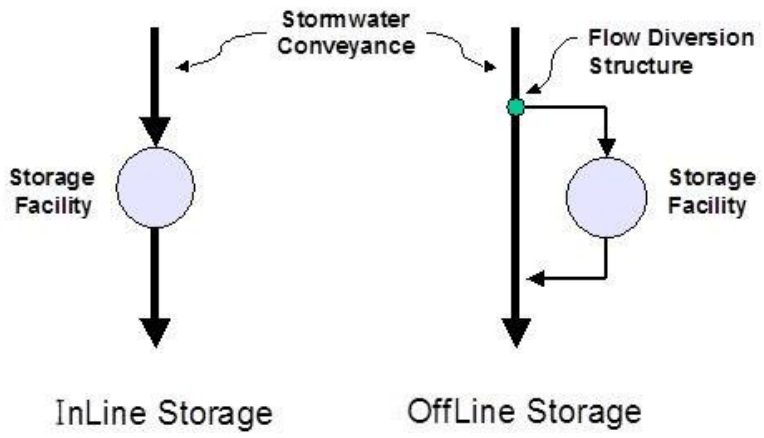


Figure 7.2. Depiction of inline versus offline storage

7.4.1. Structural Controls Appropriate for Detention

The following sections list the structural control practices appropriate for detention that are approved for use in the City of Little Rock. Mosquito control measures shall be taken for all proposed ponds. Avoid creating areas of shallow stagnant water and low dissolved oxygen which create mosquito habitat. To avoid creating a mosquito habitat, for wet detention, pools of water should be at least 5 feet deep and for dry detention hydraulic residence time should be less than 72 hours.

7.4.1.1. Stormwater Ponds

Stormwater ponds (also referred to as *retention ponds*, *wet ponds*, *or wet extended detention ponds*) are constructed stormwater retention basins that have a permanent (dead storage) pool of water throughout the year. They are categorized in this Manual as water quality structural controls and can meet the intent of the water quality criteria, however; they also can provide detention storage to meet the other stormwater criteria (Section 2.1).

In a stormwater pond, a certain design volume of runoff from each rain event is detained and treated in the pool through gravitational settling and biological uptake until it is displaced by runoff from the next storm. The permanent pool also serves to protect deposited sediments from re-suspension. Above the permanent pool level, additional temporary storage (live storage) is provided for runoff quantity control. Stormwater ponds are among the most cost-effective and widely used stormwater practices. A well-designed and landscaped pond can be an aesthetic feature on a development site when planned and located properly.

The most common of stormwater pond designs include the wet pond, the wet extended detention pond, and the micropool extended detention pond. In addition, multiple stormwater ponds can be placed in series or parallel to increase performance or meet site design constraints.

7.4.1.2. Stormwater Wetlands

Stormwater wetlands (also referred to as constructed wetlands) are constructed shallow marsh systems that are designed to both treat urban stormwater and control runoff volumes. As stormwater runoff flows through the wetland facility, pollutant removal is achieved through settling and uptake by marsh vegetation.

Stormwater wetlands are categorized as water quality structural controls to meet the water quality criteria, however; they also can provide detention storage to meet the other stormwater criteria (Section 2.1).

Wetlands are an effective stormwater practices in terms of water quality and offer aesthetic value and wildlife habitat. Stormwater wetlands require a continuous base flow or a high-water table to support aquatic vegetation. There are several design variations of the stormwater wetland, each design differing in the relative amounts of shallow and deep water, and dry storage above the wetland. These include the shallow wetland, the extended detention shallow wetland, pond/wetland system and pocket wetland.

7.4.1.3. Dry Detention / Dry ED Basins

Dry detention and dry extended detention (ED) basins are surface facilities intended to provide for the temporary storage of stormwater runoff to meet the downstream flood protection criteria. These facilities temporarily detain stormwater runoff, releasing the flow over a period of time. They are designed to completely drain following a storm event and are normally dry between rain events.

Both dry detention and dry ED basins provide limited pollutant removal benefits and are not intended for water quality treatment. Detention-only facilities should be used in a treatment train approach with other structural controls to provide water quality treatment.

7.4.1.4. Multi-purpose Detention Areas

Multi-purpose detention areas are site areas primarily used for one or more specific activities that are also designed to provide for the temporary storage of stormwater runoff to reduce downstream water quantity impacts. Examples of multi-purpose detention areas include:

- Sports Fields
- Recessed Plazas

Multi-purpose detention areas are normally dry between rain events, and by their nature must be usable for their primary function the majority of the time. As such, multi-purpose detention areas should be used for meeting the downstream flood protection criteria, but not for water quality criteria.

Multi-purpose detention areas should be used in a treatment train approach with other structural controls to provide water quality treatment.

7.4.1.5. Underground Detention

Underground detention facilities such as vaults, pipes, tanks, and other subsurface structures are designed to temporarily store stormwater runoff for water quantity control. As with above ground detention ponds, underground detention facilities are designed to drain completely between runoff events, thereby providing storage capacity for subsequent events. Underground detention facilities are intended to control peak flows, limit downstream flooding, and provide some channel protection. However, they provide little, if any, pollutant removal and are susceptible to re-suspension of sediment during subsequent storms.

Underground detention systems serve as an alternative to surface dry detention for stormwater quantity control, particularly for space-limited areas where there is not adequate land for a dry detention basin or multi-purpose detention area. Basic storage design and routing methods are the same as for detention basins except that the bypass for high flows is typically included.

Underground detention facilities may only be used where the hydraulic grade line (HGL) of the existing storm sewer network is low enough to allow adequate drainage to meet City design requirements within 72 hours after any design storm event. Underground detention facilities are not generally intended for water quality treatment and, unless it is specifically accommodated in design, should be used in a treatment train approach with other structural controls to provide water quality treatment. Providing treatment prior to discharging to the underground detention facility will help prevent the underground

system from becoming clogged with trash or sediment and significantly reduces the maintenance requirements for the system.

7.5 Detention Design Criteria

Stormwater detention systems shall be designed to meet the stormwater sizing criteria described in Section 2.1 and shall provide structural control as needed to meet the pre-development rate for the design storm events.

7.5.1 Design Procedure

A general procedure for the design of storage facilities is presented below.

Step 1 Perform preliminary calculations to evaluate detention storage requirements for the hydrographs as described above in Sections 7.21 and 7.3.

Step 2 Determine the physical dimensions necessary to hold the estimated volume from Step 1. The maximum storage requirement calculated from Sections 7.21 and 7.3 should be used. From the selected shape determine the maximum depth in the pond. Develop the stage-storage curve for the detention basin.

Step 3 Select the desired type of outlet and size the outlet structures based on allowable discharges for the design storm events, beginning with outlet structure sizing for the smaller events to the extreme flood event and taking into consideration the tailwater in the receiving stream. The estimated peak stage for each storm event (2-, 25-, and 100-year) will occur for the maximum associated volume from Step 2. The outlet structure(s) should be sized to convey the allowable discharge for the corresponding stage for each flood event. The outfall structure shall be designed with appropriate erosion prevention measures.

Step 4 Perform routing calculations using inflow hydrographs from Step 1 to check the preliminary design using a storage routing computer model.

Step 5 Evaluate whether the routed post-development peak discharges from the design storms exceed the existing pre-development peak discharges. If so, then revise the dimensions of the pond or outlet device geometry accordingly and repeat Steps 2 through 4 until the post-development peak discharges do not exceed the existing pre-development peak discharges for the watershed.

Step 6 Evaluate the downstream effects of detention outflows for the 100-year 24-hour storm event to ensure that the routed hydrograph does not cause downstream flooding problems. The outflow hydrograph from the storage facility should be routed through the downstream channel system to a confluence point that reflects no appreciable increase in discharges compared to the pre-development discharges at that location, or to a point designated by the City (see Section 7.5.3).

Step 7 Evaluate the control structure outlet velocity for all storms and provide channel and bank stabilization if the outlet velocities from any of the design storms will cause erosion problems downstream. Outlet protection shall include checking velocities and ensuring adequate erosion prevention measures to beyond the confluence with the receiving stream channel. Riprap placement or energy dissipator devices may be required. Guidance for riprap sizing and extents of placement and outlet design is provided in Section 6.2.

Routing of hydrographs through storage facilities is critical to the proper design of these facilities. Although storage design procedures using inflow/outflow analysis without routing have been developed, their use is not accepted by the City of Little Rock.

For this Manual, it is assumed that designers will be using one of the many computer programs available for storage routing and thus other procedures and example applications will not be given here.

7.5.2 Detention Design Standards

The following conditions and limitations shall be observed in selection and use of the method or type of detention.

7.5.2.1 General

Detention facilities shall be located within the parcel limits of the project under consideration. No detention or ponding will be permitted within public road right-of-ways. Location of detention facilities immediately upstream or downstream of the project will be considered by special request if proper documentation is submitted with reference to practicality, feasibility, and proof of ownership or right-of-use of the area proposed. To be in accordance with design requirements and not exceed pre-development discharges, orifices shall be provided to limit outflows.

7.5.2.2 Dry Detention / Dry ED Basins

Dry detention ponds or dry reservoirs shall be designed with proper safety, stability, and ease of maintenance facilities. Maximum side slopes for grass reservoirs shall not exceed 1-foot vertical for 3-feet horizontal (3:1) unless approved by the City Design Review Engineer. For dry detention ponds, pond bottom slopes must be a minimum of 1% (longitudinal and cross-slope) to ensure positive drainage to outlet works. In no case shall the limits of maximum ponding elevation be closer than 20 feet horizontally from any building and less than 1 foot vertically below the lowest adjacent grade. The entire reservoir area shall be stabilized with vegetation established prior to final approval or issuance of certificate of occupancy unless approved by the Director of Planning and Development. Any area susceptible to, or designed as, overflow by higher design intensity rainfall shall be stabilized with sod or other approved vegetative stabilization practice or paved depending upon the outflow velocity. Plan view and cross-sections with adequate details for any dry detention basins and forebays and dry ED basins shall be provided in the construction plans.

7.5.2.3 Stormwater Ponds

Stormwater ponds with fluctuating volume controls may be used as detention areas provided that the limits of maximum ponding elevations are no closer than 50-feet horizontal from any building, are at least 2 feet below the lowest sill or floor elevation of any building, and at least 1 foot below lowest adjacent grade.

Maximum side slopes for the fluctuating area of stormwater ponds shall be l-foot vertical to 3-feet horizontal (3:1) unless provisions are included for safety, stability, and ease of maintenance. Safety railing or other safety measures such as a shallow shelf shall be provided for ponds located in residential areas. All stormwater ponds shall include a sediment forebay at the inflow to the basin to allow heavier sediments to drop out of suspension before runoff enters the permanent pool. Sediment forebays shall be located at each point where piping or other conveyances discharge into the stormwater pond. Forebays shall be located such that they are accessible by maintenance equipment. Forebays shall be designed with adequate depth (preferably 4 to 6 feet to dissipate turbulent inflow - lesser design depths may be justified with supporting velocity computations) and volume to dissipate the energy of incoming stormwater flows and allow coarse-grained sediments and particulates to settle out of the runoff. The sediment forebay should be sized to accommodate 0.25 inches of runoff per contributing on-site impervious acre of drainage area and should allow flow to exit the forebay at non-erosive velocities from the 1-year to 10-year 24-hour storm events. The forebay may be included as part of the required volume for detention with permanent pools.

The entire fluctuating area of the permanent reservoir shall be stabilized with vegetation established prior to final approval or issuance of certificate of occupancy unless approved by the Director of Planning and Development. Also, calculations must be provided to ensure adequate "live storage" is provided for the difference between the post- and pre-developed 100-year, 24-hour storm. Any area susceptible to or designed as overflow by higher design intensity rainfall (100-year frequency) shall be sodded, stabilized with an approved vegetative stabilization practice, or paved, depending on the design velocities. An engineering analysis shall be furnished of any proposed earthen dam or embankment configuration, with appropriate geotechnical testing and computations. Earthen dam structures shall be designed by an Arkansas Licensed Professional Engineer. Plan view and cross-sections with adequate details for any stormwater ponds shall be provided in the Construction Plans. Detention basin embankment shall have a minimum 10-foot crown width.

7.5.2.4 Low Impact Development Practices

Low impact development (LID) practices can help reduce the peak flow of stormwater leaving the site. If LID practices are used on the project, they should be used upstream of any proposed detention facility. This will potentially result in reducing the quantity of stormwater necessary to be detained. Refer to Chapter 8, section 8.2 for detailed design requirements for LID practices and for the approach to adjust peak discharges, where appropriate, based on implementation of LID features.

7.5.2.5 Other Methods

If other methods of detention are proposed, proper documentation of hydrologic and hydraulic calculations, soil data, percolation, geological features, etc., will be needed for review and consideration.

7.5.2.6 Outlet Works

Detention facilities shall be provided with effective outlet works. Flows shall be limited to pre-development rates for the design storm events.

Safety considerations shall be an integral part of the design of all outlet works. Plan view and sections of the structure with adequate construction details shall be included in Plans.

Overflow openings (emergency spillway) are required for all ponds. The overflow opening shall be designed to accept the fully urbanized 100-year flood event assuming blockage of the closed conduit portion of the outlet works with 6 inches of freeboard. Spillway requirements must also meet all appropriate state and federal criteria. Design calculations shall be included for all spillways.

7.5.2.7 Discharge Systems

Existing upstream detention structures may be accounted for in design. Field investigations and hydrologic analysis shall be performed to substantiate benefits. A field survey of the existing physical characteristics of both the outlet structure and ponding volume shall be performed. A comprehensive hydrologic analysis shall be performed that simulates the attenuation of the contributing area ponds. This should not be limited to a linear additive analysis but rather should consist of a network of hydrographs that considers incremental timing of discharge and potential coincidence of outlet peaks.

7.5.2.8 Ownership of Stormwater Detention Ponds

Ownership of stormwater detention ponds that are not dedicated by the City of Little Rock shall be vested in the property owner.

The City will not process the Final Plat if all the drainage features are not complete. No alteration of the drainage system will be allowed without the approval of the Design Review Engineer.

7.5.2.9 Easements

Easements shall be provided on the plans for detention facilities. A minimum 20-feet wide drainage easement shall be provided along the reservoir area, providing vehicular access to the facility, and connecting the tributary pipes and the discharge system along the most passable route, when the discharge system is part of the public drainage system.

7.5.2.10 Maintenance

Detention facilities, when required, are to be built in conjunction with storm sewer installation and/or grading. Since these facilities are intended to control increased runoff, they must be partially or fully operational soon after the clearing of the vegetation. During project construction, silt and debris shall be removed as needed from the detention area and control structure(s) after each storm event to maintain the storage capacity of the facility.

Post-construction maintenance of detention facilities is divided into two components. The first is long-term maintenance that involves removal of sediment from the basin and outlet control structure. Maintenance to an outlet structure is minimal with proper initial design of permanent concrete or pipe structures. Studies indicate that in developing areas, basin cleaning by front-end loader or grader is estimated to be needed once every 5 to 10 years.

Annual maintenance is the second component and is the responsibility of the developer or association throughout the construction phases and of the pond owner in perpetuity after acceptance of the final plat or filing of the last subdivision phase that substantially adds stormwater to a detention basin.

These items include:

1. Minor dirt and mud removal,
2. Outlet cleaning,
3. Mowing,
4. Herbicide spraying (in strict conformance with the City's policies and procedures),
5. Litter control, and
6. Forebay cleaning (where applicable)

The responsibility for maintenance of the detention facilities and single-lot development projects shall remain with the general contractor until final inspection of the development is performed and approved, and a legal occupancy permit is issued. After legal occupancy of the project, the maintenance of detention facilities shall be vested with the owner of the detention pond.

7.5.3 Downstream Hydrologic Assessment

7.5.3.1 Introduction

The purpose of the overbank flood protection and extreme flood protection criteria is to protect downstream properties from increases in flood hazard due to upstream development. These criteria require the designer to control peak flow at the outlet of a site such that post-development peak discharge equals pre- development peak discharge. In certain cases, this does not always provide effective water quantity control downstream from the site and may exacerbate flooding problems downstream. The reasons for this have to do with the timing of the flow peaks, and the total increase in volume of runoff. This section outlines the procedure for determining the impacts of post-development stormwater peak flows and volumes on downstream flows. For sites less than 40 acres a downstream assessment is not required, however the site still must meet the pre-development discharge rates.

7.5.3.2 Reasons for Downstream Problems

Flow Timing

If water quantity control (detention) structures are indiscriminately placed in a watershed and changes to the flow timing are not considered, the structural control may increase the peak discharge downstream. The reason for this may be seen in Figure 7.3. The peak flow from the site is reduced appropriately, but the timing of the flow is such that the combined detained peak flow (the larger dashed triangle) is higher than if no detention were required. In this case, the shifting of flows to a later time brought about by the detention pond makes the downstream flooding worse than if the post- development flows were not detained.

Increased Volume

An important impact of new development is an increase in the total runoff volume of flow. Thus, even if the peak flow is effectively attenuated, the longer duration of higher flows due to the increased volume may combine with discharge from downstream tributaries to increase the downstream peak flows.
Figure 7.3 illustrates this concept. The figure shows the pre- and post-development hydrographs from a development site (Tributary 1). The post-development runoff hydrograph meets the flood protection criteria (i.e., the post-development peak flow is equal to the pre-development peak flow at the outlet from the site). However, the post-development combined flow at the first downstream tributary (Tributary 2) is higher than pre-development combined flow. This is because the increased volume and timing of runoff from the developed site increases the combined flow and flooding downstream. In this case, the detention volume needs to be increased to account for the downstream timing of the combined hydrographs to mitigate the impact of the increased runoff volume.

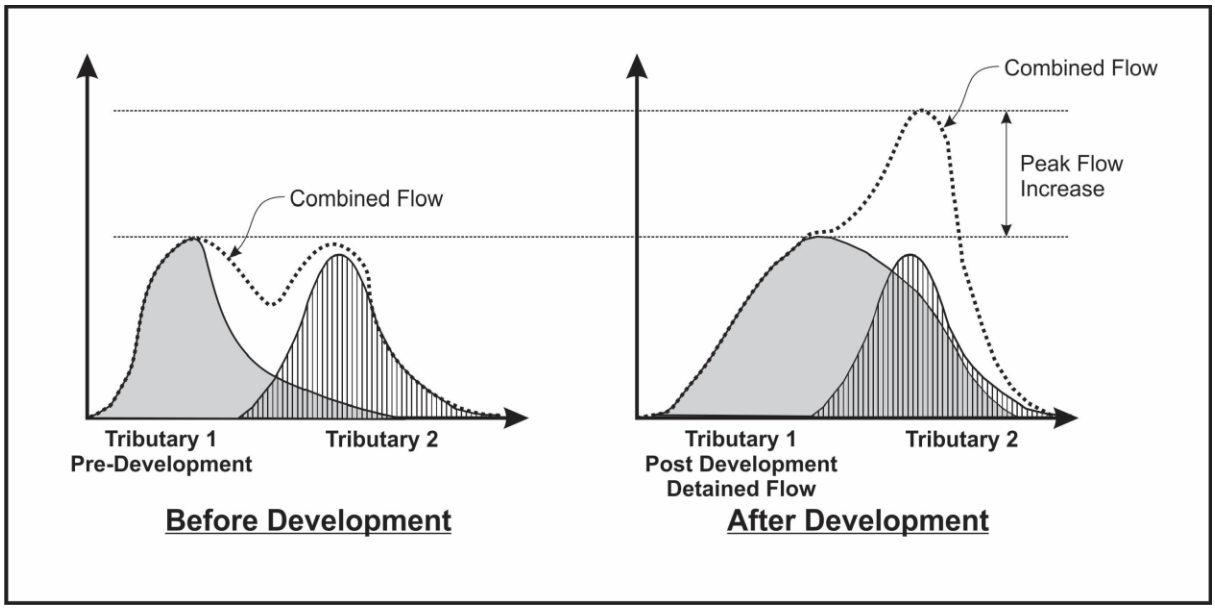


Figure 7.3. Downstream hydrograph comparison of pre and post development of detention structure.

7.5.3.3 The Ten-Percent Rule

In this Manual the “ten percent” criterion has been adopted as the most flexible and effective approach for ensuring that stormwater quantity detention ponds perform the desired function of maintaining pre-development peak flows throughout the system downstream.

The ten-percent rule recognizes the fact that a structural control providing detention has a “zone of influence” downstream where its effectiveness can be felt. Beyond this zone, the influence of the

structural control becomes relatively small and insignificant compared to the runoff from the total drainage area at that point. Based on studies and master planning results for a large number of sites, that zone of influence is considered to be the point where the drainage area controlled by the detention or storage facility comprises 10% of the total drainage area. For example, if the structural control drains 40 acres, the zone of influence ends at the point where the total drainage area is 400 acres or greater, see Figure 7.4. If the downstream assessment point extends past the confluence with a FEMA detailed study (Zone AE) stream, then the downstream assessment will end at the confluence. The City Design Review Engineer may assign additional locations for assessment based on locations of known downstream flooding, high erosion potential, downstream development, and channel constrictions.

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

1. Determine the target peak flow for the site for pre-development conditions.
2. Using a topographic map, assess the anticipated lower limit of the zone of influence (10% point).
3. Using a hydrologic model, to the same level of detail as for site project design, 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. The designer shall use hydrologic models obtained from the City of Little Rock or the data therefrom, if available, for the assessment of the downstream subareas.
4. Change the land use on the site to post-development and rerun the model.
5. Design the structural control facility such that the post development peak discharges are not increased above pre-development discharges at the outlet and the determined tributary junctions.

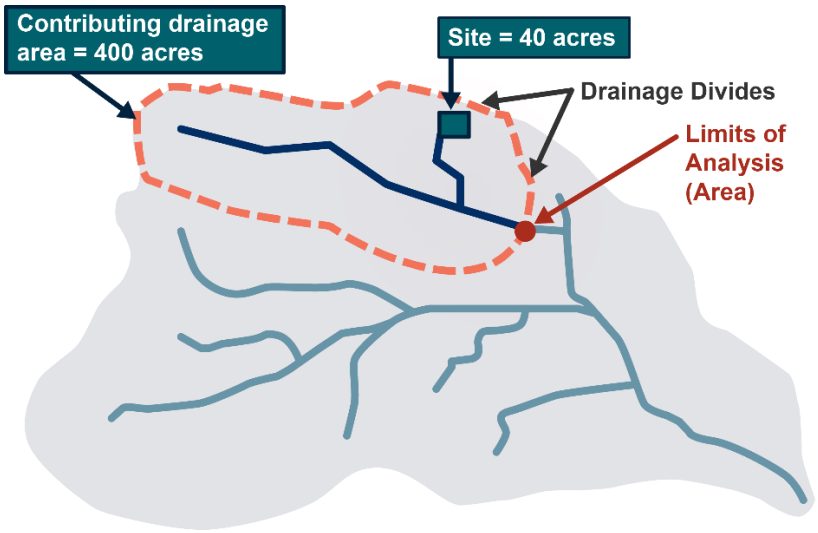


Figure 7.4. Schematic of zone of influence using the 10% rule.

7.6 References

- Drainage Criteria Manual, Fayetteville 2014.
- Georgia Stormwater Management Manual, Volume 1: Chapter 6, Floodplain Management, 2001.
- Statewide Urban Design and Specifications Design Manual, Iowa 2013.
- Urban Drainage Design Manual, USDOT 2013.

8. Water Quality

8.1 General

The City of Little Rock does not require any additional water quality requirements other than the water quality requirements of the current ADEE NPDES Municipal Separate Storm Sewer System (MS4) permit and ADEE NPDES General Stormwater Permit. The MS4 permit requires permanent controls that will reduce, to the Maximum Extent Practicable (MEP), the discharge of pollutants from areas of new development and significant redevelopment after construction is completed. Section 8.2 and 8.3 contain information related to the current permits and will be revised when a new MS4 permit is issued to the City or when the General Stormwater Permit is revised.

8.2 Natural Buffer Areas

ADEE requires that a natural buffer zone be established between the top of stream bank and the disturbed area. The Stormwater Pollution Prevention Plan (SWPPP) shall contain a description of how the site will maintain natural buffer zones. For construction projects where clearing and grading activities will occur, SWPPP shall provide at least twenty-five (25) feet of natural buffer zone from any named or unnamed streams, creeks, rivers, lakes or other water bodies. The plan shall also provide at least fifty (50) feet of natural buffer zone from established TMDL waterbodies, waterbodies listed on the 303(d) list, an ERW, ESW, NSW, or other uses at the discretion of the Director. If the site will be disturbed within the recommended buffer zone, then the buffer zone area shall be stabilized as soon as possible. Exceptions from this requirement for areas such as water crossings, limited water access, and restoration of the buffer are allowed if the permittee fully documents in the SWPPP the circumstances and reasons for the buffer zone encroachment. Additionally, this requirement is not intended to interfere with any other ordinance, rule or regulation, statute or other provision of law. Please note that above-grade clearing that does not disturb the soil in the buffer zone area does not have to comply with buffer zone requirements.

8.3 Post-Construction Stormwater Management

The engineer is required to provide a description of the stormwater control measures (SCMs) that will be installed during the construction process to control pollutants in stormwater discharges that will occur after construction operations have been completed. Structural measures shall be placed on upland soils to the degree attainable. The installation of these devices may be subject to Section 404 (Corps of Engineers) of the Clean Water Act. Such stormwater control measures and practices may include but are not limited to:

- infiltration of runoff onsite;

- flow attenuation by use of open vegetated swales and natural depressions;

- stormwater retention structures;

- stormwater detention structures (including wet ponds);

- sequential systems, which combine several practices.

A goal of at least eighty percent 80% removal of total suspended solids from these flows which exceed predevelopment levels should be used in designing and installing stormwater management controls (where practicable). Design guidance for stormwater controls are included in Appendix C. Where this goal is not met, the engineer shall provide justification for rejecting each practice listed above based on site conditions.

8.4 Stormwater Control Measure Maintenance

Each water quality BMP installed on a site requires regular maintenance to ensure that it functions properly. A BMP-specific maintenance agreement for each development site is required. The maintenance agreement consists of the following:

1. An Inspection and Maintenance Agreement signed by the developer or BMP owner.
2. A long-term maintenance plan written by the engineer or site designer that includes a description of the stormwater system and its components, inspection priorities and schedule for each component, and BMP schematics for each BMP. The plan should also include requirements for the proper disposal of any materials removed from the BMP during maintenance.
3. A drawing of easements on a plat or a system location map to enable the City to locate BMPs as needed.

The maintenance agreement and its attachments must be submitted for review by the City with the site plans. After the plans and the agreement are approved, the property owner shall record the maintenance agreement and its attachments with the register of deeds. The property owner, under the maintenance agreement, shall be responsible for inspecting and maintaining the BMPs and for turning in inspection reports annually to show that the facilities have been inspected and maintained.

8.5 References

- ADEE NPDES Municipal Separate Storm Sewer System (MS4) permit (ARS000002)
- ADEE NPDES General Stormwater Permit (ARR150000)

9. Construction Site Stormwater Management

9.1 General

Construction activities produce many kinds of pollutants which can cause water quality problems. In addition to erosion and sedimentation, construction activities often require the use of toxic or hazardous materials such as petroleum products and fuels, pesticides, herbicides, fertilizers, asphalt, concrete and sealants. These types of materials often contain small amounts of toxic substances which may harm human, plant and animal life along receiving streams and within lakes and ponds.

Management practices which control erosion and sedimentation fall into three main types; those which divert runoff from construction areas, those which prevent erosion on the construction site, and those which trap sediment before it can leave the construction site.

This section of the Stormwater Management and Drainage Manual provides information on many management practices and controls which can be used to comply with the conditions of a grading and land alteration permit. While specific practices are identified, careful consideration must be given to selecting the most appropriate management practices based upon site-specific conditions and installing controls in a timely and proper manner. New, novel, innovative, and proven BMPs for stabilization, erosion control, sediment control, and outlet protection measures not explicitly mentioned below are allowed and encouraged. New and novel measures require a submittal of a proposed alternative for staff's review and approval. It also must be noted that proper maintenance is required on all controls for them to remain effective.

9.1.1 Grading and Drainage Plans

City code and state regulations provide that any person proposing to engage in grading, clearing, filling, cutting, quarrying, construction or similar activities regulated by this article shall apply to the Director of Planning and Development for approval of plans and issuance of a grading and land alteration permit.

There are several exceptions to grading and land alteration permit requirements, and these exceptions are considered by the City Design Review Engineer during Grading and Drainage Plan review. There are no exceptions to the need to prepare and submit a Grading and Drainage Plan. Proposed development which does not meet the criteria for a Grading and Drainage Plan, as set forth in the following paragraphs, must include certification from the Architect and/or Engineer that the criteria are not applicable to the proposed development. Failure to provide certification will result in the plan being rejected by the City Design Review Engineer.

A Grading and Drainage Plan is required for **any** of the following activities:

A. Cut or fill activity greater than fifteen (15) vertical feet in height.
B. Cut or fill volume equal to or greater than one thousand (1,000) cubic yards.
C. Clearing that exceeds one (1) acre in size.
D. Clearing that is less than one (1) acre in size but is a part of a larger development.
E. Any land alteration on properties that are located within the 100-year floodplain boundary.

The Grading and Drainage Plan is submitted to and reviewed by the City Design Review Engineer to determine if a Grading Permit is required. Where vertical cut or fill activity greater than thirty (30) feet is indicated, Planning Commission approval is required. It should be noted that all construction work must include appropriate drainage and erosion control measures, regardless of whether a grading and land alteration permit is required.

The Universal Soil Loss Equation has been adopted by the City of Little Rock to enable planners and developers to predict the average rate of soil erosion from construction sites. The City has established an allowable rate of soil loss at five tons per acres per year (5 t/a/yr). Grading plan development requires the application of the Universal Soil Loss Equation to determine the potential soil erosion from a site, which establishes the need for erosion controls. Once erosion controls are identified, the Universal Soil Loss Equation can be used to estimate the effectiveness and adequacy of erosion controls.

The application of erosion and sedimentation controls falls within the sequence of design; stabilization practices; erosion controls; sedimentation controls; and other controls, as applicable. It is possible to control site runoff with stabilization practices alone, where stabilization can reduce the potential soil loss from the site to at or below five (5) tons per acre per year. Where stabilization cannot reduce the potential soil loss to at or below the allowable limit, then erosion controls are also required. Where a combination of stabilization practices and erosion controls are not effective in reducing potential soil loss, sedimentation controls are also required.

9.1.2 Sketch Grading and Drainage Plan Requirements

A Sketch Grading and Drainage Plan may be submitted for agricultural land uses or forestry activities on land owned by forest-related industry. A Sketch Plan is required as a part of the Planning Commission Application for any of the activities identified in Section 9.1.1 above, and for planned unit developments, conditional use permits, site plan reviews, subdivision approvals, or multiple building site approvals.

A Sketch Plan must identify all the following:

A. Acreage of the proposed project.
B. Land areas to be disturbed, (hatching or shading).
C. Stages of grading which identify the limits of sections to be disturbed and the approximate order of development.
D. Extent of cut and fill, shown by placing a dashed line at the top and toe of cut or fill slopes, and indicating on the plan the height and slope of the cut or fill.
E. Provisions for collecting and discharging surface water.
F. Erosion and sediment control measures, including structural and vegetative measures.

The Sketch Plan requires the seal and signature of a registered engineer, architect or landscape architect certifying that the Sketch Plan complies with Municipal Code. Plans for areas of less than five (5) acres where vertical cut or fill height does not exceed ten (10) feet, or where only tree clearing activities will take place may be prepared by a contractor or the property owner.

A Grading and Drainage Plan Checklist has been prepared and is included in Appendix A. Refer to Section 9.1.4 for information on other local, state, and federal permitting requirements.

9.1.3 Complete Grading and Drainage Plan Requirements

A Complete Grading and Drainage Plan includes the requirements for a Sketch Plan and the following additional information:

A. A vicinity drawing identifying property lines, existing or platted streets and public ways within or immediately adjacent to the site.
B. Location of all known existing sewers, water mains, culverts, underground utilities, and existing permanent buildings within and adjacent to the tract.
C. Citation of any existing legal rights-of-way or easements affecting the property.
D. Soil loss calculations as estimated by the Universal Soil Loss Equation (Section 9.6).

E. A plan of the site at a minimum scale of 1'' = 100', showing:
   1. Address and telephone number of the owner, developer, and permittee.
   2. Approximate location and width of proposed streets.
   3. Approximate location and dimension of all proposed or existing lots.
   4. Approximate location and dimension of all parcels of land which will be dedicated to open space, public use, or will remain undisturbed.
   5. Existing and proposed topography at a maximum contour interval of five (5) feet.
   6. An approximate timing schedule indicating the starting and completion dates of the development.
   7. A timing schedule for the sequence of grading and the application of erosion and sediment control measures.
   8. Acreage of the proposed project.
   9. Identification of unusual material or soils in land areas to be disturbed and engineering recommendations for correcting any problems.
   10. Identification of suitable fill materials, including the type and source of outside fill materials.
   11. Specification of measures to control runoff, erosion, and sedimentation during construction, noting the areas where controls are required, and the type of controls employed.
   12. Measures to protect neighboring built-up areas and city property during construction.
   13. Provisions to stabilize soils and slopes after construction is complete, including when and where stabilization measures will be employed.

The Complete Plan must include the seal and signature of a registered engineer. If all boundary street and drainage improvements are in place, the seal and signature of a registered architect or landscape architect is acceptable.

The stormwater pollution prevention plan (SWPPP) shall be included with the grading & land alteration permit submittal to verify compliance with state's MS4 permit agreement with the City. A Grading and Drainage Plan Checklist has been prepared and is included in Appendix A.

9.1.4 Other permit Requirements

Application to Develop in a Flood Hazard Area

Proposed development within Special Flood Hazard Areas of the City requires the developer or their agent to obtain and complete an Application and Permit Form to Develop in a Flood Hazard Area. Work within the 100-year floodplain requires the applicant to complete FEMA Form TOD-1: Certification/Application Forms for Letters of Map Amendment/Revision Based on Fill. Work within the regulatory floodway, including changes in base flood elevations, fill, channelization, and bridge/culvert replacement projects require the applicant to prepare and submit applicable portions of FEMA Form RSD-1: Revisions to National Flood Insurance Maps. Additional information is available from the City's Floodplain Administrator.

Section 10 of the Rivers and Harbors Act of 1899

This Act prohibits the obstruction or alteration of navigable waters of the Unites States without a permit. Section 10 Permits are issued by the United States Army Corps of Engineers Permits Branch, who should be contacted at (501) 324-5295 for additional information.

Clean Water Act Section 404 Permits

Section 301 of this Act prohibits the discharge of dredged or fill material into waters of the United States without a permit. Section 404 Permits are issued by the United States Army Corps of Engineers. Contact the United States Army Corps of Engineers Permits Branch at (501) 324-5295 for more information.

Arkansas Solid Waste Management Code of 1984

The Arkansas Department of Pollution Control and Ecology authorizes all legitimate fill operations. An Application for Request for Fill Area is required to be completed and approved for development consisting of fill for the purpose of surface leveling. Contact the Solid Waste Division of the Arkansas Department of Pollution Control and Ecology at (501) 570-2858 for more information.

Arkansas State Water and Air Pollution Control Act

The Arkansas Department of Pollution Control and Ecology authorizes the discharge of stormwater associated with industrial activity from construction sites - those areas or common plans of development or sale that will result in the disturbance of five or more acres total land area. The Department has issued General Permit ARR10A000 for construction activities. A Notice of Intent is required at least 48 hours prior to commencing land disturbance activities. Contact the NPDES Branch of the Arkansas Department of Pollution Control and Ecology at (501) 562-7444 for more information.

Burning Permit

Site clearing often generates timber debris which may be mulched or burned on site. Burning requires approval from the City Fire Marshall on a permit form furnished by the Fire Department. Contact the Fire Department at (501) 371-4796 for more information, and to obtain permit forms.

Burning of demolition and construction debris is regulated by the Arkansas Department of Pollution Control and Ecology Air Division. Contact the Air Division at (501) 570-2161 for more information.

9.2 Sediment Control Criteria and Requirements

Stabilization

A record of the dates when grading activities occur, when construction activities temporarily or permanently cease on a portion of the site except as provided within bulleted text below, and when stabilization measures are initiated shall be included in the erosion and sediment control plan. Stabilization measures shall be initiated as soon as practicable in portions of the site where construction activities have temporarily or permanently ceased, but in no case more than 14 days after the construction activity in that portion of the site has temporarily or permanently ceased.

- Where the initiation of stabilization measures by the 14th day after construction activity temporarily or permanently ceases is precluded by snow cover, stabilization measures shall be initiated as soon as practicable.
- Where construction activity will resume on a portion of the site within 21 days from when activities ceased, (e.g., the total time that construction activity is temporarily ceased is less than 21 days) then stabilization measures do not have to be initiated on that portion of the site by the 14th day after construction activity temporarily ceased.

Stabilization practices may include temporary seeding, permanent seeding, mulching, geotextiles, sod stabilization, vegetative buffer strips, protection of trees, and preservation of mature vegetation and other appropriate measures. See Section 15-52 of the Little Rock City Code for tree protection requirements.

9.3 Stabilization Practices

EPA NPDES Fact Sheets for the following Erosion and Sediment Control BMPs can be found on their website under the National Menu of Best Management Practices (BMPs) for Stormwater-Construction | US EPA.

Examples of construction site stabilization practices:

- **Chemical Stabilization**
- **Filter Strips**
- **Preservation of natural vegetation**
- **Seeding**
- **Sod stabilization**
- **Stream bank stabilization**
- **Subsurface drains**

Chemical Stabilization. Chemical stabilization practices involve spraying soil surfaces with various man-made materials to hold the soil in place temporarily. This control is an alternative where temporary seeding is not practical because of the season or climate. Chemical stabilization can provide immediate and effective erosion control anywhere on a construction site.

Existing and Natural vegetation. Every means shall be taken to conserve and protect existing vegetation. The potential for soil loss shall be minimized by retaining natural vegetation wherever possible. Development in the Hillside/Hilltop Overlay District should comply with the recommendations of the Hillside/Hilltop Best Management Practices Manual about the retention of natural vegetation on Hillside/Hilltops.

Establishing New Vegetation. Vegetation practices may be either temporary or permanent and, at a minimum, should comply with Little Rock City Code Section 29-197, Advanced Grading Plan Requirements. They may be applied singularly or in combination with other practices. Cutting, filling, and grading soils with heavy equipment results in areas of exposed subsoils or mixtures of soil horizons. Conditions such as acidity, low fertility, compaction, and dryness or wetness often prevail and are unfavorable to plant growth and should be accounted for in the selection of plantings is required as specified for each BMP.

Long slopes and steep grades shall not be created. Stormwater drainage structures where such conditions already exist are normally subjected to hydraulic forces requiring both special establishment techniques and grasses that have high resistance to scouring. Vegetation practices and structural techniques are available to provide both temporary and permanent protective cover on these difficult sites, where encountered.

Filter Strips. These are areas of the site left undisturbed by clearing and construction activities. They are like buffer zones, slowing runoff velocity to allow sediment to settle out. However, filter strips serve the additional purpose of allowing runoff to infiltrate into the ground. Filter strips are an effective control applicable to sites where adequate space exists to leave undisturbed areas. Filter strips should be aligned perpendicular to the line of flow and can be used along with diversion ditches or berms to direct flow onto the vegetated surface area.

Temporary Vegetation. Earth moving activities such as heavy cutting, filling, and grading are generally performed in several stages and are often interrupted by lengthy periods, during which the land lies idle and is subject to accelerated erosion especially during rainfall events. In addition, final land grading may be completed during a season not favorable for immediate establishment of permanent vegetation. In such conditions, rapid growing annual grasses shall be used to rapidly

establish protective cover. This can later be worked into the soil for use as mulch when the site is prepared for establishment of permanent vegetation.

Permanent Vegetation. Final selection should be based on adaptation of the plants to the soils and climate, suitability for their specific use, ease of establishment, longevity, or ability to reseed, maintenance requirements, aesthetics, and other special qualities. Maintenance must be the most important consideration in selecting plants for permanent stabilization.

Plants that provide long-lived stabilization with the minimum amount of required maintenance should be selected. Where management potential is limited because of specialized circumstances, the best plants to choose are those that are well adapted to the site and to the specific purpose for which they are to be used. For example, grasses used for waterway stabilization must be able to withstand submergence and provide a dense cover to prevent scouring of the channel boundary.

In playgrounds, grasses must lend themselves to close grooming and be able to withstand heavy trampling. In some places, such as southern-exposed cut-and-fill slopes, the plants must be adapted to full sunlight and drought conditions. In other places, plants must be able to tolerate shade or high moisture conditions. Some plants can be used for beautification as well as for soil stabilization.

9.4 Erosion Control Methods

Control of erosion during construction requires an examination of the entire site to identify potential problem areas such as steep slopes, highly erodible soils, soil areas that could be unprotected for long periods or during peak rainy seasons, and natural drainageways. Assure erosion control in these critical areas. After a rain, the effectiveness of erosion control measures must be re-evaluated. Maintenance and cleaning of these facilities is also important.

Examples of Erosion Control Methods:

- **Buffer zone**
- **Compost Blankets**
- **Drainage Swale**
- **Diversion Dike**
- **Geotextiles**
- **Gradient Terraces**
- **Interceptor Dikes and Swales**
- **Mulching**
- **Outlet Protection**
- **Riprap**
- **Seeding**
- **Sodding**
- **Soil Retention**
- **Surface Roughening**
- **Temporary Slope Drain**

Additional items with respect to City erosion control requirements are provided herein:

Buffer zones. These are vegetated strips of land which control erosion by reducing the speed of runoff. Buffer zones can be areas left undisturbed during clearing and construction (filter strips), or they can be newly planted in areas that were previously disturbed by clearing and site activity.

Diversion Dike. This is a compacted earthen ridge constructed immediately above a cut or fill slope. Its purpose is to intercept storm runoff from upstream soil drainage areas and divert the water away from the exposed stabilized **outlet.**

Interceptor Dike. This is a temporary ridge of compacted soil or, preferably, gravel constructed across disturbed rights-of-way. An interceptor dike reduces erosion by intercepting stormwater and diverting it to stabilized outlets.

Perimeter Dike. This is a compacted earthen dike constructed along the perimeter of a disturbed area to divert sediment-laden stormwater to onsite trapping facilities. It is maintained until the disturbed area is permanently stabilized.

Flexible Down Drain. This is a temporary structure used to convey stormwater from one elevation to another without causing erosion. It is made of heavy-duty fabric or other material that can be removed when the permanent water disposal system is installed.

Mulching. When final grading has not been completed, straw, wood chips, jute matting, or similar materials can be applied to provide temporary protection. Areas brought to final grade during midsummer or winter can be mulched immediately and overseeded at the proper season with several permanent grasses or legume species. Application of mulch to disturbed areas allows for more infiltration of water into the soil, reduces runoff, holds seed, fertilizer, and lime in place, retains soil moisture; helps maintain temperatures conducive to germination, and greatly retards erosion. Mulch is essential in establishing good stands of grasses and legumes in disturbed areas. It is important to stabilize or anchor mulch using such practices as an anchoring tool, biodegradable tackifier (hydromulch), netting, peg and twine, or slitting to prevent it from blowing or washing off the site. Use of mulch in combination with Green Stormwater Practices shall comply with the requirements established in Chapter 8 and Appendix C.

Riprap. This is a layer of loose rock placed over the soil surface to prevent erosion by surface flow or wave action. Riprap may be used, as appropriate, at storm drain outlets, channel bank and bottom protection, roadside ditch protection, drop structures, etc.

Storm Drain Outlet Protection. This practice involves putting paving or riprap on channel sections immediately below storm drain outlets. A storm drain outlet is designed to reduce the velocity of flow and prevent downstream channel erosion. It is also known as an energy dissipator.

Temporary Storage, Shop and Staging Areas. Locate storage and shop yards where erosion and sediment hazards are slight. If this is not feasible, apply necessary paving and erosion control practices.

9.5 Siltation and Sediment Control

Examples of Siltation and Sediment Control Methods:

- **Compost Filter Berms**
- **Construction Entrance/Exits**
- **Compost Filter Socks**
- **Dust Control**
- **Fiber Rolls**
- **Filter Berms**
- **Sediment Basins and Rock Dams**
- **Sediment Filters and Chambers**
- **Sediment Traps**
- **Silt Fences**
- **Storm Drain Inlet Protection**
- **Temporary Storm Drain Diversion**
- **Temporary Stream Crossing**
- **Vegetated Buffers**

Additional items with respect to City sediment control requirements are provided herein:

Control and prevention of soil erosion during and after construction is the most important element of siltation and sediment control. However, it is physically and economically impractical to eliminate all soil erosion. Therefore, provisions must be made to trap eroded material at specified points. Some measures to implement are as follows:

- As inlet protection and on long slopes or runs, silt fence or rock check dams shall be used to create temporary ponds that store runoff and allow suspended solids to settle. These temporary ponds may be retained as part of the permanent storage system after construction; however, they must be inspected / surveyed to ensure that the design capacity of the system was not compromised by siltation.
- Inlet protection shall be maintained throughout construction and shall not be removed until vegetation is established. Such measures shall be periodically inspected in accordance with the requirement of the SWPPP and repaired / replaced when no longer functioning in accordance with design. Silted-in areas shall be mucked out after significant rainfall to restore capacity.
- Egress points from construction sites should be controlled so that the sediment is not carried offsite by construction traffic. A temporary construction entrance shall be constructed at points where traffic will be entering from or leaving a construction site to public right-of-way, street, alley, sidewalk, or parking area. Its purpose is to reduce or eliminate the transport of mud from the construction area onto the public right-of-way by motor vehicles or by runoff. An additional track-out area should be established where appropriate if traffic from heavy equipment is limited to areas not typically disturbed by passenger vehicles entering / leaving the construction site.

Construction Entrance/Exits. A stabilized rock exit is required on construction sites. Rock exits must be at least 20 feet wide by 20 feet long (1 & 2 family residential) or 50 feet long (all other construction sites) by 6-inch-thick stabilized rock having a minimum average diameter of 3 inches. If there is an existing curb, loose material such as fill dirt or gravel shall not be used to ramp up to it from the street. Temporary wooden ramps in front of curbs are acceptable.

Dust Control. Saturate ground or apply dust suppressors. Keeping dust down to tolerable limits on the construction site and haul roads is very important.

Outlet Protection. Outlet protection reduces the speed of concentrated stormwater flows. Stone, riprap, concrete aprons, paved sections and settling ponds below outlets prevent scouring and erosion around the outlet. Outlet protection should be applied at locations of all pipes, dike, swale, and channel outlets. Outlet protection should be installed early in the development process and can be added later as necessary to prevent erosion.

Sediment Basins and Rock Check Dams. A rock check dam is an auxiliary structure installed in combination with and as a part of a diversion, interceptor, or perimeter dike, or other structures designed to temporarily detain sediment-laden stormwater. The rock check dam provides a means of draining off and filtering the stormwater while retaining the sediment behind the structure.

Sediment basins can be used to trap runoff waters and sediment from disturbed areas. The water is temporarily detained to allow sediment to drop out and be retained in the basin while the water is automatically released. Sediment basins usually consist of a dam or embankment, a pipe outlet, and an emergency spillway. They are usually situated in natural drainageways or at the low corner of the site. In situations where embankments may not be feasible, a basin excavated below the earth's surface may serve the same purpose. A special provision, however, must be made for draining such an impoundment.

Sediment basins may be temporary or permanent. Temporary basins serve only during the construction stage and are eliminated when vegetation is established, and the area is stabilized. Permanent structures are designed to fit into the plan for the permanent installation. Design shall conform to the requirements within this manual. State and local safety regulations must be observed regarding design, warning signs, and fencing of these structures.

Sediment Traps. A sediment trap is a structure of limited capacity designed to create a temporary pond around storm drain inlets or at points where silt-laden stormwater is discharged. It is used to trap sediment on construction sites, prevent storm drains from being blocked, and prevent sediment pollution of watercourses. Sediment traps should also be located where permanent Stormwater Control Measures (SCM) will be constructed.

Silt Fences. This is a temporary barrier constructed across or at the toe of the slope. Its purpose is to intercept and detain sediment from areas one-half acre or smaller where only sheet erosion may be a problem.

Dewatering. All rainwater pumped out of sumps and depressions on construction sites should be clear and free of sediment, and must discharge to a sedimentation pond, sediment bag, or settling tank in such a manner as to not cause additional erosion problems.

Water Control. Subsurface drains used to remove excess groundwater are sometimes required at the base of fill slopes or around building foundations. When heavy grading is done and natural water channels are filled, the subsurface drains may be used to prevent accumulation of groundwater. Subsurface drains may be needed in vegetated channels to lower a high-water table and to improve drainage conditions so vegetation can be established and maintained.

9.6 Design of Erosion and Sediment Controls

Universal Soil Loss Equation

The Universal Soil Loss Equation adopted by the City of Little Rock is:

$$\boldsymbol{A = R(K)(L)(S)(C)(P)} \qquad \textbf{Eq. 9.1}$$

Where: A= soil loss in tons per acre per year

R = rainfall and runoff factor

K = soil erodibility factor

L = slope length factor

S = slope-steepness factor

c = cover and management factor

P = practice factor

The rainfall and runoff factor (R) is 300 for Pulaski County, Arkansas. Therefore, the Universal Soil Loss Equation takes the following form:

$$\boldsymbol{A = 300(K)(L)(S)(C)(P)} \qquad \textbf{Eq. 9.2}$$

Soil erodibility factors K are presented in Table D-1 (Appendix D) for all soils identified by soil classification within Pulaski County, Arkansas. Soil association maps are found in the Soil survey of

Pulaski County, Arkansas, published by the USDA Soil Conservation Service (See References, Appendix E.)

The slope length factor (L) and the slope-steepness factor (S) can be combined and identified as the topographic factor (LS). Values for LS are found in Table D-2 (Appendix D), which identifies the topographic factor for specific combinations of slope length and steepness.

The cover and management factor (C) represents the ratio of soil loss from land managed through mulching, vegetation, and revegetation to soil loss from disturbed and unprotected lands. Where site preparation removes all vegetation and the root zone of plants, the soil is left completely unprotected and the value of C = 1. Cover and management factors for mulching are presented in Table D-3 (Appendix D). Cover and management factors for vegetation practices are presented in Table D-4 (Appendix D).

The practice factor (P) represents the ratio of soil loss with a specific management practice to the corresponding loss from unprotected slopes. Where no management practice for erosion control is provided, the value of P = 1. Practice factors for gradient terraces, earth dikes, and interceptor dikes and swales are presented in Table D-5 (Appendix D). Practice factors for buffer zones, filter strips and natural vegetation are presented in Table D-6 (Appendix D).

The above information, when incorporated into the Universal Soil Loss Equation, produces the following:

$$\boldsymbol{A = 300(K)(LS)(C)(P)} \qquad \textbf{Eq. 9.3}$$

Where: A = soil loss in tons/acre/year

R = 300, a constant

K = soil erodibility factor (Table D-1)

LS = slope length factor (Table D-2)

C = cover & management factor (Tables D-3,4)

P = practice factor (Tables D-5, 6)

9.7 References

- Fayetteville Drainage Criteria Manual, 2014.
- Little Rock Stormwater Management Drainage Manual, 2016.

Appendix A: Report and Plan Checklists

Final Drainage Report Template and Checklist

The City of Little Rock, Arkansas

Project name _____

Engineer of Record _____

Planning Project Number _____

Revision no. _____

Date _____

Submittal should include the following:

1. ☐ **PROJECT TITLE & DATE**
2. ☐ **PROJECT LOCATION** - Include street address and Vicinity Map.
3. ☐ **PROJECT DESCRIPTION** - Brief description of the proposed project.
4. ☐ **NAME, ADDRESS, TELEPHONE NUMBER, AND EMAIL** of the owner and developer of the property to be permitted.
5. ☐ **NARRATIVE SUMMARY** - The summary shall include a description of the methods used to meet the conveyance, detention, and water quality requirements. This includes at a minimum a description of the target pollutants and treatment train for water quality and a description of the detention strategy used to meet the downstream flood protection requirement. Also include a description of the off-site areas, onsite areas, condition of the downstream receiving areas, existing problems, changes to flows and flow volume, proposed improvements, detention, areas with potential for high pollutant loading, and final conclusions.
6. ☐ **EXISTING DRAINAGE AREA MAP** – Existing drainage area map on a 1-inch = 200-feet minimum scale plan drawing, with 2 foot contours (1 foot contours on “flat” sites), that includes: study points at property lines, time of concentration path, bar scale, and the following information:
    a. ☐ Aerial photograph of the project vicinity, covering the project area and the total lands that contribute runoff;
    b. ☐ Existing drainage areas and flow patterns to downstream property line, establishing the study points;
    c. ☐ Upstream and downstream drainage flow paths for all areas that contribute runoff to the existing site or receive runoff from the site. The downstream area(s) shall be shown as necessary to document the receiving conveyance system; and
    d. ☐ Existing land use conditions for the drainage areas that contribute runoff.

7. ☐ **SOIL MAP** - Provide the most recent U.S. Soil Conservation Service soils and vegetation information for both the project area and the drainage area that contributes runoff on a separate map from the Existing Drainage Area Map.

8. ☐ **PROPOSED DRAINAGE AREA MAP** – Proposed drainage area site map on a 1-inch = 200-feet minimum scale plan drawing, with 2' contours (1 foot contours on "flat" sites), that include: study points, time of concentration path, bar scale, and the following information:

   a. ☐ Proposed drainage areas and flow patterns and, if applicable, natural feature protection areas, green stormwater practice and infiltration areas;

   b. ☐ Upstream and downstream drainage flow paths for all areas that contribute runoff to the proposed development site or receive runoff from the site. The downstream area(s) shall be shown as necessary to document the receiving conveyance system;

   c. ☐ Proposed land use conditions for the development site and drainage areas that contribute runoff; and

   d. ☐ Proposed locations of grading and placement of fill material within the project area and drainage areas that contribute runoff.

9. ☐ **WATER QUALITY** – Calculations and documentation indicating the target pollutants and the required water quality treatment volume.

   a. ☐ Provide calculations for each structural control indicating the corresponding level of treatment; and,

   b. ☐ Provide a map showing the impervious area and structural controls

10. ☐ **DOWNSTREAM FLOOD PROTECTION** – Provide calculations and documentation indicating that the post-development peak discharge rate does not exceed the pre-development rate for the 2-year, 25-year, and 100-year, 24-hour storm events. The calculations shall include the following information:

    a. ☐ A summary table of runoff discharge flows for the 2-year, 25-year, and 100-year, 24- hour storm events for the pre-development and post-development conditions for each study point. The summary shall include the existing and proposed flows along with supporting calculations for all of the discharge points to the receiving system. This includes the flow entering each drainage area and the flow generated within each drainage area on the site (do not separate onsite and offsite flows).

    b. ☐ The effects of the 100-year, 24-hour storm event on the stormwater management system, adjacent property, and downstream facilities and property shall be evaluated. The 100-year flow shall be controlled through the use of structural stormwater controls to protect existing downstream property with no increase in the existing base flood elevation, or calculations shall be provided to

indicate that the on-site conveyance system will safely pass the flow and allow it to discharge into receiving waters where the floodplain is of capacity sufficient to accommodate significant additional discharges without causing damage.

11. ☐ **CHECK FOR EXISTING DOWNSTREAM FLOODING** – Describe the existing downstream capacity of each receiving area (study point). Provide documentation of a downstream assessment using the 10% Rule in accordance with Section 7.5.3. Documentation shall include photographs of the existing structures downstream of the development as well as a map showing the locations and distances of downstream structures from the development.

12. ☐ **STORMWATER DETENTION DESIGN** – If detention is required, include all computations and backup/support data including:

    a. ☐ Detention basin size requirement computations (using an approved method).

    b. ☐ Release structure design computations including design Water Surface Elevations for the 2-year, 25-year, and 100-year storms.

    c. ☐ Stage-Storage and Stage-Discharge curves for the detention facility.

    d. ☐ A summary hydrograph of the effect of the detention facility for relevant storms, incorporated with bypass.

    e. ☐ Overflow structure(s) size and location(s);

    f. ☐ Outfall structure(s), location(s), and orifice size(s).

    g. ☐ Emergency overflow path.

    h. ☐ Results of downstream analysis.

13. ☐ **PAVEMENT DRAINAGE DESIGN** – Include a table listing street classification, width, allowable spread and actual spread for design storm.

14. ☐ **STORM SEWER INLET DESIGN** - Include all computations for the design storm. Reference Table 4.3 in Chapter 4 for allowable spread and depth.

15. ☐ **INLET DRAINAGE AREA MAP** – Provide a separate map showing the inlet layout and design including the drainage areas. The map should include the proposed design, drainage areas, time of concentration paths, runoff coefficients, and bar scale.

16. ☐ **STORM SEWER DESIGN** - Include all computations and hydraulic profiles for the design storm and 100-year, 24-hour storm.

17. ☐ **CULVERT DESIGN** - Include all computations, hydraulic profile, and energy transition to channel.

18. ☐ **OPEN CHANNEL FLOW DESIGN** - Include computations for normal depth and velocity. Specify required channel lining if necessary to mitigate velocity impacts on stability.

19. ☐ **FEDERAL AND STATE REQUIREMENTS** (if required).

a. ☐ Wetlands determination (if wetlands are present on the site).

b. ☐ 404 permit required (include letter from USACE as an exhibit).

c. ☐ NPDES Construction Stormwater "Notice of Intent" (ADEQ)(include as an exhibit if required).

d. ☐ ANRC permit/review for "dams"(required if a stormwater impoundment qualifies as a dam per ANRC regulations).

e. ☐ Floodplain Development permit (required if proposed development is located within 100-year regulatory floodplain as defined by FEMA FIRM Panel)

f. ☐ Other

20. ☐ **EXHIBITS** – Attach the following exhibits to the final drainage report.

a. ☐ Grading and drainage construction drawings.

b. ☐ Landscaping Plan.

c. ☐ Operations and maintenance plan.

d. ☐ Letter from USACE if answered Yes to 19.b. above.

e. ☐ Notice of Coverage (NOC) and completed SWPPP (sites 1 acre or larger).

f. ☐ Master Drainage Plan (if part of a larger or phased project).

21. ☐ The following paragraph with relevant information included:

"I, _____, Registered Professional Engineer No. _____in the State of Arkansas, hereby certify that the drainage studies, reports, calculations, designs, and specifications contained in this report have been prepared in accordance with sound engineering practice and principles, and the requirements of the City of Little Rock. Further, I hereby acknowledge that the review of the drainage studies, reports, calculations, designs, and specifications by the City of Little Rock or its representatives cannot and does not relieve me from any professional responsibility or liability."

_____

Signed & Sealed by
Professional Engineer

22. ☐ ARKANSAS REGISTERED ENGINEER SEAL

Electronic As-Built Domain Values

- **Control Features**
  - *Control ID* – unique number corresponding to the feature's annotation on the drawing.
  - *Control type* – type of control device*.
  - Diameter.
  - *Location Description* – Text description of the physical location of this feature.
  - *Owner* – The name of the city the conveyance is located in or responsible for maintenance.
  - *Date of Information* – The date of most recent update or verification of data.
  - *Installation Date* – The date of actual construction or installation.
  - *As-built File Name* – Filename for linking a document or drawing file.
  - Comments.
- **Linear Features**
  - *Linear ID* – Unique number corresponding to the feature's annotation on the drawing.
  - *Linear Type* – Type of linear feature*.
  - *Quantity* – Number of identical parallel parts at location, such as a multi-barrel culvert.
  - *Pipe Size* – Diameter of a round pipe (or round-equivalent for other shapes), in inches.
  - *Cross Section Shape* – Shape of the cross section of the linear feature or conveyance.
  - *Material* – Material forming the linear feature or conveyance.
  - *Depth* – Depth of an open channel, or interior height of a covered conveyance, in inches.
  - *Top Width* – Width of the top of the feature opening or open channel, in inches.
  - *Bottom Width* – Width of the bottom of the feature opening or open channel, in inches.
  - Upstream Invert.
  - Downstream Invert.
  - Slope.
  - Date of Information.
  - *Owner* – The name of the city the conveyance is located in or responsible for maintenance.
  - *Installation Date* – The date of actual construction or installation.
  - *As-built File Name* – Filename for linking a document or drawing file.
  - Comments/Notes.
- **Junction Features**
  - *Junction ID* – Unique number corresponding to the feature's annotation of the drawing.
  - *Junction Type* – Type of network junction feature*.
  - *Box Type* – Type or function of the Stormwater box. (Leave blank for other Junction types) *.
  - *Material* – Construction material of the structure*.
  - *Curb Inlet Wing* – Approximate width of opening.
  - *Head-wall Wing* – Wing walls at a Headwall/End wall.
  - *Flared End* – Flared End Section Yes/No.
  - *Trash Rack* – Whether an inlet has a trash rack to keep trash or debris from entering (Y/N).

- *Manhole* – Whether the structure has a manhole entry (Y/N).
- *Access Type* – Method of access entry into structure.
- *Access Diameter* – Size of access, in inches.
- *Throat Width* – Width of inlet opening, in inches.
- *Throat Area* – Area of inlet opening, in square feet.
- *Inlet Dimensions* – Text description for dimensions of an irregular curb inlet.
- *Grate Size* – Text description for dimensions of a grate opening.
- *Top Elevation* – Elevation of the top of the structure, or manhole rim, in decimal feet.
- *Outlet Invert Elevation* – Elevation of the invert of the flow outlet, in decimal feet.
- *Inlet (1, 2, 3, etc.) Invert Elevations* – Elevation of the invert of each of the flow inlets (clockwise from outlet), in decimal feet.
- *Box Depth* – Depth from the top of the structure is located in or responsible from maintenance.
- *Owner* – The name of the city the structure is located in or responsible for maintenance.
- *Date of Information* – The date of most recent update or verification of date.
- *As-Built File Name* – File name for linking a document or drawing file.
- Comment.

* NOTE: For list of possible values for each attribute column see “SW Attribute Data Entry Specifications.pdf” as provided by City of Little Rock Public Works. If the material is not listed, choose “Other” and describe in the comments field.

Appendix B: Example Submittal

DRAINAGE REPORT

Sources for Community Independent Living

Apple Blossom Lane
Little Rock, Pulaski County, Arkansas

May 4, 2025

Prepared for:

BLACK BEAR BUILDERS

Prepared By:

Wholee Associates, Inc.
1836 Tusk Parkway, Suite 500, Little Rock,
Arkansas, 72211 (Tel) 501 123 4567 (Fax) 501 765 4321
engineer@email.com

DRAINAGE REPORT
LIST OF EXHIBITS AND APPENDICES

EXHIBIT I	Location Map
EXHIBIT II	FEMA FIRMette
EXHIBIT III	Hydrologic Data
EXHIBIT IV	Pre-Developed Drainage Map
EXHIBIT V	Post-Developed Drainage Map
EXHIBIT VI	Site Drainage Map
APPENDIX	USGA Soil Report
APPENDIX	Hydrologic Calculations
APPENDIX	Detention System Details
APPENDIX	Hydraulic Calculations
APPENDIX	Maintenance Agreement

Drainage Report
for
Sources for Community Independent Living
Apple Blossom Lane
Little Rock, Pulaski County, Arkansas

A. GENERAL

An approximately 13,390 sq ft. 2 story Independent Living Facility is being proposed on the west side of Apple Blossom Lane in Little Rock, Arkansas as shown in Exhibit I. The project also includes new asphalt parking lots and driveways, associated heavy duty concrete driving lanes, associated sidewalk and landscaping, new storm drains and inlets, an underground detention system, as well as other site improvements.

The project area is located outside the FEMA designated floodplain, as shown in Exhibit II.

The targeted primary pollutant for the site is heavy metals and total suspended solids (TSS). Water quality treatment will be provided through bioretention in a parking lot island and additional treatment will be provided in the underground detention system.

Peak stormwater runoff for the collection and conveyance system were calculated, in accordance with the City of Little Rock Drainage Manual using the Rational Method. Detention calculations were performed using the SCS Method. Rainfall intensities and depths were obtained from the Little Rock drainage manual and this hydrological information can be found in Exhibit III.

Hydrologic calculations were conducted using Hydraflow Hydrographs software. Pre- and post-development peak flows were calculated for the 2, 5, 10, 25, and 100-year events. Hydraulic and inlet calculations were done using StormCAD and the systems were evaluated for the 25-year and 100-year storm events. A downstream assessment was performed to determine potential downstream impacts.

B. PRE-DEVELOPED CONDITIONS

As shown on the predeveloped drainage area map, Exhibit IV, the proposed location of development is primarily grass covered with an area of tree cover and brush located on the west side of the site. Additionally, the site drops nearly 30 feet in elevation from the west side of the site to the east where Apple Blossom Lane is located. A majority of the site (approximately 1.79 acres) drains to an existing curb inlet located on Apple Blossom Lane. This curb inlet has been designated as Study Point “A” for the purposes of drainage analysis. The other portion of the site (approximately 0.83 acres) drains to an existing curb inlet just south of the southeast corner of the project site. This inlet has been designated Study Point “B” for the purpose of drainage analysis.

A USGS soils report of the site can be found in the submittal Appendix .

The following runoff coefficients were used to calculate composite coefficients for each catchment:

Table 1: Runoff Coefficients

Surface	Runoff Coefficient
Grass (Average)	69
Pavement/Roof	98

The calculation results for the peak pre-developed flows at the Study Point are contained in Appendix B and summarized in the following table:

Table 2. Pre-developed Peak Flow Summary at the Study Points

Pre-Developed Flow	Q_2 (cfs)	Q_5 (cfs)	Q_{10} (cfs)	Q_{25} (cfs)	Q_{100} (cfs)
Study Point A	3.7	5.6	7.2	9.9	14.6
Study Point B	2.1	3.0	3.9	5.1	7.3

C. POST-DEVELOPED CONDITIONS

Stormwater Hydrology

As shown on the Post-Development Drainage Area Map, Exhibit V, most of the site is directed to onsite collection systems that discharges into a proposed underground ground detention system, then to Study Point "A". The remaining runoff will be directed to a proposed storm drain that ties into the existing system on the private drive, thence to Study Point "B". Results of the post-developed hydrologic calculations are included in the submittal Appendix .

Stormwater Quantity

Section 2.1.2 (Downstream Flood Protection) of the Little Rock stormwater drainage manual requires that new developments not increase the peak flow as compared to pre-development conditions. In order to meet this requirement, an underground detention system will be provided. The system will be the Advanced Drainage Systems, Inc. (ADS) Stormtech Chamber. As shown by the technical information in the submittal Appendix, the detention system has a maximum storage volume of 16,380 cubic feet. This exceeds the storage required for the 100-year storm event of 16,277 cubic feet. A 10" orifice in the outlet structure will control the discharge rate from the detention system. The detention system performance results using the modified rational method are included in the submittal Appendix and summarized in Tables 3, 4, and 5 below.

Table 3: Pond Performance Summary

Underground Storage Pond	2-Year	5-Year	10-Year	25-Year	100-Year
Maximum Water Elevation	1,230.34	1,230.67	1,230.97	1,231.24	1,231.70
Maximum Storage (cubic feet)	4,339	6,362	8,237	11,121	16,277
Peak Inflow (cfs)	6.8	8.9	10.8	13.6	18.1
Peak Discharge (cfs)	2.2	2.5	2.6	2.8	3.0
Difference (cfs)	**-4.6**	**-6.4**	**-8.2**	**-10.8**	**-15.1**

Table 4: Pre-developed vs Post-developed Peak Flow Summary at Study Point "A"

Outfall	Q_2 (cfs)	Q_5 (cfs)	Q_{10} (cfs)	Q_{25} (cfs)	Q_{100} (cfs)
Pre-Developed	3.7	5.6	7.2	9.9	14.6
Post-Developed	2.7	3.1	3.5	4.1	5.0
Difference (cfs)	**-1.0**	**-2.5**	**-3.7**	**-5.8**	**-9.6**

Table 5: Pre-developed vs Post-developed Peak Flow Summary at Study Point "B"

Outfall	Q_2 (cfs)	Q_5 (cfs)	Q_{10} (cfs)	Q_{25} (cfs)	Q_{100} (cfs)
Pre-Developed	2.1	3.0	3.9	5.1	7.3
Post-Developed	2.1	2.9	3.6	4.8	6.7
Difference (cfs)	**0.0**	**-0.1**	**-0.3**	**-0.3**	**-0.6**

As shown in Table 4 and 5, the post-development peak flows are equal to or lower than their pre-development counterparts for all storm events at study point "A" and study point "B", respectively.

D. Water Quality

The ADS System also meets the requirements of Section 2.1.1 (Water Quality) of the Little Rock stormwater drainage manual via the TSS Reduction Method. According to the City of Little Rock Drainage Criteria Manual, the stormwater system shall be designed to be capable of removing at least 80% of TSS from an equivalent onsite impervious area. The detention volume of the ADS Stormtech system is sufficient in holding the Water Quality Volume equal to the first 1.05 inches of rainfall. The total site area is 2.0 acres; the R_v value was determined using 55% impervious cover of the catchment area.

$R_v = 0.05 + 0.009(I)$
$R_v = 0.05 + 0.009(55) = 0.545$
$WQV = (1.05 * R_v * A)/12$
$WQV = (1.05 * 0.545 * 2.0)/12$
$WQV = 0.095$ acre-ft
WQV = 4,155 cubic ft

The total volume of the Stormtech chambers is 16,380 CF, which is above the WQv of 4,155 CF. The bioretention area, located in the parking lot island, is used as pre-treatment for the underground detention to remove heavy metals from the parking lot, reduce TSS loading, and catch floatables. Specifications for the ADS system can be found in the submittal Appendix .

E. Downstream Analysis

A downstream analysis was completed to determine potential impacts. The site area and the contributing drainage areas to the limit of the zone of influence (10% point) is shown in Exhibit VI. Hydraflow Hydrographs software was used to assess the peak flow and timing of the watershed. Figures 1 and 2 provide the pre-development and post-development hydrographs at the 10% point. The calculations and results for all storm events can be found in the submittal Appendix.

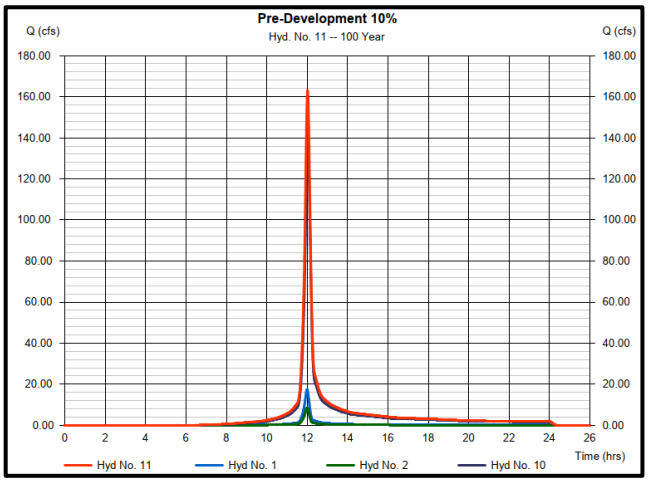


Figure 1: Pre-Development

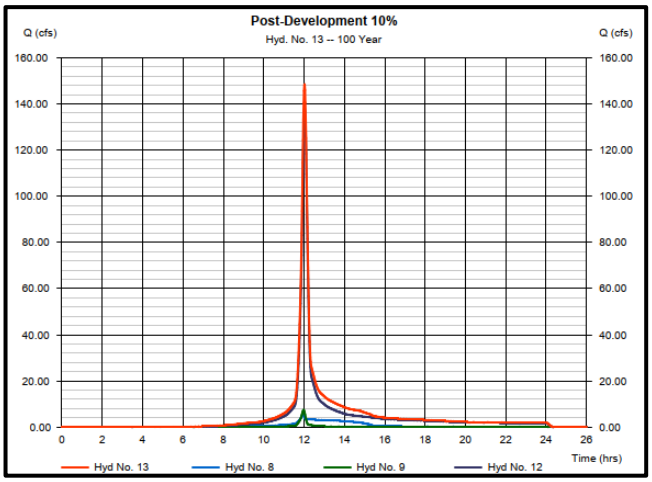


Figure 2: Post Development

F. Storm System Design

Hydraulic calculations for the proposed storm drains were done using Rational Method hydrology and StormCAD software. The contributing drainage areas to the drainage systems are shown in Exhibit VII. The results of the hydraulic and inlet calculations are contained in the submittal Appendix. A summary of the roadway inlets is provided in Table 6. As shown, the storm sewer system is adequate to convey the 25-Year storm event to discharge the points without the surcharge. Analysis of the system under the 100-Year storm event was also completed and can be found in the submittal Appendix.

Table 6: Roadway Inlet Calculations

Inlet	Roadway Classification	Roadway Width (ft)	Allowable Spread	Actual Spread
CB-6	Local	30	10	6.4
CB-7	Local	30	10	4.9
CB-9	Local	30	10	6.1

G. Conclusion

As demonstrated above by the supporting calculations, the proposed drainage system meets the requirements of the City of Little Rock.

I, Stoney Little , Registered Professional Engineer No. 12345 in the State of Arkansas, hereby certify that the drainage studies, reports, calculations, designs, and specifications contained in this report have been prepared in accordance with sound engineering practice and principles, and the requirements of the City of Little Rock. Further, I hereby acknowledge that the review of the drainage studies, reports, calculations, designs, and specifications by the City of Little Rock or its representatives cannot and does not relieve me from any professional responsibility or liability.

WHOLEE Associates, Inc.

Stoney Little, P.E.
Project Manager

Sawyer Pine, E.I.
Project Engineer

Final Drainage Report Template and Checklist

The City of Little Rock, Arkansas

Project name Apple Blossom Development

Engineer of Record _____

Planning Project Number 1 2 3 4

Revision no. 567

Date 5/04/2025

Submittal should include the following:

1. ☒ **PROJECT TITLE & DATE**
2. ☒ **PROJECT LOCATION** - Include street address and Vicinity Map.
3. ☒ **PROJECT DESCRIPTION** - Brief description of the proposed project.
4. ☒ **NAME, ADDRESS, TELEPHONE NUMBER, AND EMAIL** of the owner and developer of the property to be permitted.
5. ☒ **NARRATIVE SUMMARY** - The summary shall include a description of the methods used to meet the conveyance, detention, and water quality requirements. This includes at a minimum a description of the target pollutants and treatment train for water quality and a description of the detention strategy used to meet the downstream flood protection requirement. Also include a description of the off-site areas, onsite areas, condition of the downstream receiving areas, existing problems, changes to flows and flow volume, proposed improvements, detention, areas with potential for high pollutant loading, and final conclusions.
6. ☒ **EXISTING DRAINAGE AREA MAP** – Existing drainage area map on a 1-inch = 200-feet minimum scale plan drawing, with 2 foot contours (1 foot contours on “flat” sites), that includes: study points at property lines, time of concentration path, bar scale, and the following information:
    a. ☐ Aerial photograph of the project vicinity, covering the project area and the total lands that contribute runoff;
    b. ☐ Existing drainage areas and flow patterns to downstream property line, establishing the study points;
    c. ☐ Upstream and downstream drainage flow paths for all areas that contribute runoff to the existing site or receive runoff from the site. The downstream area(s) shall be shown as necessary to document the receiving conveyance system; and
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7. ☒ **SOIL MAP** - Provide the most recent U.S. Soil Conservation Service soils and vegetation information for both the project area and the drainage area that contributes runoff on a separate map from the Existing Drainage Area Map.

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   c. ☐ Proposed land use conditions for the development site and drainage areas that contribute runoff; and

   d. ☐ Proposed locations of grading and placement of fill material within the project area and drainage areas that contribute runoff.

9. ☒ **WATER QUALITY** – Calculations and documentation indicating the target pollutants and the required water quality treatment volume.

   a. ☐ Provide calculations for each structural control indicating the corresponding level of treatment; and,

   b. ☐ Provide a map showing the impervious area and structural controls

10. ☒ **DOWNSTREAM FLOOD PROTECTION** – Provide calculations and documentation indicating that the post-development peak discharge rate does not exceed the pre-development rate for the 2-year, 25-year, and 100-year, 24-hour storm events. The calculations shall include the following information:

    a. ☐ A summary table of runoff discharge flows for the 2-year, 25-year, and 100-year, 24- hour storm events for the pre-development and post-development conditions for each study point. The summary shall include the existing and proposed flows along with supporting calculations for all of the discharge points to the receiving system. This includes the flow entering each drainage area and the flow generated within each drainage area on the site (do not separate onsite and offsite flows).

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    c. ☐ Stage-Storage and Stage-Discharge curves for the detention facility.

    d. ☐ A summary hydrograph of the effect of the detention facility for relevant storms, incorporated with bypass.

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19. ☐ **FEDERAL AND STATE REQUIREMENTS** (Answer Yes or No if required).
    a. ☐ Wetlands determination (if wetlands are present on the site).
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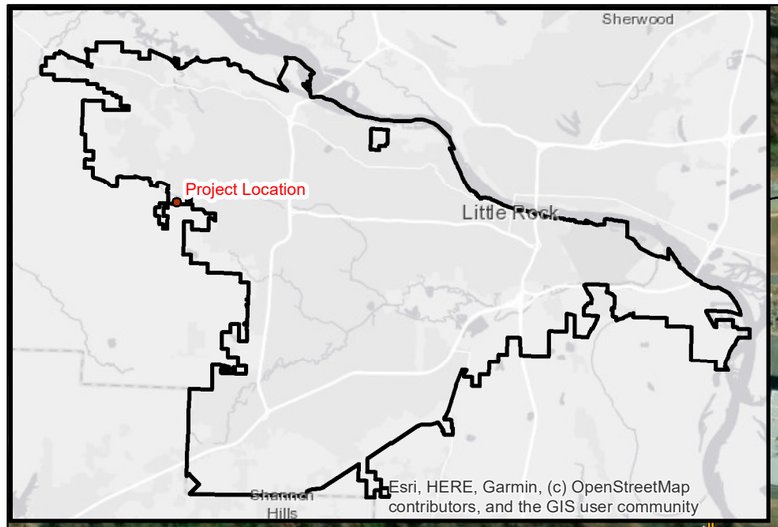
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_____

Signed & Sealed by
Professional Engineer

22. ☐ ARKANSAS REGISTERED ENGINEER SEAL

EXHIBIT I



ChenalPkwy
ChenalPkwy
KirkRd
Delta Trust And Bank
Arvest Bank
Kroger
Little Rock
Highland Valley Church
ChenalPky
Unnamed
ChenalPky
First Arkansas Bank And Trust
Bldg D
Southwest Power Pool
AppleBlossomLn
WorthenDr
EdswoodCt
GauntletLn
LaymanLn
ZanzibarSt
NationSt
Pride ValleyRd

Legend

- Development Site
- Drainage Area
- Roadway
- City Boundary

Esri, HERE, Garmin, (c) OpenStreetMap contributors, and the GIS user community, Source: Esri, Maxar, Earthstar Geographics, and the GIS User Community

WHOLEE

1836 Tusk Pkwy
Little Rock, AR 72205

N
W E
S

APPLE BLOSSOM DEVELOPMENT LOCATION MAP

0 5,000 10,000 Feet

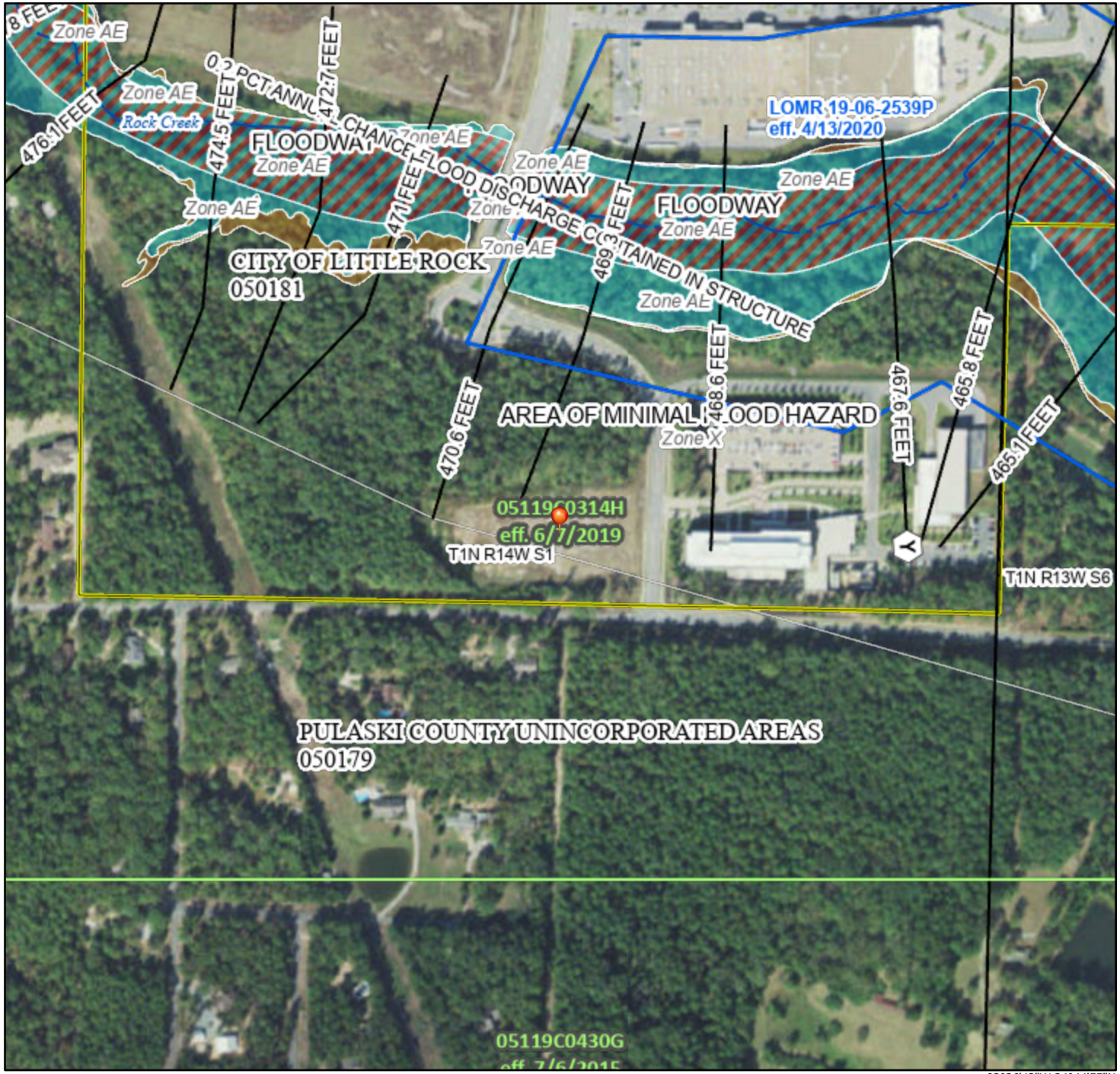
1234.56
Black Bear Builders
Site Drainage

EXHIBIT II

National Flood Hazard Layer FIRMette

FEMA

92°27'21"W 34°45'24"N



92°26'43"W 34°44'55"N

0 250 500 1,000 1,500 2,000 Feet

1:6,000

Basemap Imagery Source: USGS National Map 2023

Legend

SEE FIS REPORT FOR DETAILED LEGEND AND INDEX MAP FOR FIRM PANEL LAYOUT

SPECIAL FLOOD HAZARD AREAS
- Without Base Flood Elevation (BFE) *Zone A, V, A99*
- With BFE or Depth *Zone AE, AO, AH, VE, AR*
- Regulatory Floodway

OTHER AREAS OF FLOOD HAZARD
- 0.2% Annual Chance Flood Hazard, Areas of 1% annual chance flood with average depth less than one foot or with drainage areas of less than one square mile *Zone X*
- Future Conditions 1% Annual Chance Flood Hazard *Zone X*
- Area with Reduced Flood Risk due to Levee. See Notes. *Zone X*
- Area with Flood Risk due to Levee *Zone D*

OTHER AREAS
- NO SCREEN Area of Minimal Flood Hazard *Zone X*
- Effective LOMRs
- Area of Undetermined Flood Hazard *Zone D*

GENERAL STRUCTURES
- Channel, Culvert, or Storm Sewer
- Levee, Dike, or Floodwall

OTHER FEATURES
- 20.2 Cross Sections with 1% Annual Chance
- 17.5 Water Surface Elevation
- Coastal Transect
- Base Flood Elevation Line (BFE)
- Limit of Study
- Jurisdiction Boundary
- Coastal Transect Baseline
- Profile Baseline
- Hydrographic Feature

MAP PANELS
- Digital Data Available
- No Digital Data Available
- Unmapped

N

The pin displayed on the map is an approximate point selected by the user and does not represent an authoritative property location.

This map complies with FEMA's standards for the use of digital flood maps if it is not void as described below. The basemap shown complies with FEMA's basemap accuracy standards

The flood hazard information is derived directly from the authoritative NFHL web services provided by FEMA. This map was exported on 5/6/2024 at 3:12 PM and does not reflect changes or amendments subsequent to this date and time. The NFHL and effective information may change or become superseded by new data over time.

This map image is void if the one or more of the following map elements do not appear: basemap imagery, flood zone labels, legend, scale bar, map creation date, community identifiers, FIRM panel number, and FIRM effective date. Map images for unmapped and unmodernized areas cannot be used for regulatory purposes.

EXHIBIT III

Table 3.1 Rainfall Depths for 24-hour Duration Storm.

Storm Event	2-year	5-year	10-year	25-year	50-year	100-year
Depth (in)	4.19	5.12	5.94	7.14	8.13	9.17

Table 3.2 e,b,d Variable Coefficients.

Return Period	Variable		
	e	***b***	***d***
2yr	0.625	22.461	3.488
5yr	0.628	27.071	3.592
10yr	0.626	30.302	3.550
25yr	0.620	34.032	3.395
50yr	0.619	37.392	3.507
100yr	0.612	39.450	3.253
500yr	0.598	44.067	2.969

Table 3.3 TR-55 Runoff Curve Numbers1 (CN)

Cover type and hydrologic condition[2]	Average percent impervious area[3]	Curve numbers for hydrologic soil groups[1]			
		A	B	C	D
Cultivated land					
Without conservation treatment		72	81	88	91
With conservation treatment		62	71	78	81
Pasture or range land					
Poor condition		68	79	86	89
Good condition		39	61	74	80
Meadow					
Good condition		30	58	71	78
Wood or forest land					
Thin stand, poor cover		45	66	77	83
Good cover		30	55	70	77
Open space (lawns, parks, golf courses, cemeteries, etc.)[4]					
Poor condition (grass cover <50%)		68	79	86	89
Fair condition (grass cover 50% to 75%)		49	69	79	84
Good condition (grass cover > 75%)		39	61	74	80
Impervious areas:					
Paved parking lots, roofs, driveways, etc. (excluding right-of-way)		98	98	98	98
Streets and roads					
Paved; curbs and storm drains (excluding right-of-way)		98	98	98	98
Paved; open ditches (including right-of-way)		83	89	92	93
Gravel (including right-of-way)		76	85	89	91
Dirt (including right-of-way)		72	82	87	89
Urban districts					
Commercial and business	85%	89	92	94	95
Industrial	72%	81	88	91	93
Residential districts by average lot size					
1/8 acre or less (town houses)	65%	77	85	90	92
1/4 acre	38%	61	75	83	87
1/3 acre	30%	57	72	81	86
1/2 acre	25%	54	70	80	85
1 acre	20%	51	68	79	84
2 acres	12%	46	65	77	82
Developing urban areas and newly graded areas (pervious areas only, no vegetation).		77	86	91	94

1. Antecedent Moisture Condition II, and Ia = 0.2S.
2. Areas of compacted earthen fill shall be classified as Hydrologic Soil Group D.
3. The average percent impervious area shown was used to develop the composite CNs.

Other assumptions are as follows: impervious areas are directly connected to the drainage system, impervious areas have a CN of 98, and pervious areas are considered equivalent to open space in good hydrologic condition.

4. CNs shown are equivalent to those of pasture. Composite CNs may be computed for other combinations of open space cover type.

Source: USDA Technical Release 55 (TR-55)

EXHIBIT IV

Predevelopment A
1.79 Acres

Predevelopment B
0.83 Acres

Study Point "A"

Study Point "B"

Apple Blossom Lane

34°44'48"N 92°17'16"W

Existing Chain link Fence

Existing Wall

Existing Sewer Manhole
Rim =1233.18
Inv.(In) =1220.33

Existing Sewer Manhole
Rim =1246.71
Inv.(Out) =1241.68

Existing Sewer Manhole
Rim =1236.28
Inv.(In) =1226.42
Inv.(Out) =1228.07

Existing Sewer Manhole
Rim =1230.38
Inv.(IN) =1221.76
Inv.(Out) =1221.65

Existing Curb Inlet
Rim = 1238.57
Inv. = 1232.58

Existing Curb Inlet
Rim = 1231.28
Inv. = 1225.71

Existing Curb Inlet
W/2-8' Extensions
Rim = 1231.14
Inv. = 1226.82

Existing Curb Inlet
Rim = 1233.45
Inv. = 1226.96

TBM Chiseled X
Elev.=1232.44
NW Corner Of Inlet

Blow Off Valve

20' Utility Easement

24' Access Easement

44' Existing R/W

30' Water & Sewer Easement

18"CMP

8"SS

6"SS

8"W

3"F

Scale In Feet
30 0 30

Revision	By	Date

WHOLEE

Drawn By:	ICF
Approved By:	AD
Date:	05-04-2022
Project No.:	BBD-01

Vertical Scale:	–
Horizontal Scale:	1"=30'
Plotting Scale:	1-1
Drawing Name:	DAM

SOURCES FOR COMMUNITY
INDEPENDENT LIVING

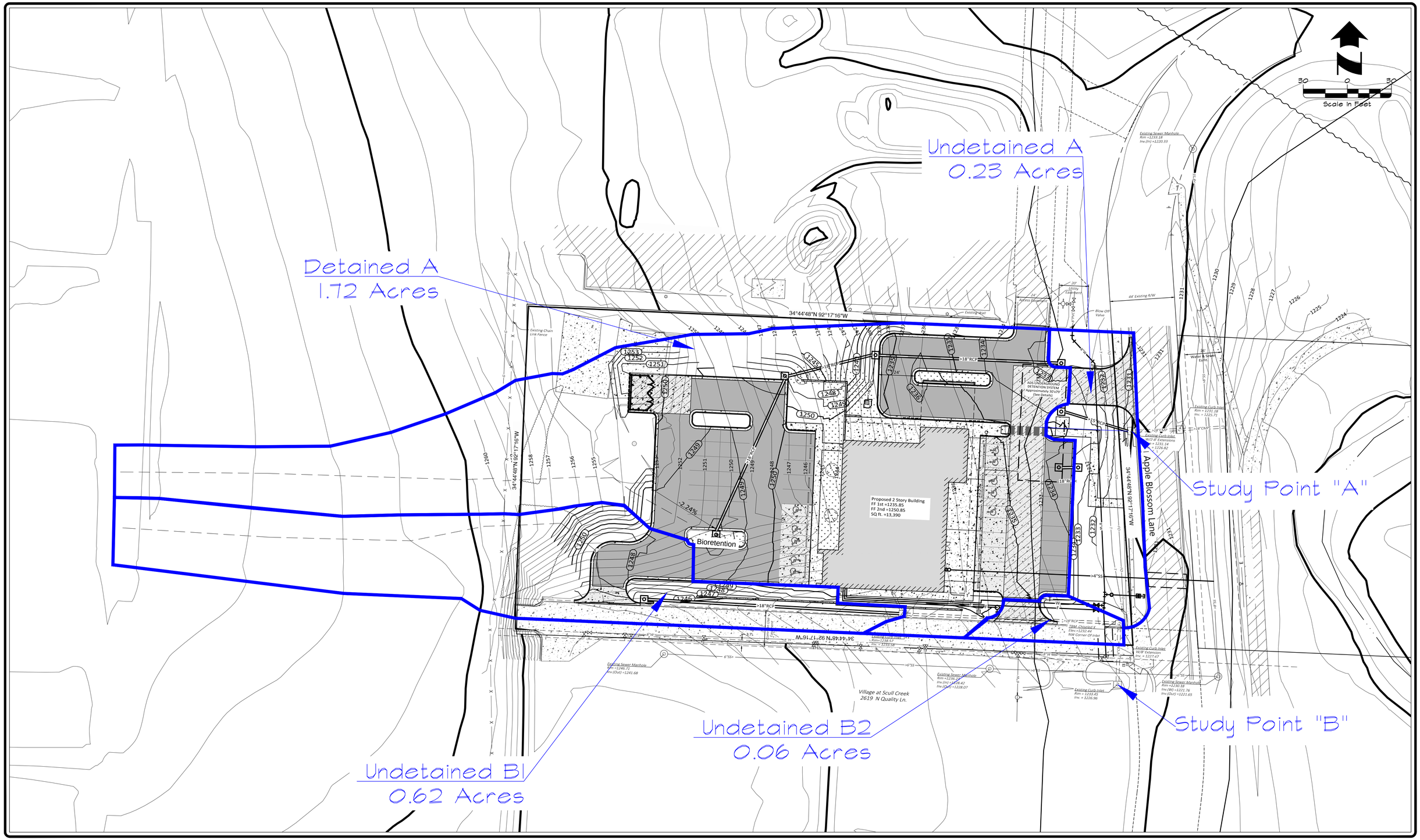
Little Rock, Pulaski County, Arkansas

EXHIBIT IV:
PRE-DEVELOPMENT
DRAINAGE AREA MAP

Issued for Review - 05-04-22

Sheet No.: 1 of 3

EXHIBIT V



Revision	By	Date

WHOLEE

Drawn By:	ICF
Approved By:	AD
Date:	05-04-2022
Project No.:	BBD-01

Vertical Scale:	–
Horizontal Scale:	1"=30'
Plotting Scale:	1-1
Drawing Name:	DAM

SOURCES FOR COMMUNITY INDEPENDENT LIVING

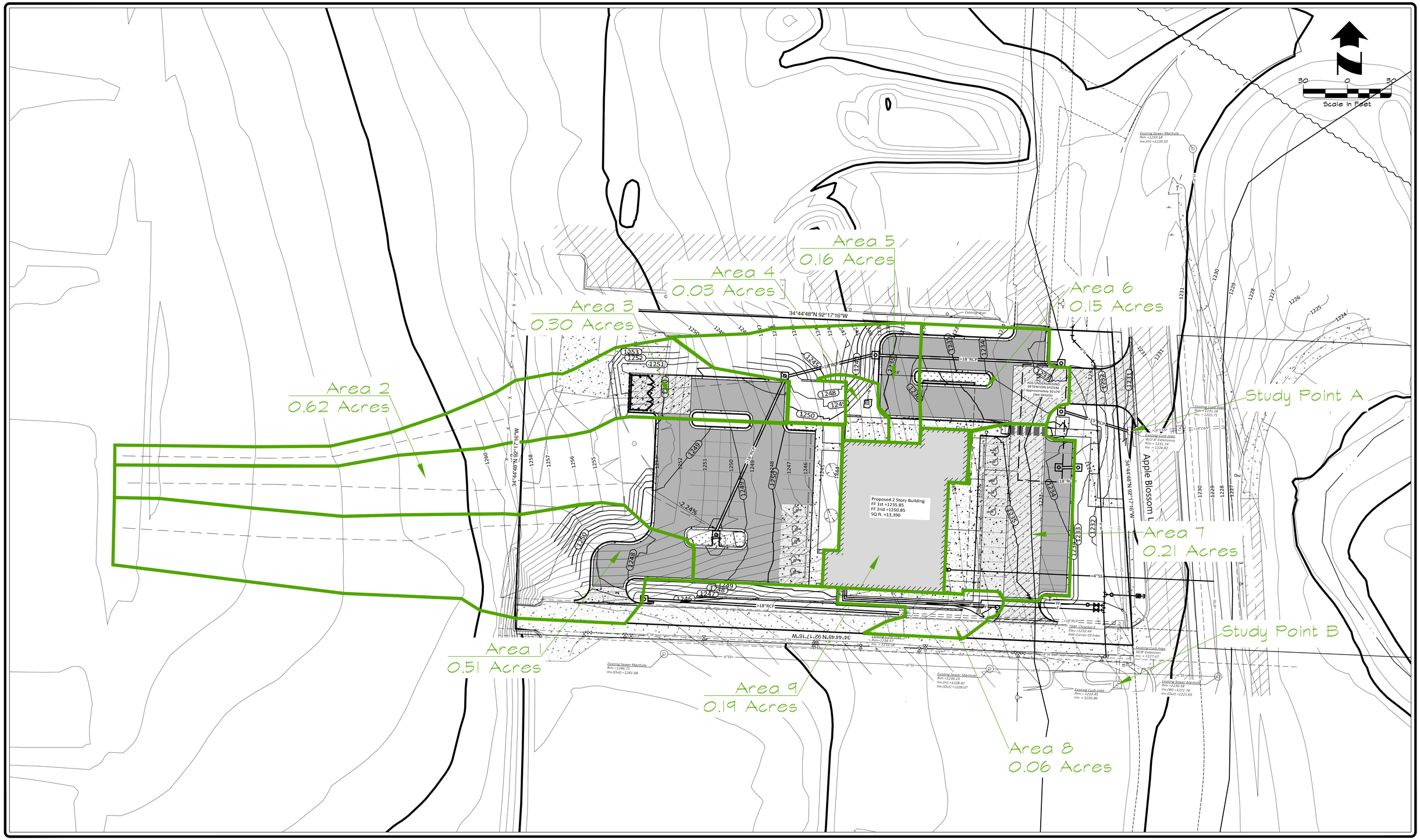
Little Rock, Pulaski County, Arkansas

EXHIBIT V:
POST-DEVELOPMENT
DRAINAGE AREA MAP

Issued for Review - 05.04.22

Sheet No.: 2 of 3

EXHIBIT VI



Revision	By	Date

WHOLEE

Drawn By:	ICF
Approved By:	AD
Date:	05-04-2022
Project No.:	BBD-01

Vertical Scale:	–
Horizontal Scale:	1"=30'
Plotting Scale:	1-1
Drawing Name:	DAM

SOURCES FOR COMMUNITY INDEPENDENT LIVING

Little Rock, Pulaski County, Arkansas

EXHIBIT VI:
SITE DRAINAGE AREA MAP

Issued for Review - 05.04.22

Sheet No.: 3 of 3

APPENDIX

USDA Soil Report

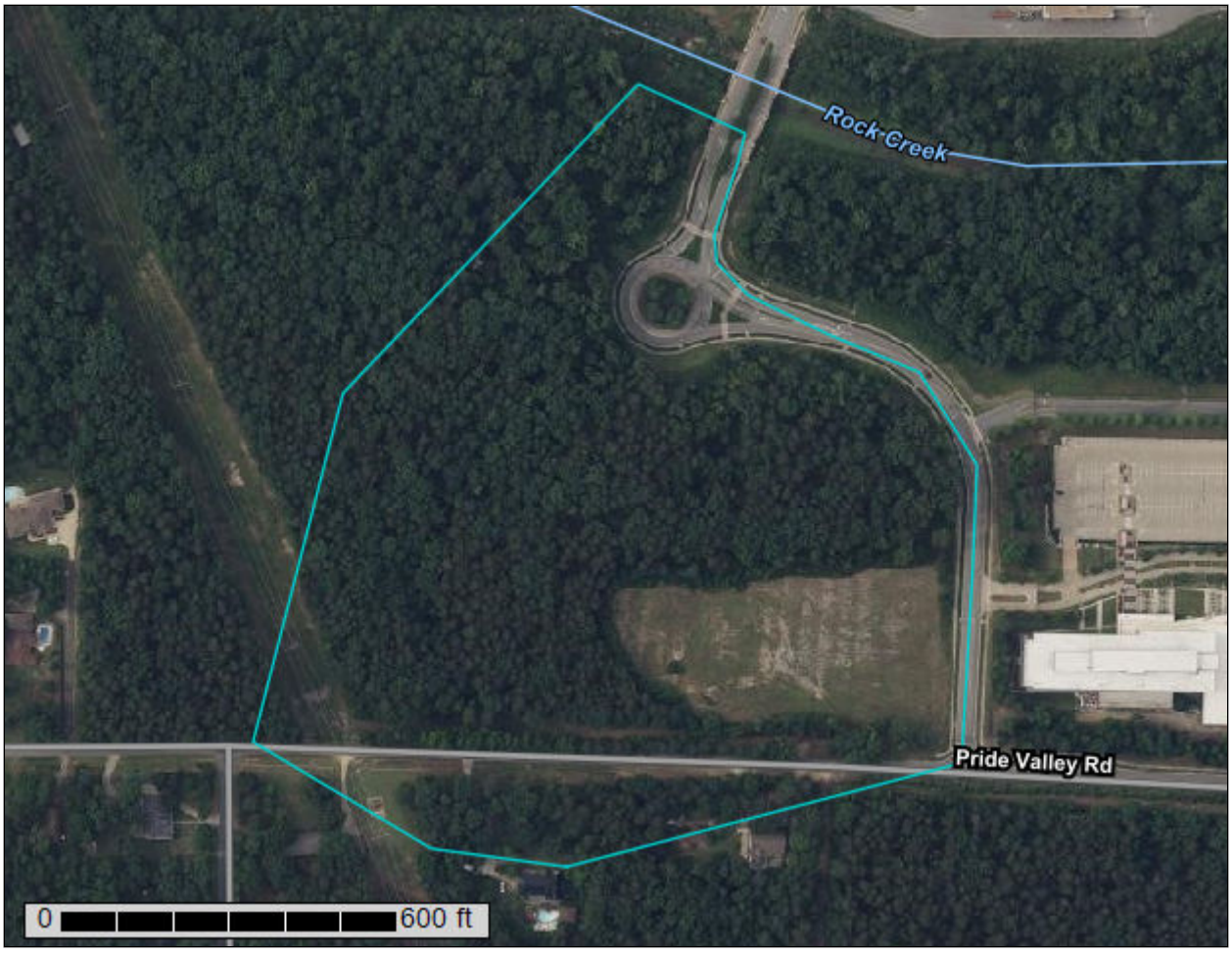
United States Department of Agriculture

NRCS

Natural Resources Conservation Service

A product of the National Cooperative Soil Survey, a joint effort of the United States Department of Agriculture and other Federal agencies, State agencies including the Agricultural Experiment Stations, and local participants

Custom Soil Resource Report for Pulaski County, Arkansas



May 8, 2024

Preface

Soil surveys contain information that affects land use planning in survey areas. They highlight soil limitations that affect various land uses and provide information about the properties of the soils in the survey areas. Soil surveys are designed for many different users, including farmers, ranchers, foresters, agronomists, urban planners, community officials, engineers, developers, builders, and home buyers. Also, conservationists, teachers, students, and specialists in recreation, waste disposal, and pollution control can use the surveys to help them understand, protect, or enhance the environment.

Various land use regulations of Federal, State, and local governments may impose special restrictions on land use or land treatment. Soil surveys identify soil properties that are used in making various land use or land treatment decisions. The information is intended to help the land users identify and reduce the effects of soil limitations on various land uses. The landowner or user is responsible for identifying and complying with existing laws and regulations.

Although soil survey information can be used for general farm, local, and wider area planning, onsite investigation is needed to supplement this information in some cases. Examples include soil quality assessments (http://www.nrcs.usda.gov/wps/portal/nrcs/main/soils/health/) and certain conservation and engineering applications. For more detailed information, contact your local USDA Service Center (https://offices.sc.egov.usda.gov/locator/app?agency=nrcs) or your NRCS State Soil Scientist (http://www.nrcs.usda.gov/wps/portal/nrcs/detail/soils/contactus/?cid=nrcs142p2_053951).

Great differences in soil properties can occur within short distances. Some soils are seasonally wet or subject to flooding. Some are too unstable to be used as a foundation for buildings or roads. Clayey or wet soils are poorly suited to use as septic tank absorption fields. A high water table makes a soil poorly suited to basements or underground installations.

The National Cooperative Soil Survey is a joint effort of the United States Department of Agriculture and other Federal agencies, State agencies including the Agricultural Experiment Stations, and local agencies. The Natural Resources Conservation Service (NRCS) has leadership for the Federal part of the National Cooperative Soil Survey.

Information about soils is updated periodically. Updated information is available through the NRCS Web Soil Survey, the site for official soil survey information.

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How Soil Surveys Are Made

Soil surveys are made to provide information about the soils and miscellaneous areas in a specific area. They include a description of the soils and miscellaneous areas and their location on the landscape and tables that show soil properties and limitations affecting various uses. Soil scientists observed the steepness, length, and shape of the slopes; the general pattern of drainage; the kinds of crops and native plants; and the kinds of bedrock. They observed and described many soil profiles. A soil profile is the sequence of natural layers, or horizons, in a soil. The profile extends from the surface down into the unconsolidated material in which the soil formed or from the surface down to bedrock. The unconsolidated material is devoid of roots and other living organisms and has not been changed by other biological activity.

Currently, soils are mapped according to the boundaries of major land resource areas (MLRAs). MLRAs are geographically associated land resource units that share common characteristics related to physiography, geology, climate, water resources, soils, biological resources, and land uses (USDA, 2006). Soil survey areas typically consist of parts of one or more MLRA.

The soils and miscellaneous areas in a survey area occur in an orderly pattern that is related to the geology, landforms, relief, climate, and natural vegetation of the area. Each kind of soil and miscellaneous area is associated with a particular kind of landform or with a segment of the landform. By observing the soils and miscellaneous areas in the survey area and relating their position to specific segments of the landform, a soil scientist develops a concept, or model, of how they were formed. Thus, during mapping, this model enables the soil scientist to predict with a considerable degree of accuracy the kind of soil or miscellaneous area at a specific location on the landscape.

Commonly, individual soils on the landscape merge into one another as their characteristics gradually change. To construct an accurate soil map, however, soil scientists must determine the boundaries between the soils. They can observe only a limited number of soil profiles. Nevertheless, these observations, supplemented by an understanding of the soil-vegetation-landscape relationship, are sufficient to verify predictions of the kinds of soil in an area and to determine the boundaries.

Soil scientists recorded the characteristics of the soil profiles that they studied. They noted soil color, texture, size and shape of soil aggregates, kind and amount of rock fragments, distribution of plant roots, reaction, and other features that enable them to identify soils. After describing the soils in the survey area and determining their properties, the soil scientists assigned the soils to taxonomic classes (units). Taxonomic classes are concepts. Each taxonomic class has a set of soil characteristics with precisely defined limits. The classes are used as a basis for comparison to classify soils systematically. Soil taxonomy, the system of taxonomic classification used in the United States, is based mainly on the kind and character of soil properties and the arrangement of horizons within the profile. After the soil

scientists classified and named the soils in the survey area, they compared the individual soils with similar soils in the same taxonomic class in other areas so that they could confirm data and assemble additional data based on experience and research.

The objective of soil mapping is not to delineate pure map unit components; the objective is to separate the landscape into landforms or landform segments that have similar use and management requirements. Each map unit is defined by a unique combination of soil components and/or miscellaneous areas in predictable proportions. Some components may be highly contrasting to the other components of the map unit. The presence of minor components in a map unit in no way diminishes the usefulness or accuracy of the data. The delineation of such landforms and landform segments on the map provides sufficient information for the development of resource plans. If intensive use of small areas is planned, onsite investigation is needed to define and locate the soils and miscellaneous areas.

Soil scientists make many field observations in the process of producing a soil map. The frequency of observation is dependent upon several factors, including scale of mapping, intensity of mapping, design of map units, complexity of the landscape, and experience of the soil scientist. Observations are made to test and refine the soil-landscape model and predictions and to verify the classification of the soils at specific locations. Once the soil-landscape model is refined, a significantly smaller number of measurements of individual soil properties are made and recorded. These measurements may include field measurements, such as those for color, depth to bedrock, and texture, and laboratory measurements, such as those for content of sand, silt, clay, salt, and other components. Properties of each soil typically vary from one point to another across the landscape.

Observations for map unit components are aggregated to develop ranges of characteristics for the components. The aggregated values are presented. Direct measurements do not exist for every property presented for every map unit component. Values for some properties are estimated from combinations of other properties.

While a soil survey is in progress, samples of some of the soils in the area generally are collected for laboratory analyses and for engineering tests. Soil scientists interpret the data from these analyses and tests as well as the field-observed characteristics and the soil properties to determine the expected behavior of the soils under different uses. Interpretations for all of the soils are field tested through observation of the soils in different uses and under different levels of management. Some interpretations are modified to fit local conditions, and some new interpretations are developed to meet local needs. Data are assembled from other sources, such as research information, production records, and field experience of specialists. For example, data on crop yields under defined levels of management are assembled from farm records and from field or plot experiments on the same kinds of soil.

Predictions about soil behavior are based not only on soil properties but also on such variables as climate and biological activity. Soil conditions are predictable over long periods of time, but they are not predictable from year to year. For example, soil scientists can predict with a fairly high degree of accuracy that a given soil will have a high water table within certain depths in most years, but they cannot predict that a high water table will always be at a specific level in the soil on a specific date.

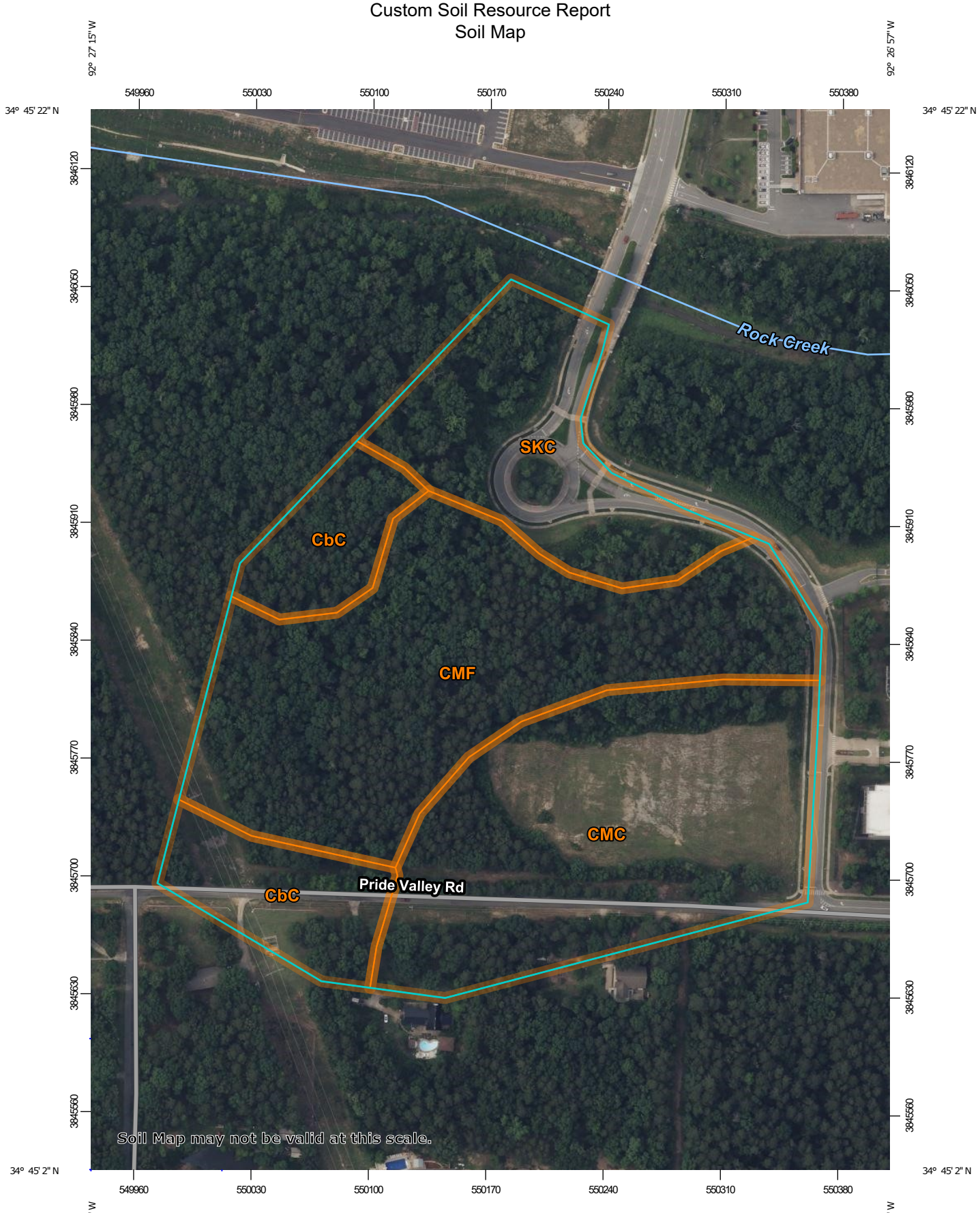
After soil scientists located and identified the significant natural bodies of soil in the survey area, they drew the boundaries of these bodies on aerial photographs and

identified each as a specific map unit. Aerial photographs show trees, buildings, fields, roads, and rivers, all of which help in locating boundaries accurately.

Soil Map

The soil map section includes the soil map for the defined area of interest, a list of soil map units on the map and extent of each map unit, and cartographic symbols displayed on the map. Also presented are various metadata about data used to produce the map, and a description of each soil map unit.

Custom Soil Resource Report
Soil Map



Map Scale: 1:3,070 if printed on A portrait (8.5" x 11") sheet.

N

Meters: 0 45 90 180 270

Feet: 0 100 200 400 600

Map projection: Web Mercator Corner coordinates: WGS84 Edge tics: UTM Zone 15N WGS84

MAP LEGEND

Area of Interest (AOI)
- Area of Interest (AOI)

Soils
- Soil Map Unit Polygons
- Soil Map Unit Lines
- Soil Map Unit Points

Special Point Features
- Blowout
- Borrow Pit
- Clay Spot
- Closed Depression
- Gravel Pit
- Gravelly Spot
- Landfill
- Lava Flow
- Marsh or swamp
- Mine or Quarry
- Miscellaneous Water
- Perennial Water
- Rock Outcrop
- Saline Spot
- Sandy Spot
- Severely Eroded Spot
- Sinkhole
- Slide or Slip
- Sodic Spot
- Spoil Area
- Stony Spot
- Very Stony Spot
- Wet Spot
- Other
- Special Line Features

Water Features
- Streams and Canals

Transportation
- Rails
- Interstate Highways
- US Routes
- Major Roads
- Local Roads

Background
- Aerial Photography

MAP INFORMATION

The soil surveys that comprise your AOI were mapped at 1:20,000.

Warning: Soil Map may not be valid at this scale.

Enlargement of maps beyond the scale of mapping can cause misunderstanding of the detail of mapping and accuracy of soil line placement. The maps do not show the small areas of contrasting soils that could have been shown at a more detailed scale.

Please rely on the bar scale on each map sheet for map measurements.

Source of Map: Natural Resources Conservation Service
Web Soil Survey URL:
Coordinate System: Web Mercator (EPSG:3857)

Maps from the Web Soil Survey are based on the Web Mercator projection, which preserves direction and shape but distorts distance and area. A projection that preserves area, such as the Albers equal-area conic projection, should be used if more accurate calculations of distance or area are required.

This product is generated from the USDA-NRCS certified data as of the version date(s) listed below.

Soil Survey Area: Pulaski County, Arkansas
Survey Area Data: Version 20, Sep 12, 2023

Soil map units are labeled (as space allows) for map scales 1:50,000 or larger.

Date(s) aerial images were photographed: May 1, 2022—May 29, 2022

The orthophoto or other base map on which the soil lines were compiled and digitized probably differs from the background imagery displayed on these maps. As a result, some minor shifting of map unit boundaries may be evident.

Map Unit Legend

Map Unit Symbol	Map Unit Name	Acres in AOI	Percent of AOI
CbC	Carnasaw-Urban land complex, 3 to 8 percent slopes	3.8	13.7%
CMC	Carnasaw-Mountainburg association, undulating	8.9	32.5%
CMF	Carnasaw-Mountainburg association, steep	10.2	37.2%
SKC	Sallisaw-Leadvale association, undulating	4.6	16.7%
Totals for Area of Interest		**27.5**	**100.0%**

Map Unit Descriptions

The map units delineated on the detailed soil maps in a soil survey represent the soils or miscellaneous areas in the survey area. The map unit descriptions, along with the maps, can be used to determine the composition and properties of a unit.

A map unit delineation on a soil map represents an area dominated by one or more major kinds of soil or miscellaneous areas. A map unit is identified and named according to the taxonomic classification of the dominant soils. Within a taxonomic class there are precisely defined limits for the properties of the soils. On the landscape, however, the soils are natural phenomena, and they have the characteristic variability of all natural phenomena. Thus, the range of some observed properties may extend beyond the limits defined for a taxonomic class. Areas of soils of a single taxonomic class rarely, if ever, can be mapped without including areas of other taxonomic classes. Consequently, every map unit is made up of the soils or miscellaneous areas for which it is named and some minor components that belong to taxonomic classes other than those of the major soils.

Most minor soils have properties similar to those of the dominant soil or soils in the map unit, and thus they do not affect use and management. These are called noncontrasting, or similar, components. They may or may not be mentioned in a particular map unit description. Other minor components, however, have properties and behavioral characteristics divergent enough to affect use or to require different management. These are called contrasting, or dissimilar, components. They generally are in small areas and could not be mapped separately because of the scale used. Some small areas of strongly contrasting soils or miscellaneous areas are identified by a special symbol on the maps. If included in the database for a given area, the contrasting minor components are identified in the map unit descriptions along with some characteristics of each. A few areas of minor components may not have been observed, and consequently they are not mentioned in the descriptions, especially where the pattern was so complex that it was impractical to make enough observations to identify all the soils and miscellaneous areas on the landscape.

The presence of minor components in a map unit in no way diminishes the usefulness or accuracy of the data. The objective of mapping is not to delineate pure taxonomic classes but rather to separate the landscape into landforms or landform segments that have similar use and management requirements. The delineation of such segments on the map provides sufficient information for the development of resource plans. If intensive use of small areas is planned, however, onsite investigation is needed to define and locate the soils and miscellaneous areas.

An identifying symbol precedes the map unit name in the map unit descriptions. Each description includes general facts about the unit and gives important soil properties and qualities.

Soils that have profiles that are almost alike make up a *soil series*. Except for differences in texture of the surface layer, all the soils of a series have major horizons that are similar in composition, thickness, and arrangement.

Soils of one series can differ in texture of the surface layer, slope, stoniness, salinity, degree of erosion, and other characteristics that affect their use. On the basis of such differences, a soil series is divided into *soil phases*. Most of the areas shown on the detailed soil maps are phases of soil series. The name of a soil phase commonly indicates a feature that affects use or management. For example, Alpha silt loam, 0 to 2 percent slopes, is a phase of the Alpha series.

Some map units are made up of two or more major soils or miscellaneous areas. These map units are complexes, associations, or undifferentiated groups.

A *complex* consists of two or more soils or miscellaneous areas in such an intricate pattern or in such small areas that they cannot be shown separately on the maps. The pattern and proportion of the soils or miscellaneous areas are somewhat similar in all areas. Alpha-Beta complex, 0 to 6 percent slopes, is an example.

An *association* is made up of two or more geographically associated soils or miscellaneous areas that are shown as one unit on the maps. Because of present or anticipated uses of the map units in the survey area, it was not considered practical or necessary to map the soils or miscellaneous areas separately. The pattern and relative proportion of the soils or miscellaneous areas are somewhat similar. Alpha-Beta association, 0 to 2 percent slopes, is an example.

An *undifferentiated group* is made up of two or more soils or miscellaneous areas that could be mapped individually but are mapped as one unit because similar interpretations can be made for use and management. The pattern and proportion of the soils or miscellaneous areas in a mapped area are not uniform. An area can be made up of only one of the major soils or miscellaneous areas, or it can be made up of all of them. Alpha and Beta soils, 0 to 2 percent slopes, is an example.

Some surveys include *miscellaneous areas*. Such areas have little or no soil material and support little or no vegetation. Rock outcrop is an example.

Pulaski County, Arkansas

CbC—Carnasaw-Urban land complex, 3 to 8 percent slopes

Map Unit Setting

National map unit symbol: m04n
Elevation: 500 to 1,800 feet
Mean annual precipitation: 43 to 58 inches
Mean annual air temperature: 50 to 72 degrees F
Frost-free period: 200 to 260 days
Farmland classification: Not prime farmland

Map Unit Composition

Carnasaw and similar soils: 51 percent
Urban land: 49 percent
Estimates are based on observations, descriptions, and transects of the mapunit.

Description of Carnasaw

Setting

Landform: Hills
Landform position (three-dimensional): Crest
Down-slope shape: Linear
Across-slope shape: Convex
Parent material: Clayey residuum weathered from shale

Typical profile

A - 0 to 2 inches: gravelly silt loam
E - 2 to 6 inches: gravelly silt loam
Bt1 - 6 to 38 inches: silty clay
Bt2 - 38 to 49 inches: silty clay loam
Cr - 49 to 52 inches: bedrock

Properties and qualities

Slope: 3 to 8 percent
Depth to restrictive feature: 40 to 60 inches to paralithic bedrock
Drainage class: Well drained
Runoff class: Very high
Capacity of the most limiting layer to transmit water (Ksat): Moderately low to moderately high (0.06 to 0.20 in/hr)
Depth to water table: More than 80 inches
Frequency of flooding: None
Frequency of ponding: None
Available water supply, 0 to 60 inches: Moderate (about 6.0 inches)

Interpretive groups

Land capability classification (irrigated): None specified
Land capability classification (nonirrigated): 4e
Hydrologic Soil Group: C
Ecological site: F119XY006AR - Clayey Upland
Hydric soil rating: No

Description of Urban Land

Interpretive groups

Land capability classification (irrigated): None specified

Land capability classification (nonirrigated): 8
Hydrologic Soil Group: D
Hydric soil rating: No

CMC—Carnasaw-Mountainburg association, undulating

Map Unit Setting

National map unit symbol: m04j
Elevation: 500 to 2,800 feet
Mean annual precipitation: 43 to 58 inches
Mean annual air temperature: 50 to 72 degrees F
Frost-free period: 200 to 260 days
Farmland classification: Not prime farmland

Map Unit Composition

Carnasaw and similar soils: 65 percent
Mountainburg and similar soils: 25 percent
Minor components: 10 percent
Estimates are based on observations, descriptions, and transects of the mapunit.

Description of Carnasaw

Setting

Landform: Mountains
Landform position (three-dimensional): Mountaintop
Down-slope shape: Convex
Across-slope shape: Convex
Parent material: Clayey residuum weathered from shale

Typical profile

A - 0 to 2 inches: gravelly silt loam
E - 2 to 6 inches: gravelly silt loam
Bt1 - 6 to 38 inches: silty clay
Bt2 - 38 to 49 inches: silty clay loam
Cr - 49 to 52 inches: bedrock

Properties and qualities

Slope: 3 to 12 percent
Depth to restrictive feature: 40 to 60 inches to paralithic bedrock
Drainage class: Well drained
Runoff class: Very high
Capacity of the most limiting layer to transmit water (Ksat): Moderately low to moderately high (0.06 to 0.20 in/hr)
Depth to water table: More than 80 inches
Frequency of flooding: None
Frequency of ponding: None
Available water supply, 0 to 60 inches: Moderate (about 6.0 inches)

Interpretive groups

Land capability classification (irrigated): None specified
Land capability classification (nonirrigated): 4e

Hydrologic Soil Group: C
Ecological site: F119XY006AR - Clayey Upland
Hydric soil rating: No

Description of Mountainburg

Setting

Landform: Mountains
Landform position (three-dimensional): Mountaintop
Down-slope shape: Convex
Across-slope shape: Convex
Parent material: Stony, loamy residuum weathered from sandstone

Typical profile

A - 0 to 1 inches: stony fine sandy loam
E - 1 to 6 inches: stony sandy loam
Bt - 6 to 15 inches: very gravelly fine sandy loam
R - 15 to 20 inches: bedrock

Properties and qualities

Slope: 3 to 12 percent
Depth to restrictive feature: 12 to 20 inches to lithic bedrock
Drainage class: Well drained
Runoff class: Very high
Capacity of the most limiting layer to transmit water (Ksat): Moderately low to moderately high (0.06 to 0.20 in/hr)
Depth to water table: More than 80 inches
Frequency of flooding: None
Frequency of ponding: None
Available water supply, 0 to 60 inches: Very low (about 1.2 inches)

Interpretive groups

Land capability classification (irrigated): None specified
Land capability classification (nonirrigated): 6s
Hydrologic Soil Group: D
Ecological site: F119XY003AR - Shallow Upland
Hydric soil rating: No

Minor Components

Leadvale

Percent of map unit: 10 percent
Ecological site: F119XY007AR - Loamy Upland
Hydric soil rating: No

CMF—Carnasaw-Mountainburg association, steep

Map Unit Setting

National map unit symbol: m04k
Elevation: 500 to 2,800 feet

Mean annual precipitation: 43 to 58 inches
Mean annual air temperature: 50 to 72 degrees F
Frost-free period: 200 to 260 days
Farmland classification: Not prime farmland

Map Unit Composition

Carnasaw and similar soils: 65 percent
Mountainburg and similar soils: 20 percent
Rock outcrop: 15 percent
Estimates are based on observations, descriptions, and transects of the mapunit.

Description of Carnasaw

Setting

Landform: Mountains
Landform position (three-dimensional): Mountainflank
Down-slope shape: Convex
Across-slope shape: Linear
Parent material: Clayey residuum weathered from shale

Typical profile

A - 0 to 2 inches: gravelly silt loam
E - 2 to 6 inches: gravelly silt loam
Bt1 - 6 to 38 inches: silty clay
Bt2 - 38 to 49 inches: silty clay loam
Cr - 49 to 52 inches: bedrock

Properties and qualities

Slope: 12 to 40 percent
Depth to restrictive feature: 40 to 60 inches to paralithic bedrock
Drainage class: Well drained
Runoff class: Very high
Capacity of the most limiting layer to transmit water (Ksat): Moderately low to moderately high (0.06 to 0.20 in/hr)
Depth to water table: More than 80 inches
Frequency of flooding: None
Frequency of ponding: None
Available water supply, 0 to 60 inches: Moderate (about 6.0 inches)

Interpretive groups

Land capability classification (irrigated): None specified
Land capability classification (nonirrigated): 7e
Hydrologic Soil Group: C
Ecological site: F119XY006AR - Clayey Upland
Hydric soil rating: No

Description of Mountainburg

Setting

Landform: Mountains
Landform position (three-dimensional): Mountainflank
Down-slope shape: Convex
Across-slope shape: Linear
Parent material: Stony, loamy residuum weathered from sandstone

Typical profile

A - 0 to 1 inches: stony fine sandy loam

E - 1 to 6 inches: stony sandy loam
Bt - 6 to 15 inches: very gravelly fine sandy loam
R - 15 to 20 inches: bedrock

Properties and qualities

Slope: 12 to 40 percent
Depth to restrictive feature: 12 to 20 inches to lithic bedrock
Drainage class: Well drained
Runoff class: Very high
Capacity of the most limiting layer to transmit water (Ksat): Moderately low to moderately high (0.06 to 0.20 in/hr)
Depth to water table: More than 80 inches
Frequency of flooding: None
Frequency of ponding: None
Available water supply, 0 to 60 inches: Very low (about 1.2 inches)

Interpretive groups

Land capability classification (irrigated): None specified
Land capability classification (nonirrigated): 7s
Hydrologic Soil Group: D
Ecological site: F119XY003AR - Shallow Upland
Hydric soil rating: No

Description of Rock Outcrop

Interpretive groups

Land capability classification (irrigated): None specified
Land capability classification (nonirrigated): 8
Hydrologic Soil Group: D
Hydric soil rating: No

SKC—Sallisaw-Leadvale association, undulating

Map Unit Setting

National map unit symbol: m05k
Elevation: 250 to 2,300 feet
Mean annual precipitation: 43 to 58 inches
Mean annual air temperature: 50 to 72 degrees F
Frost-free period: 200 to 260 days
Farmland classification: Farmland of statewide importance

Map Unit Composition

Sallisaw and similar soils: 55 percent
Leadvale and similar soils: 45 percent
Estimates are based on observations, descriptions, and transects of the mapunit.

Description of Sallisaw

Setting

Landform: Stream terraces

Landform position (three-dimensional): Riser
Down-slope shape: Convex
Across-slope shape: Linear
Parent material: Loamy and gravelly alluvium

Typical profile

Ap - 0 to 7 inches: silt loam
Bt - 7 to 27 inches: silt loam
2BCt - 27 to 72 inches: very gravelly silt loam

Properties and qualities

Slope: 2 to 8 percent
Depth to restrictive feature: More than 80 inches
Drainage class: Well drained
Runoff class: Medium
Capacity of the most limiting layer to transmit water (Ksat): Moderately high to high (0.57 to 1.98 in/hr)
Depth to water table: More than 80 inches
Frequency of flooding: None
Frequency of ponding: None
Available water supply, 0 to 60 inches: Moderate (about 6.7 inches)

Interpretive groups

Land capability classification (irrigated): None specified
Land capability classification (nonirrigated): 3e
Hydrologic Soil Group: B
Ecological site: F119XY007AR - Loamy Upland
Hydric soil rating: No

Description of Leadvale

Setting

Landform: Valleys
Down-slope shape: Convex
Across-slope shape: Linear
Parent material: Loamy pedisediment

Typical profile

Ap - 0 to 7 inches: silt loam
Bt - 7 to 16 inches: silt loam
Btx1 - 16 to 48 inches: silt loam
Btx2 - 48 to 72 inches: silty clay loam

Properties and qualities

Slope: 2 to 8 percent
Depth to restrictive feature: 12 to 20 inches to fragipan; 48 to 60 inches to paralithic bedrock
Drainage class: Moderately well drained
Runoff class: High
Capacity of the most limiting layer to transmit water (Ksat): Moderately low to moderately high (0.06 to 0.57 in/hr)
Depth to water table: About 16 to 30 inches
Frequency of flooding: None
Frequency of ponding: None
Available water supply, 0 to 60 inches: Low (about 3.1 inches)

Interpretive groups

Land capability classification (irrigated): None specified

Land capability classification (nonirrigated): 3e
Hydrologic Soil Group: D
Ecological site: F119XY007AR - Loamy Upland
Hydric soil rating: No

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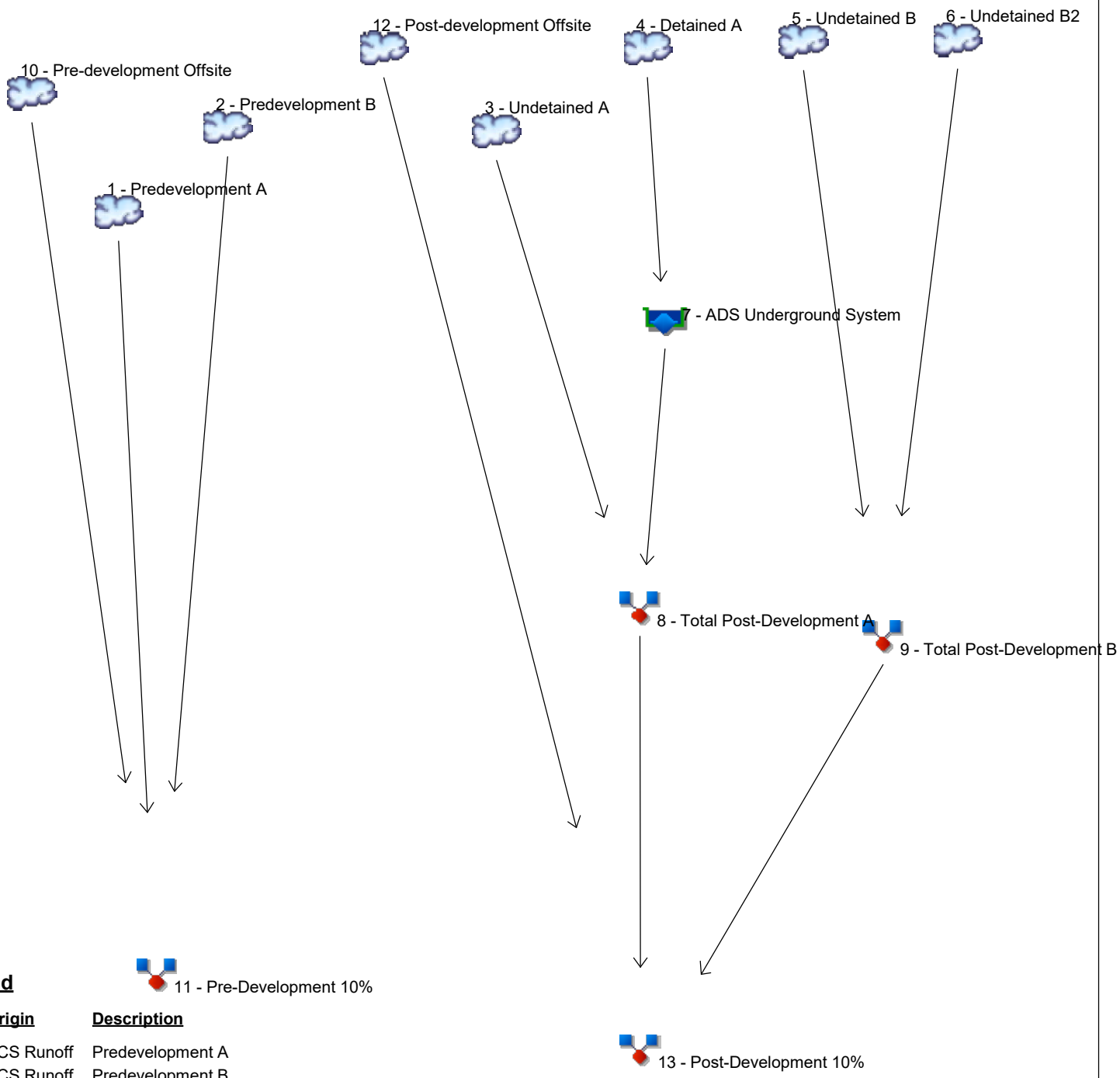
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APPENDIX
Hydrologic Calculations

Watershed Model Schematic



Legend

Hyd.	Origin	Description
1	SCS Runoff	Predevelopment A
2	SCS Runoff	Predevelopment B
3	SCS Runoff	Undetained A
4	SCS Runoff	Detained A
5	SCS Runoff	Undetained B
6	SCS Runoff	Undetained B2
7	Reservoir	ADS Underground System
8	Combine	Total Post-Development A
9	Combine	Total Post-Development B
10	SCS Runoff	Pre-development Offsite
11	Combine	Pre-Development 10%
12	SCS Runoff	Post-development Offsite
13	Combine	Post-Development 10%

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Hydrograph Summary Report

Hydraflow Hydrographs Extension for Autodesk® Civil 3D® by Autodesk, Inc. v2022

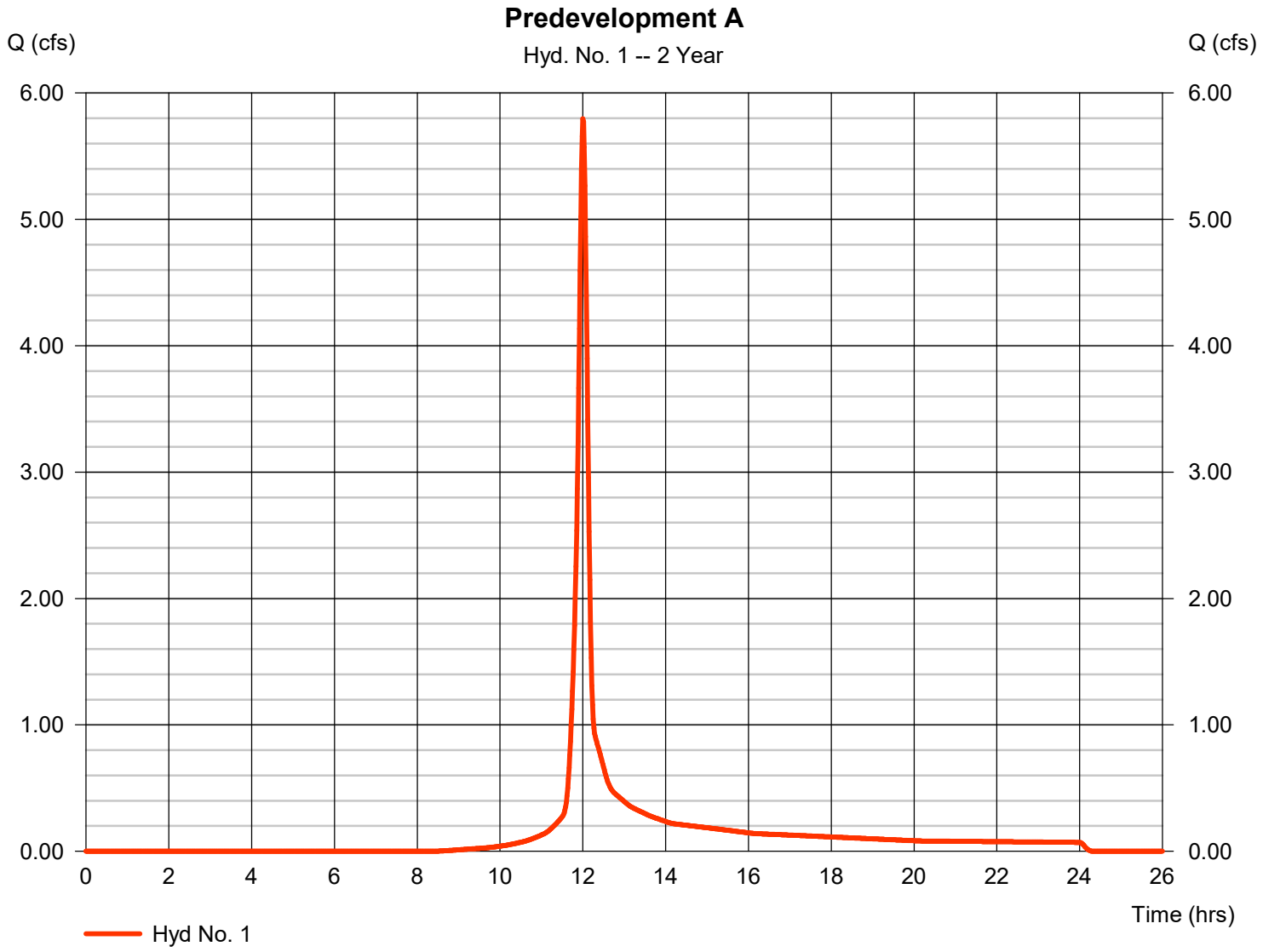
Hyd. No.	Hydrograph type (origin)	Peak flow (cfs)	Time interval (min)	Time to Peak (min)	Hyd. volume (cuft)	Inflow hyd(s)	Maximum elevation (ft)	Total strge used (cuft)	Hydrograph Description
1	SCS Runoff	5.795	1	720	14,013	------	------	------	Predevelopment A
2	SCS Runoff	2.992	1	720	7,257	------	------	------	Predevelopment B
3	SCS Runoff	0.998	1	717	1,905	------	------	------	Undetained A
4	SCS Runoff	7.835	1	720	19,699	------	------	------	Detained A
5	SCS Runoff	2.422	1	719	5,518	------	------	------	Undetained B
6	SCS Runoff	0.340	1	717	741	------	------	------	Undetained B2
7	Reservoir	2.355	1	731	19,698	4	1230.51	5,373	ADS Underground System
8	Combine	3.026	1	717	21,603	3, 7	------	------	Total Post-Development A
9	Combine	2.738	1	719	6,260	5, 6,	------	------	Total Post-Development B
10	SCS Runoff	38.09	1	723	104,199	------	------	------	Pre-development Offsite
11	Combine	46.49	1	722	125,469	1, 2, 10	------	------	Pre-Development 10%
12	SCS Runoff	38.09	1	723	104,199	------	------	------	Post-development Offsite
13	Combine	42.99	1	722	132,062	8, 9, 12	------	------	Post-Development 10%

Hydrograph Report

Hyd. No. 1

Predevelopment A

Hydrograph type	=	SCS Runoff	Peak discharge	=	5.795 cfs
Storm frequency	=	2 yrs	Time to peak	=	12.00 hrs
Time interval	=	1 min	Hyd. volume	=	14,013 cuft
Drainage area	=	1.790 ac	Curve number	=	79
Basin Slope	=	2.0 %	Hydraulic length	=	130 ft
Tc method	=	TR55	Time of conc. (Tc)	=	11.00 min
Total precip.	=	4.19 in	Distribution	=	Type II
Storm duration	=	24 hrs	Shape factor	=	484



TR55 Tc Worksheet

Hyd. No. 1

Predevelopment A

Description	A		B		C		Totals
Sheet Flow							
Manning's n-value	= 0.150		0.011		0.011		
Flow length (ft)	= 100.0		0.0		0.0		
Two-year 24-hr precip. (in)	= 4.19		0.00		0.00		
Land slope (%)	= 2.20		0.00		0.00		
Travel Time (min)	**= 8.24**	**+**	**0.00**	**+**	**0.00**	**=**	**8.24**
Shallow Concentrated Flow							
Flow length (ft)	= 600.00		0.00		0.00		
Watercourse slope (%)	= 5.70		0.00		0.00		
Surface description	= Unpaved		Paved		Paved		
Average velocity (ft/s)	=3.85		0.00		0.00		
Travel Time (min)	**= 2.60**	**+**	**0.00**	**+**	**0.00**	**=**	**2.60**
Channel Flow							
X sectional flow area (sqft)	= 3.40		0.00		0.00		
Wetted perimeter (ft)	= 14.50		0.00		0.00		
Channel slope (%)	= 0.50		0.00		0.00		
Manning's n-value	= 0.013		0.015		0.015		
Velocity (ft/s)	=3.07						
			0.00				
					0.00		
Flow length (ft)	({0})25.0		0.0		0.0		
Travel Time (min)	**= 0.14**	**+**	**0.00**	**+**	**0.00**	**=**	**0.14**

Total Travel Time, Tc ..... 11.00 min

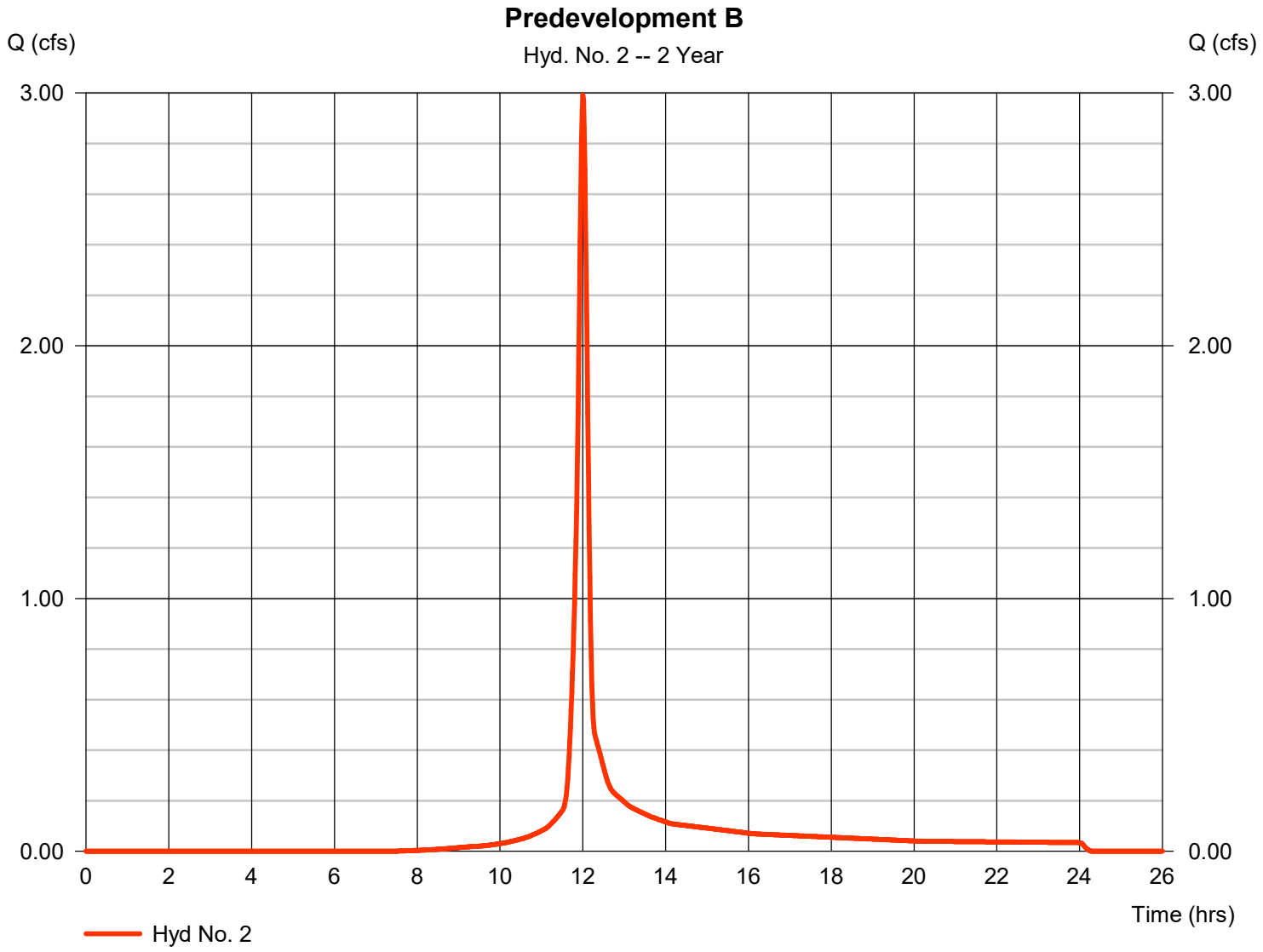
Hydrograph Report

Hyd. No. 2

Predevelopment B

Hydrograph type	=	SCS Runoff	Peak discharge	=	2.992 cfs
Storm frequency	=	2 yrs	Time to peak	=	12.00 hrs
Time interval	=	1 min	Hyd. volume	=	7,257 cuft
Drainage area	=	0.830 ac	Curve number	=	82*
Basin Slope	=	0.0 %	Hydraulic length	=	0 ft
Tc method	=	TR55	Time of conc. (Tc)	=	10.60 min
Total precip.	=	4.19 in	Distribution	=	Type II
Storm duration	=	24 hrs	Shape factor	=	484

* Composite (Area/CN) = [(0.120 x 98) + (0.710 x 79)] / 0.830



TR55 Tc Worksheet

Hydraflow Hydrographs Extension for Autodesk® Civil 3D® by Autodesk, Inc. v2022

Hyd. No. 2

Predevelopment B

Description	A		B		C		Totals
Sheet Flow							
Manning's n-value	= 0.150		0.011		0.011		
Flow length (ft)	= 100.0		0.0		0.0		
Two-year 24-hr precip. (in)	= 4.19		0.00		0.00		
Land slope (%)	= 2.20		0.00		0.00		
Travel Time (min)	**= 8.24**	**+**	**0.00**	**+**	**0.00**	**=**	**8.24**
Shallow Concentrated Flow							
Flow length (ft)	= 170.00		62.00		0.00		
Watercourse slope (%)	= 4.00		26.00		0.00		
Surface description	= Unpaved		Unpaved		Paved		
Average velocity (ft/s)	=3.23		8.23		0.00		
Travel Time (min)	**= 0.88**	**+**	**0.13**	**+**	**0.00**	**=**	**1.00**
Channel Flow							
X sectional flow area (sqft)	= 3.00		1.77		3.14		
Wetted perimeter (ft)	= 12.00		4.71		6.28		
Channel slope (%)	= 3.20		4.20		1.20		
Manning's n-value	= 0.030		0.013		0.012		
Velocity (ft/s)	=3.51						
			12.19				
					8.55		
Flow length (ft)	({0})230.0		147.0		41.0		
Travel Time (min)	**= 1.09**	**+**	**0.20**	**+**	**0.08**	**=**	**1.37**

Total Travel Time, Tc ..... 10.60 min

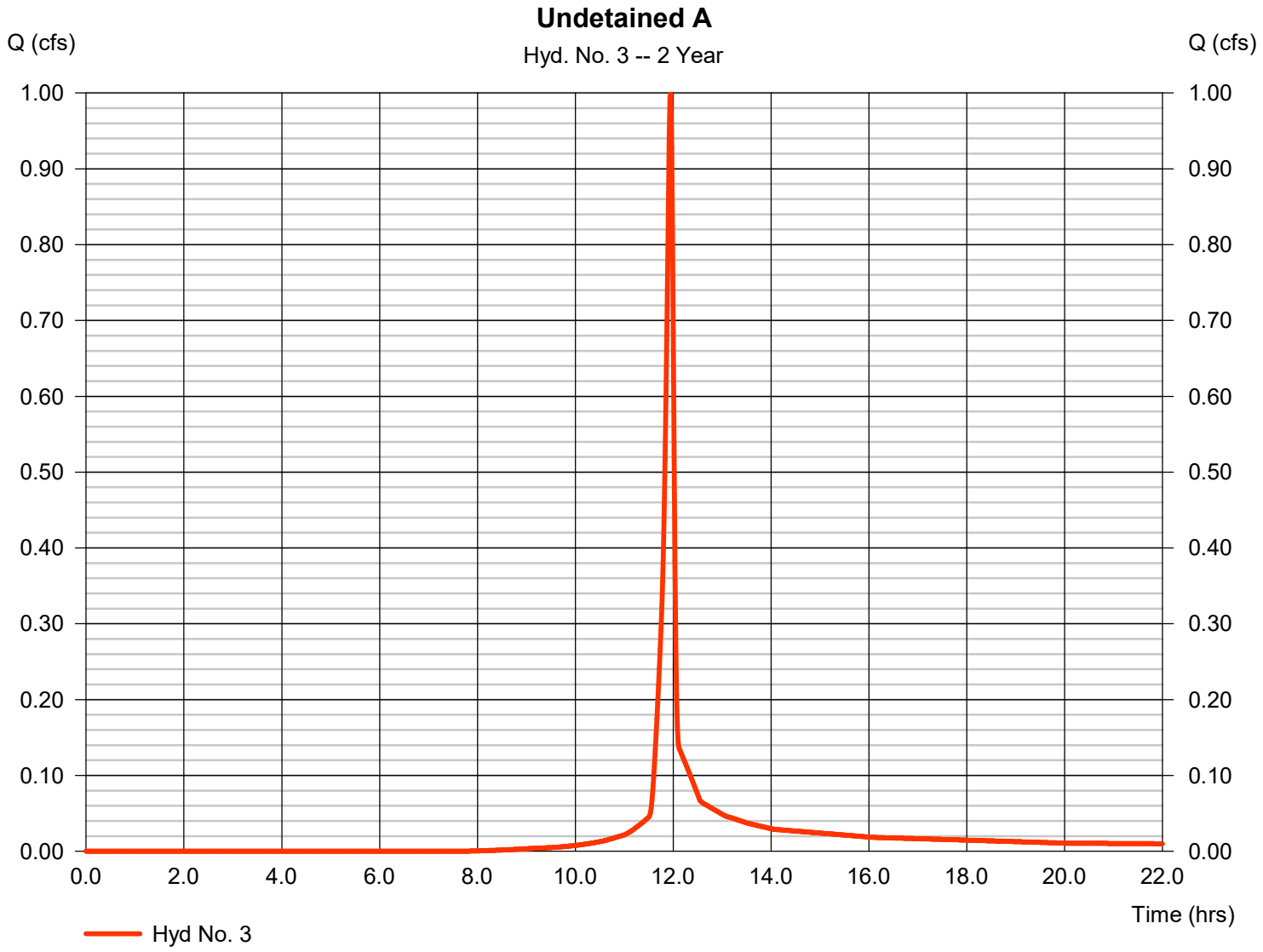
Hydrograph Report

Hyd. No. 3

Undetained A

Hydrograph type	=	SCS Runoff	Peak discharge	=	0.998 cfs
Storm frequency	=	2 yrs	Time to peak	=	11.95 hrs
Time interval	=	1 min	Hyd. volume	=	1,905 cuft
Drainage area	=	0.230 ac	Curve number	=	81*
Basin Slope	=	0.0 %	Hydraulic length	=	0 ft
Tc method	=	TR55	Time of conc. (Tc)	=	4.40 min
Total precip.	=	4.19 in	Distribution	=	Type II
Storm duration	=	24 hrs	Shape factor	=	484

* Composite (Area/CN) = [(0.200 x 79) + (0.030 x 98)] / 0.230



TR55 Tc Worksheet

Hyd. No. 3

Undetained A

Description	A		B		C		Totals
Sheet Flow							
Manning's n-value	= 0.150		0.011		0.011		
Flow length (ft)	= 37.0		0.0		0.0		
Two-year 24-hr precip. (in)	= 4.19		0.00		0.00		
Land slope (%)	= 2.20		0.00		0.00		
Travel Time (min)	**= 3.72**	**+**	**0.00**	**+**	**0.00**	**=**	**3.72**
Shallow Concentrated Flow							
Flow length (ft)	= 63.00		20.00		0.00		
Watercourse slope (%)	= 4.00		26.50		0.00		
Surface description	= Unpaved		Unpaved		Paved		
Average velocity (ft/s)	=3.23		8.31		0.00		
Travel Time (min)	**= 0.33**	**+**	**0.04**	**+**	**0.00**	**=**	**0.37**
Channel Flow							
X sectional flow area (sqft)	= 1.50		1.77		3.14		
Wetted perimeter (ft)	= 6.10		4.71		6.28		
Channel slope (%)	= 3.20		3.60		1.20		
Manning's n-value	= 0.030		0.013		0.012		
Velocity (ft/s)	=3.47						
			11.29				
					8.55		
Flow length (ft)	({0})18.5		123.0		15.0		
Travel Time (min)	**= 0.09**	**+**	**0.18**	**+**	**0.03**	**=**	**0.30**

Total Travel Time, Tc ..... 4.40 min

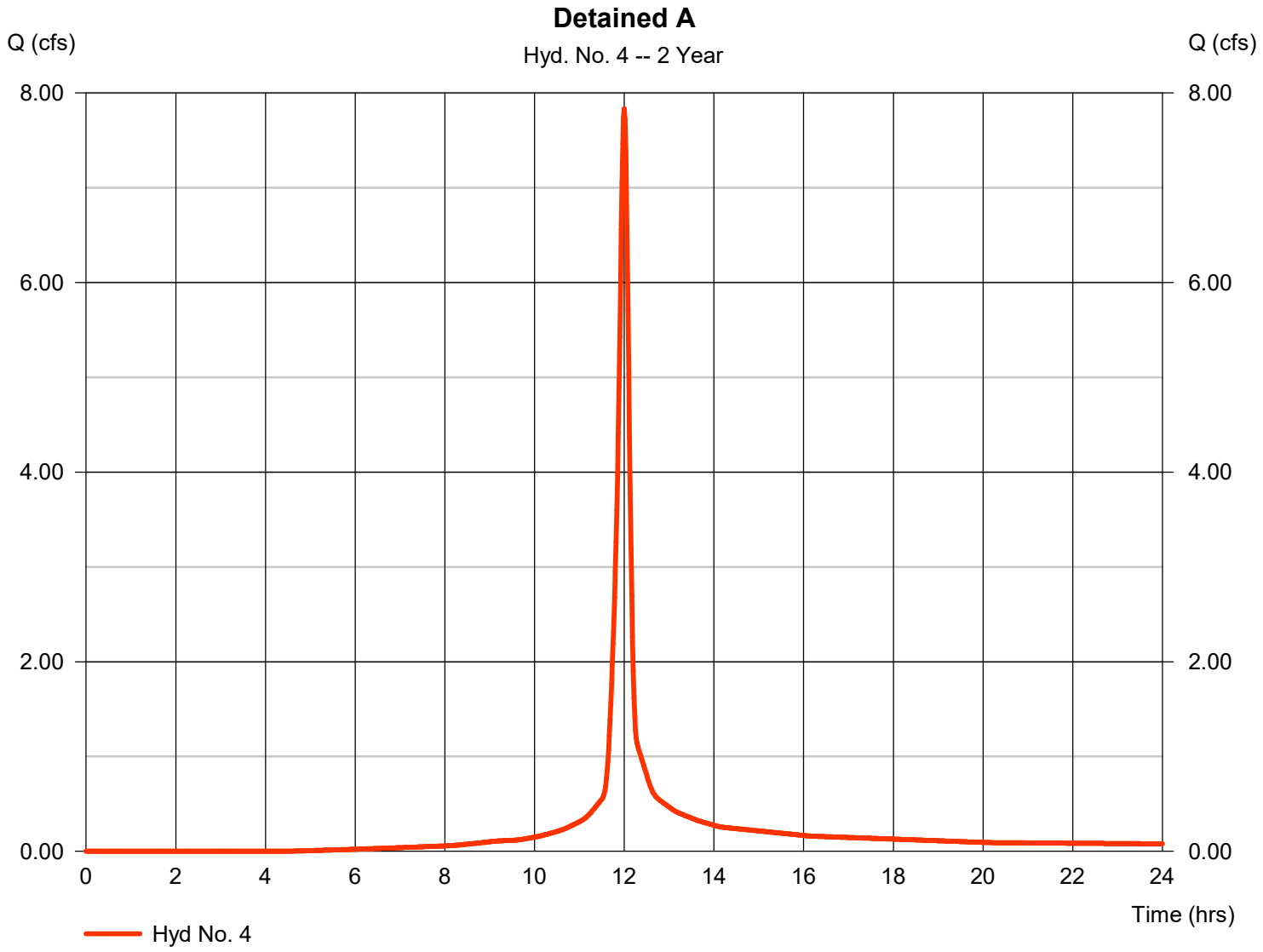
Hydrograph Report

Hyd. No. 4

Detained A

Hydrograph type	=	SCS Runoff	Peak discharge	=	7.835 cfs
Storm frequency	=	2 yrs	Time to peak	=	12.00 hrs
Time interval	=	1 min	Hyd. volume	=	19,699 cuft
Drainage area	=	1.720 ac	Curve number	=	90*
Basin Slope	=	0.0 %	Hydraulic length	=	0 ft
Tc method	=	TR55	Time of conc. (Tc)	=	10.40 min
Total precip.	=	4.19 in	Distribution	=	Type II
Storm duration	=	24 hrs	Shape factor	=	484

* Composite (Area/CN) = [(0.970 x 98) + (0.750 x 79)] / 1.720



TR55 Tc Worksheet

Hyd. No. 4

Detained A

Description	A		B		C		Totals
Sheet Flow							
Manning's n-value	= 0.150		0.011		0.011		
Flow length (ft)	= 100.0		0.0		0.0		
Two-year 24-hr precip. (in)	= 4.19		0.00		0.00		
Land slope (%)	= 2.20		0.00		0.00		
Travel Time (min)	**= 8.24**	**+**	**0.00**	**+**	**0.00**	**=**	**8.24**
Shallow Concentrated Flow							
Flow length (ft)	= 238.00		12.00		104.00		
Watercourse slope (%)	= 4.60		33.00		1.50		
Surface description	= Unpaved		Unpaved		Paved		
Average velocity (ft/s)	=3.46		9.27		2.49		
Travel Time (min)	**= 1.15**	**+**	**0.02**	**+**	**0.70**	**=**	**1.86**
Channel Flow							
X sectional flow area (sqft)	= 1.77		1.77		0.00		
Wetted perimeter (ft)	= 4.71		4.71		0.00		
Channel slope (%)	= 16.00		1.70		0.00		
Manning's n-value	= 0.013		0.013		0.015		
Velocity (ft/s)	=23.80						
			7.76				
					0.00		
Flow length (ft)	({0})61.0		122.0		0.0		
Travel Time (min)	**= 0.04**	**+**	**0.26**	**+**	**0.00**	**=**	**0.30**

Total Travel Time, Tc ..... 10.40 min

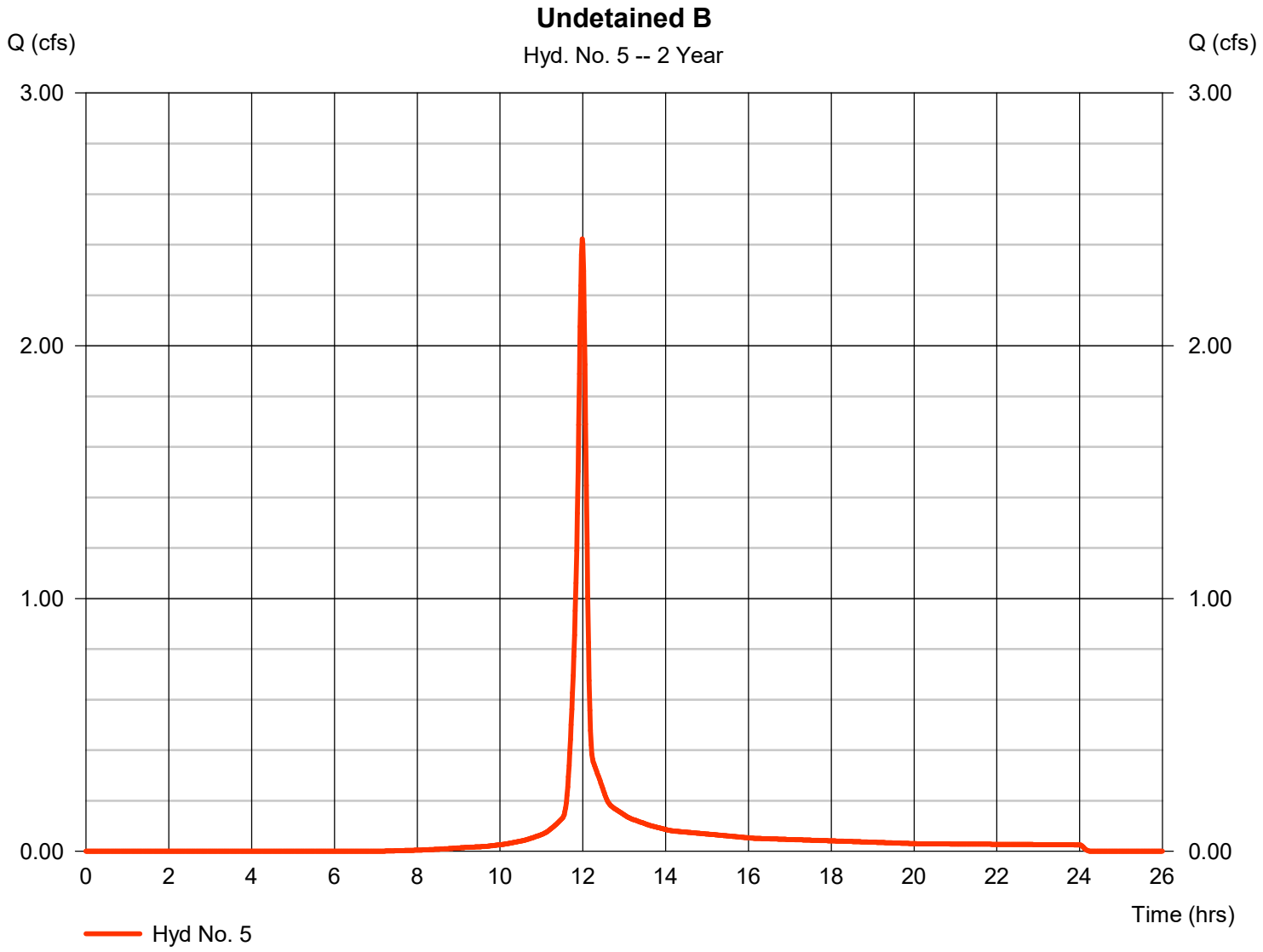
Hydrograph Report

Hyd. No. 5

Undetained B

Hydrograph type	=	SCS Runoff	Peak discharge	=	2.422 cfs
Storm frequency	=	2 yrs	Time to peak	=	11.98 hrs
Time interval	=	1 min	Hyd. volume	=	5,518 cuft
Drainage area	=	0.620 ac	Curve number	=	83*
Basin Slope	=	0.0 %	Hydraulic length	=	0 ft
Tc method	=	TR55	Time of conc. (Tc)	=	10.00 min
Total precip.	=	4.19 in	Distribution	=	Type II
Storm duration	=	24 hrs	Shape factor	=	484

* Composite (Area/CN) = [(0.130 x 98) + (0.490 x 79)] / 0.620



TR55 Tc Worksheet

Hyd. No. 5

Undetained B

Description	A		B		C		Totals
Sheet Flow							
Manning's n-value	= 0.150		0.011		0.011		
Flow length (ft)	= 100.0		0.0		0.0		
Two-year 24-hr precip. (in)	= 4.19		0.00		0.00		
Land slope (%)	= 2.20		0.00		0.00		
Travel Time (min)	**= 8.24**	**+**	**0.00**	**+**	**0.00**	**=**	**8.24**
Shallow Concentrated Flow							
Flow length (ft)	= 170.00		55.00		0.00		
Watercourse slope (%)	= 4.00		26.50		0.00		
Surface description	= Unpaved		Unpaved		Paved		
Average velocity (ft/s)	=3.23		8.31		0.00		
Travel Time (min)	**= 0.88**	**+**	**0.11**	**+**	**0.00**	**=**	**0.99**
Channel Flow							
X sectional flow area (sqft)	= 1.50		1.77		3.14		
Wetted perimeter (ft)	= 6.10		4.71		6.28		
Channel slope (%)	= 3.20		3.60		1.20		
Manning's n-value	= 0.030		0.013		0.012		
Velocity (ft/s)	=3.47						
			11.29				
					8.55		
Flow length (ft)	({0})50.0		333.0		41.0		
Travel Time (min)	**= 0.24**	**+**	**0.49**	**+**	**0.08**	**=**	**0.81**

Total Travel Time, Tc ..... 10.00 min

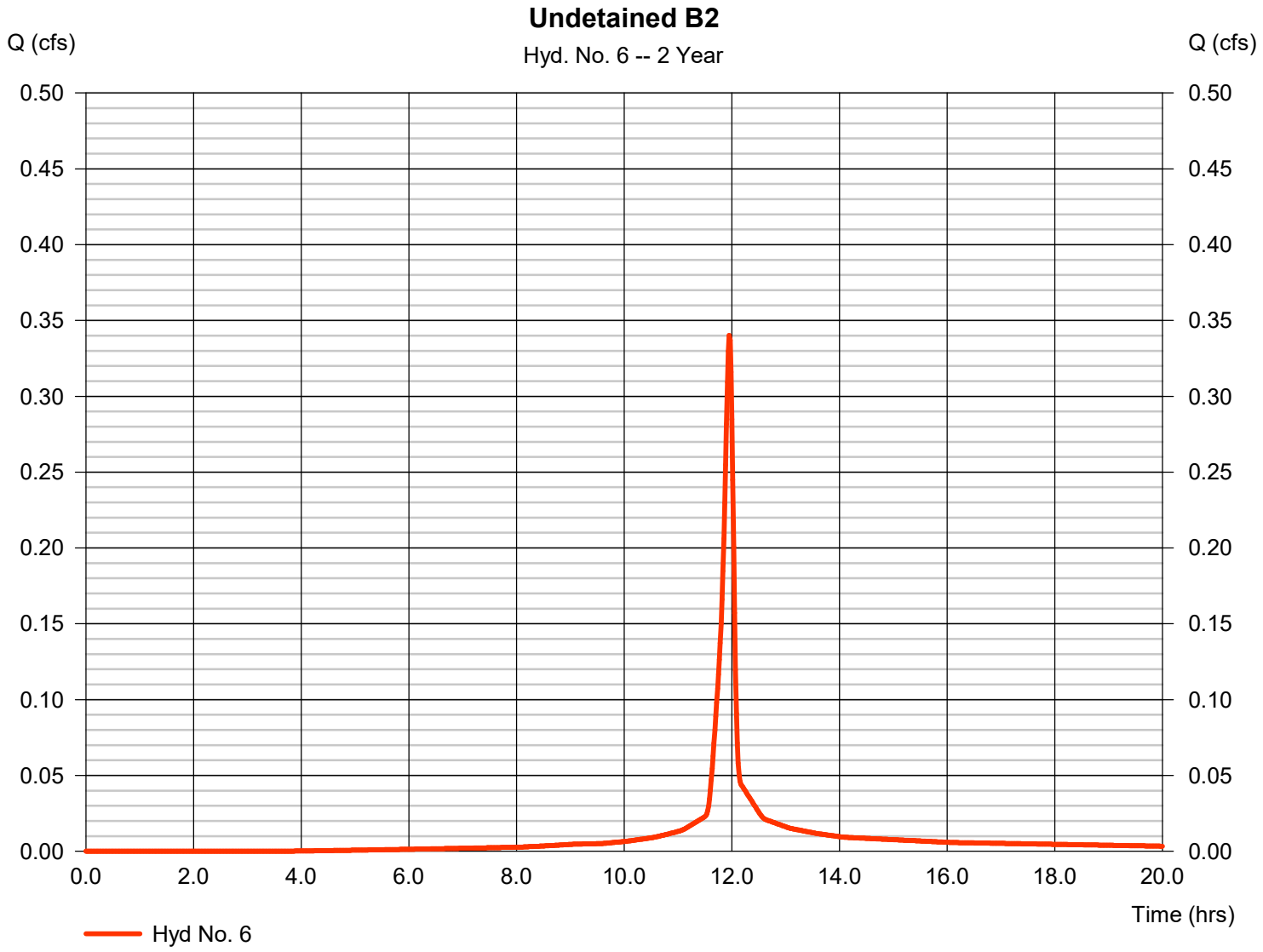
Hydrograph Report

Hyd. No. 6

Undetained B2

Hydrograph type	=	SCS Runoff	Peak discharge	=	0.340 cfs
Storm frequency	=	2 yrs	Time to peak	=	11.95 hrs
Time interval	=	1 min	Hyd. volume	=	741 cuft
Drainage area	=	0.060 ac	Curve number	=	92*
Basin Slope	=	0.0 %	Hydraulic length	=	0 ft
Tc method	=	User	Time of conc. (Tc)	=	5.00 min
Total precip.	=	4.19 in	Distribution	=	Type II
Storm duration	=	24 hrs	Shape factor	=	484

* Composite (Area/CN) = [(0.020 x 79) + (0.040 x 98)] / 0.060



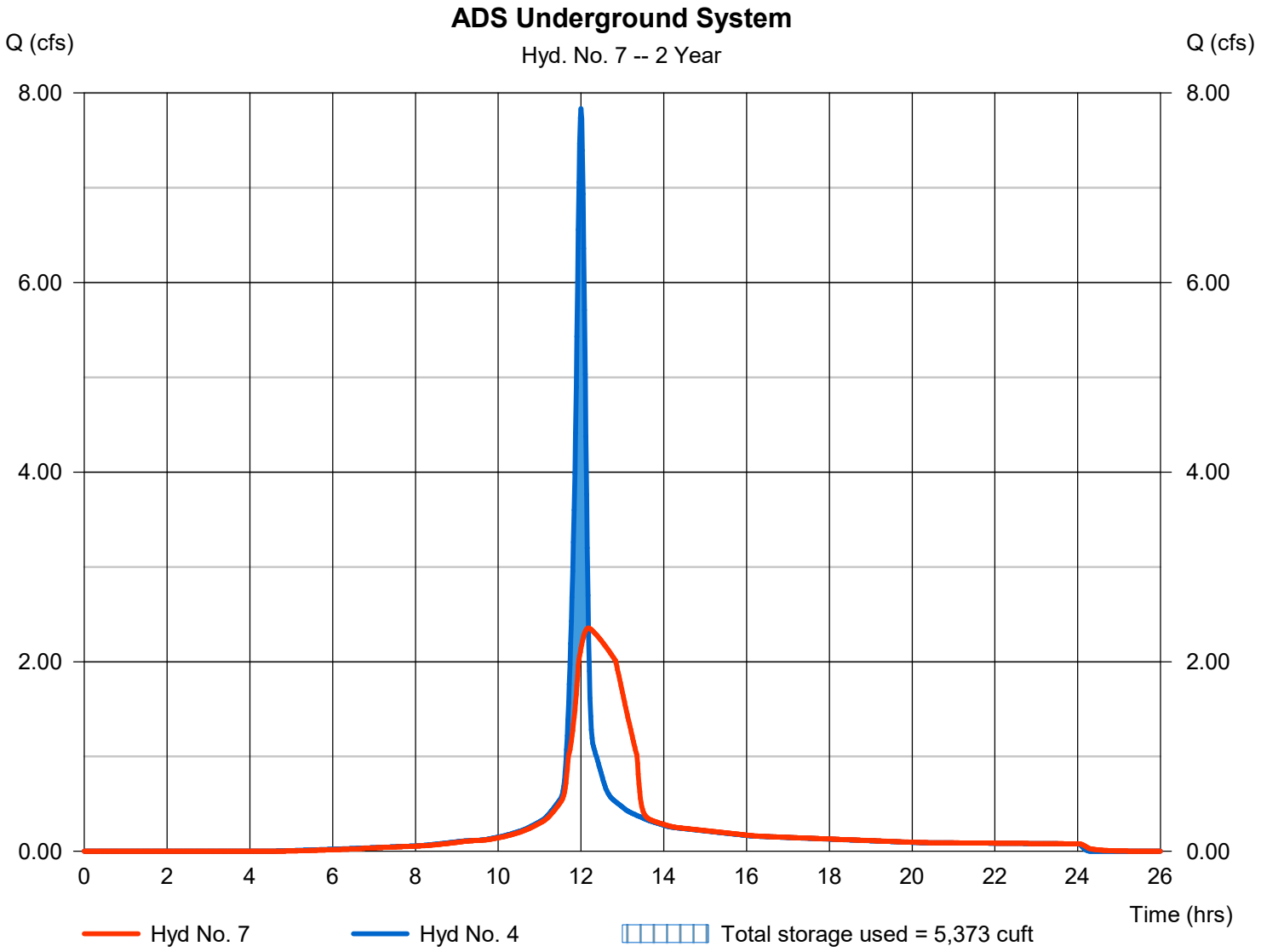
Hydrograph Report

Hyd. No. 7

ADS Underground System

Hydrograph type	=	Reservoir	Peak discharge	=	2.355 cfs
Storm frequency	=	2 yrs	Time to peak	=	12.18 hrs
Time interval	=	1 min	Hyd. volume	=	19,698 cuft
Inflow hyd. No.	=	4 - Detained A	Max. Elevation	=	1230.51 ft
Reservoir name	=	<New Pond>	Max. Storage	=	5,373 cuft

Storage Indication method used.



Pond Report

Pond No. 1 - <New Pond>

Pond Data

Contours -User-defined contour areas. Conic method used for volume calculation. Begining Elevation = 1228.00 ft

Stage / Storage Table

Stage (ft)	Elevation (ft)	Contour area (sqft)	Incr. Storage (cuft)	Total storage (cuft)
0.00	1228.00	00	0	0
1.00	1229.00	1,000	333	333
2.00	1230.00	3,000	1,910	2,244
3.00	1231.00	10,000	6,158	8,402
4.00	1232.00	12,500	11,226	19,628
5.00	1233.00	15,000	13,730	33,358
6.00	1234.00	17,000	15,988	49,346

Culvert / Orifice Structures

	[A]	[B]	[C]	[PrfRsr]
Rise (in)	= 10.00	10.00	0.00	0.00
Span (in)	= 10.00	10.00	0.00	0.00
No. Barrels	= 1	1	0	0
Invert El. (ft)	= 1228.00	1228.00	0.00	0.00
Length (ft)	= 55.00	0.50	0.00	0.00
Slope (%)	= 0.35	0.10	0.00	n/a
N-Value	= .013	.013	.013	n/a
Orifice Coeff.	= 0.60	0.60	0.60	0.60
Multi-Stage	= n/a	Yes	No	No

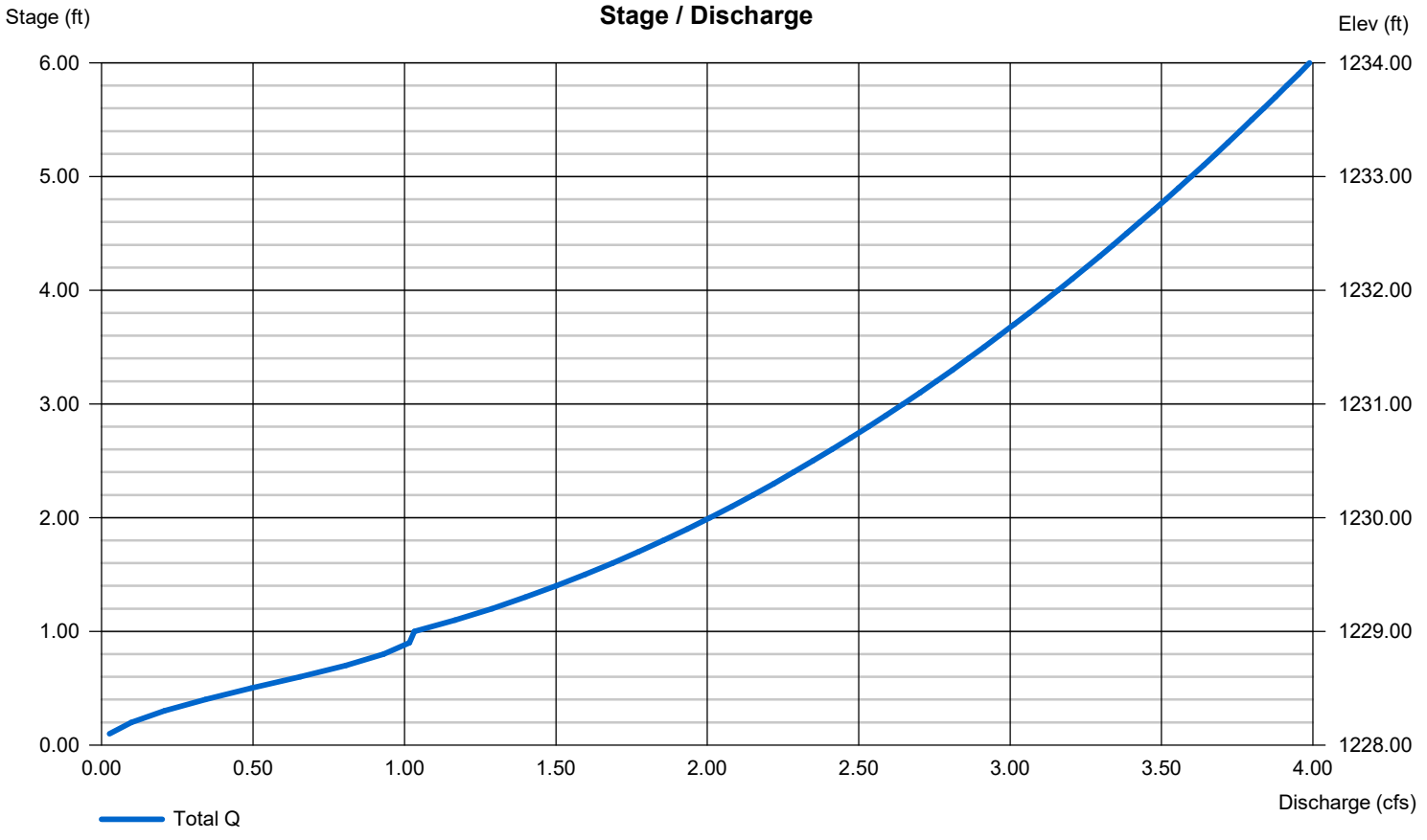
Weir Structures

	[A]	[B]	[C]	[D]
Crest Len (ft)	= 1.00	0.00	0.00	0.00
Crest El. (ft)	= 1234.00	0.00	0.00	0.00
Weir Coeff.	= 3.33	3.33	3.33	3.33
Weir Type	= Rect	---	---	---
Multi-Stage	= Yes	No	No	No

Exfil.(in/hr) = 0.000 (by Wet area)

TW Elev. (ft) = 0.00

Note: Culvert/Orifice outflows are analyzed under inlet (ic) and outlet (oc) control. Weir risers checked for orifice conditions (ic) and submergence (s).

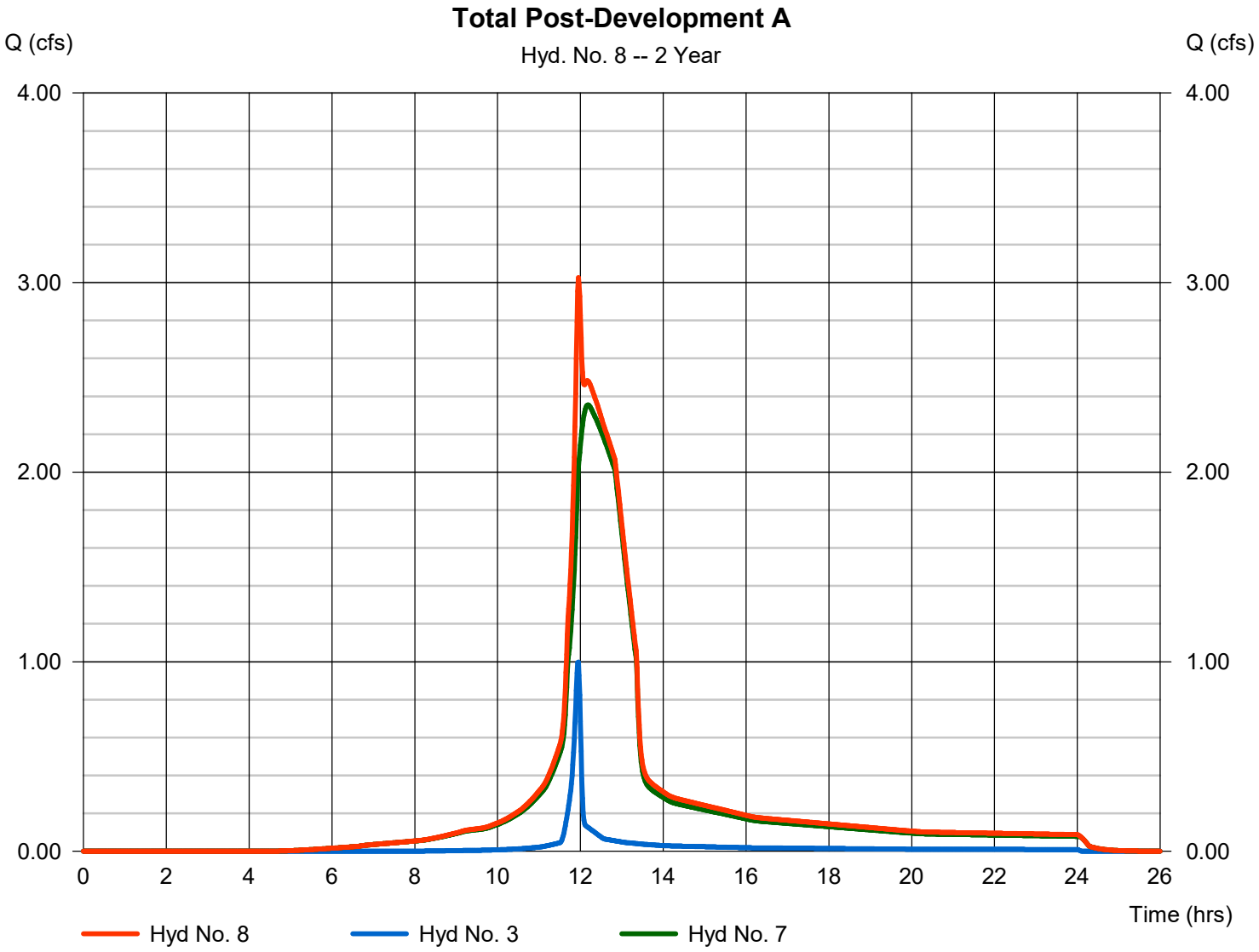


Hydrograph Report

Hyd. No. 8

Total Post-Development A

Hydrograph type	= Combine	Peak discharge	= 3.026 cfs
Storm frequency	= 2 yrs	Time to peak	= 11.95 hrs
Time interval	= 1 min	Hyd. volume	= 21,603 cuft
Inflow hyds.	= 3, 7	Contrib. drain. area	= 0.230 ac

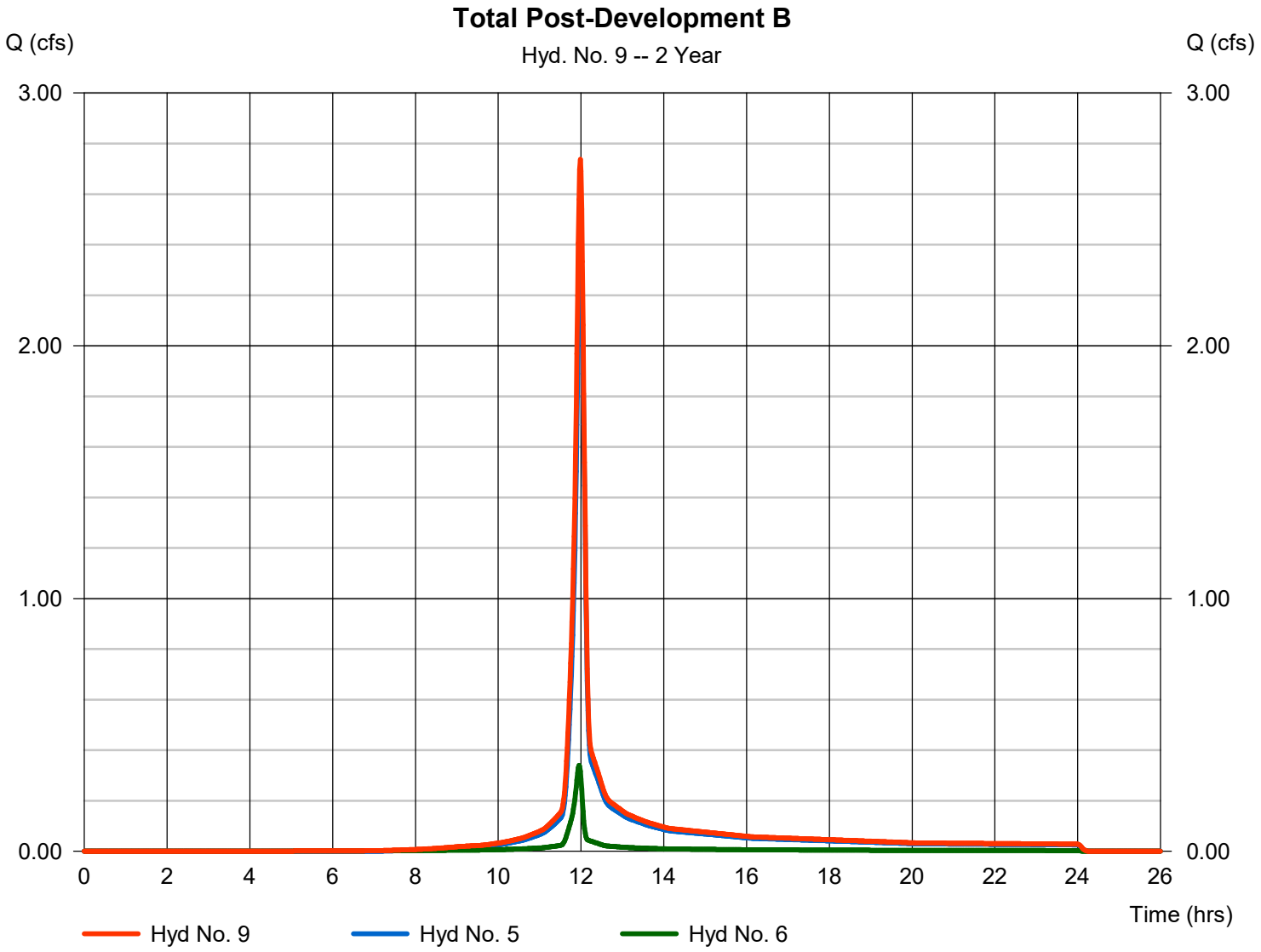


Hydrograph Report

Hyd. No. 9

Total Post-Development B

Hydrograph type	=	Combine	Peak discharge	=	2.738 cfs
Storm frequency	=	2 yrs	Time to peak	=	11.98 hrs
Time interval	=	1 min	Hyd. volume	=	6,260 cuft
Inflow hyds.	=	5, 6	Contrib. drain. area	=	0.680 ac



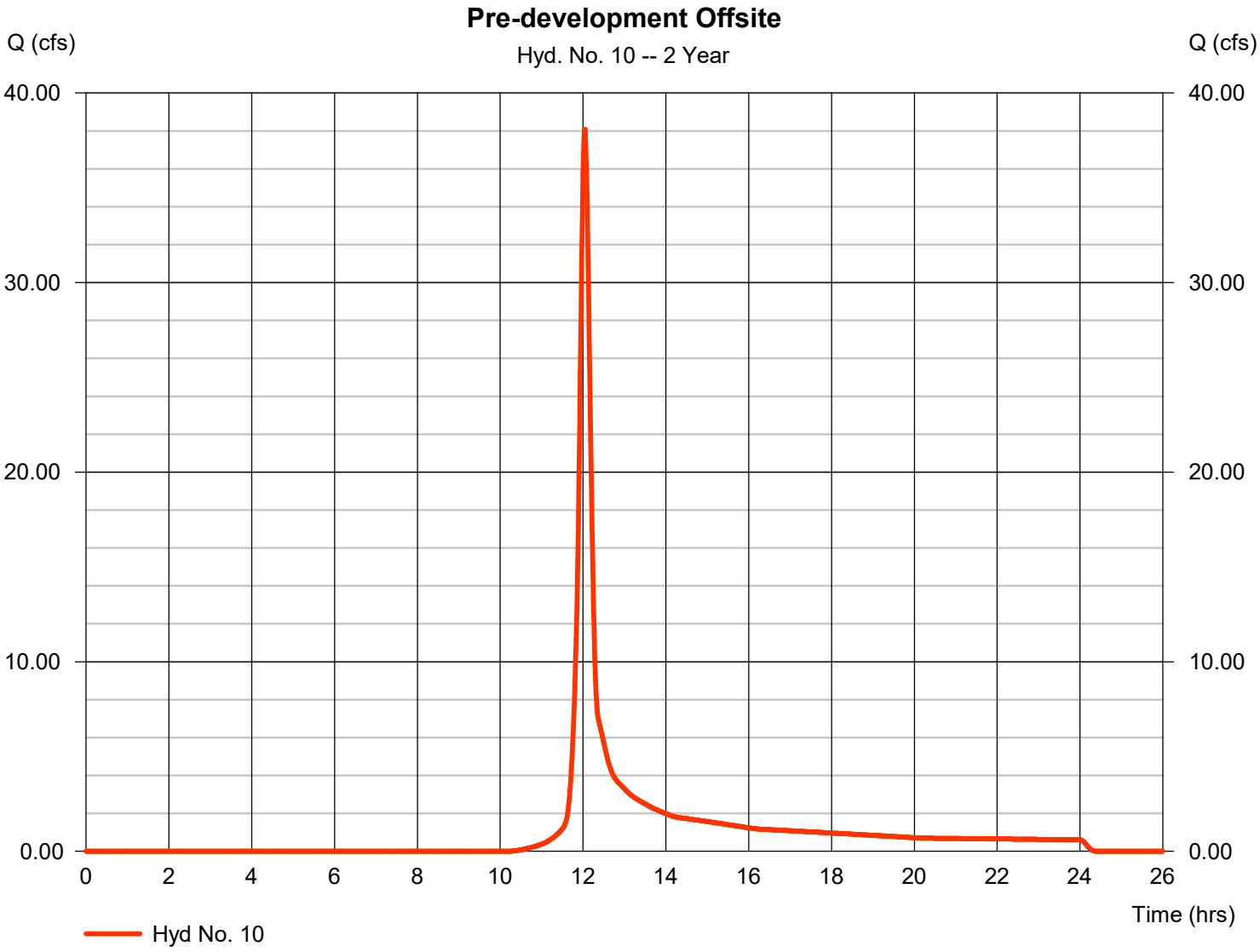
Hydrograph Report

Hyd. No. 10

Pre-development Offsite

Hydrograph type	=	SCS Runoff	Peak discharge	=	38.09 cfs
Storm frequency	=	2 yrs	Time to peak	=	12.05 hrs
Time interval	=	1 min	Hyd. volume	=	104,199 cuft
Drainage area	=	18.000 ac	Curve number	=	72*
Basin Slope	=	0.0 %	Hydraulic length	=	0 ft
Tc method	=	TR55	Time of conc. (Tc)	=	13.80 min
Total precip.	=	4.19 in	Distribution	=	Type II
Storm duration	=	24 hrs	Shape factor	=	484

* Composite (Area/CN) = [(2.140 x 79) + (0.560 x 92) + (15.300 x 70)] / 18.000



TR55 Tc Worksheet

Hyd. No. 10

Pre-development Offsite

Description	A		B		C		Totals
Sheet Flow							
Manning's n-value	= 0.150		0.011		0.011		
Flow length (ft)	= 100.0		0.0		0.0		
Two-year 24-hr precip. (in)	= 4.19		0.00		0.00		
Land slope (%)	= 9.00		0.00		0.00		
Travel Time (min)	**= 4.69**	**+**	**0.00**	**+**	**0.00**	**=**	**4.69**
Shallow Concentrated Flow							
Flow length (ft)	= 301.00		0.00		0.00		
Watercourse slope (%)	= 8.00		0.00		0.00		
Surface description	= Unpaved		Paved		Paved		
Average velocity (ft/s)	=4.56		0.00		0.00		
Travel Time (min)	**= 1.10**	**+**	**0.00**	**+**	**0.00**	**=**	**1.10**
Channel Flow							
X sectional flow area (sqft)	= 4.00		0.00		0.00		
Wetted perimeter (ft)	= 16.00		0.00		0.00		
Channel slope (%)	= 2.70		0.00		0.00		
Manning's n-value	= 0.040		0.015		0.015		
Velocity (ft/s)	=2.42						
			0.00				
					0.00		
Flow length (ft)	({0})1157.0		0.0		0.0		
Travel Time (min)	**= 7.98**	**+**	**0.00**	**+**	**0.00**	**=**	**7.98**

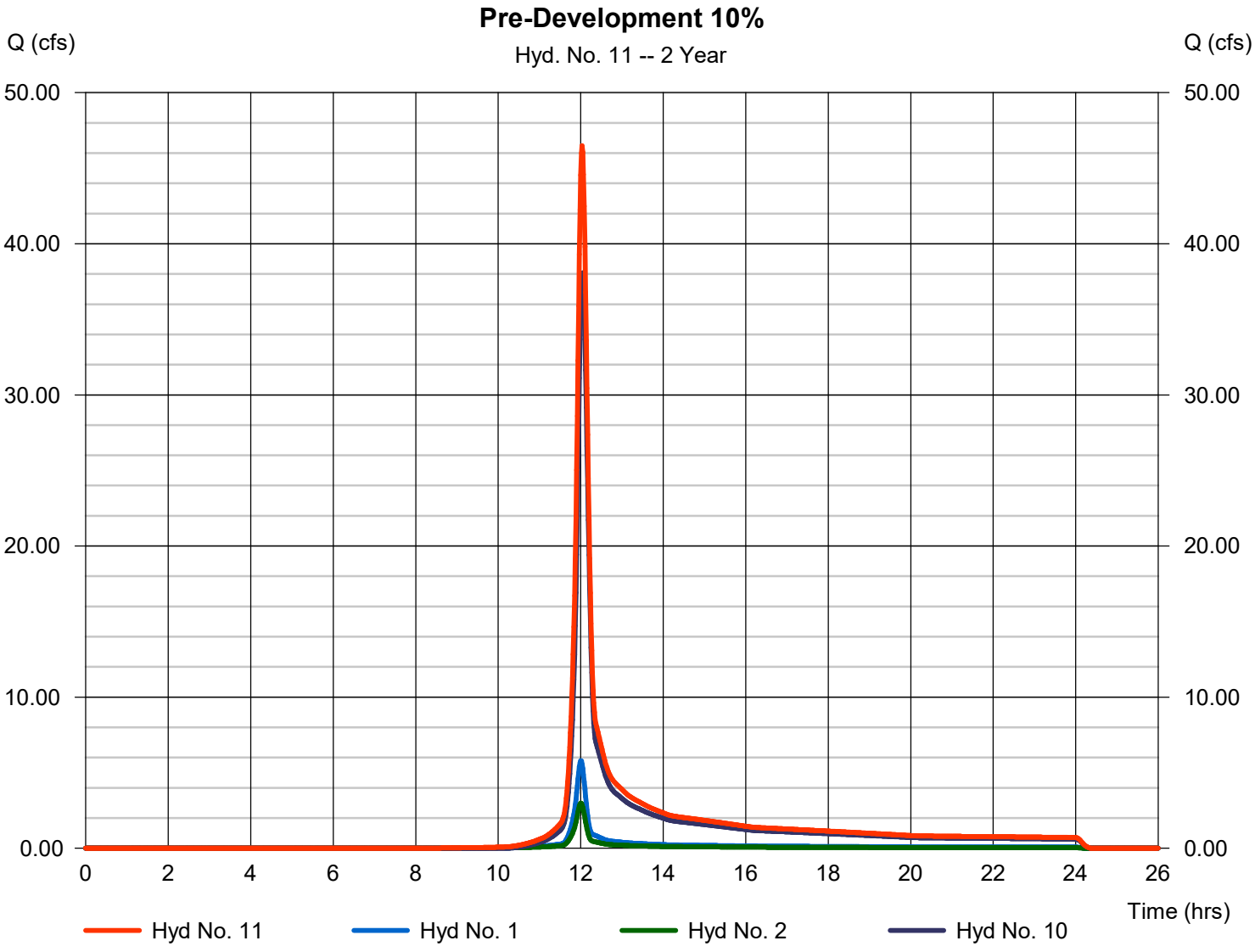
Total Travel Time, Tc ..... 13.80 min

Hydrograph Report

Hyd. No. 11

Pre-Development 10%

Hydrograph type	= Combine	Peak discharge	= 46.49 cfs
Storm frequency	= 2 yrs	Time to peak	= 12.03 hrs
Time interval	= 1 min	Hyd. volume	= 125,469 cuft
Inflow hyds.	= 1, 2, 10	Contrib. drain. area	= 20.620 ac

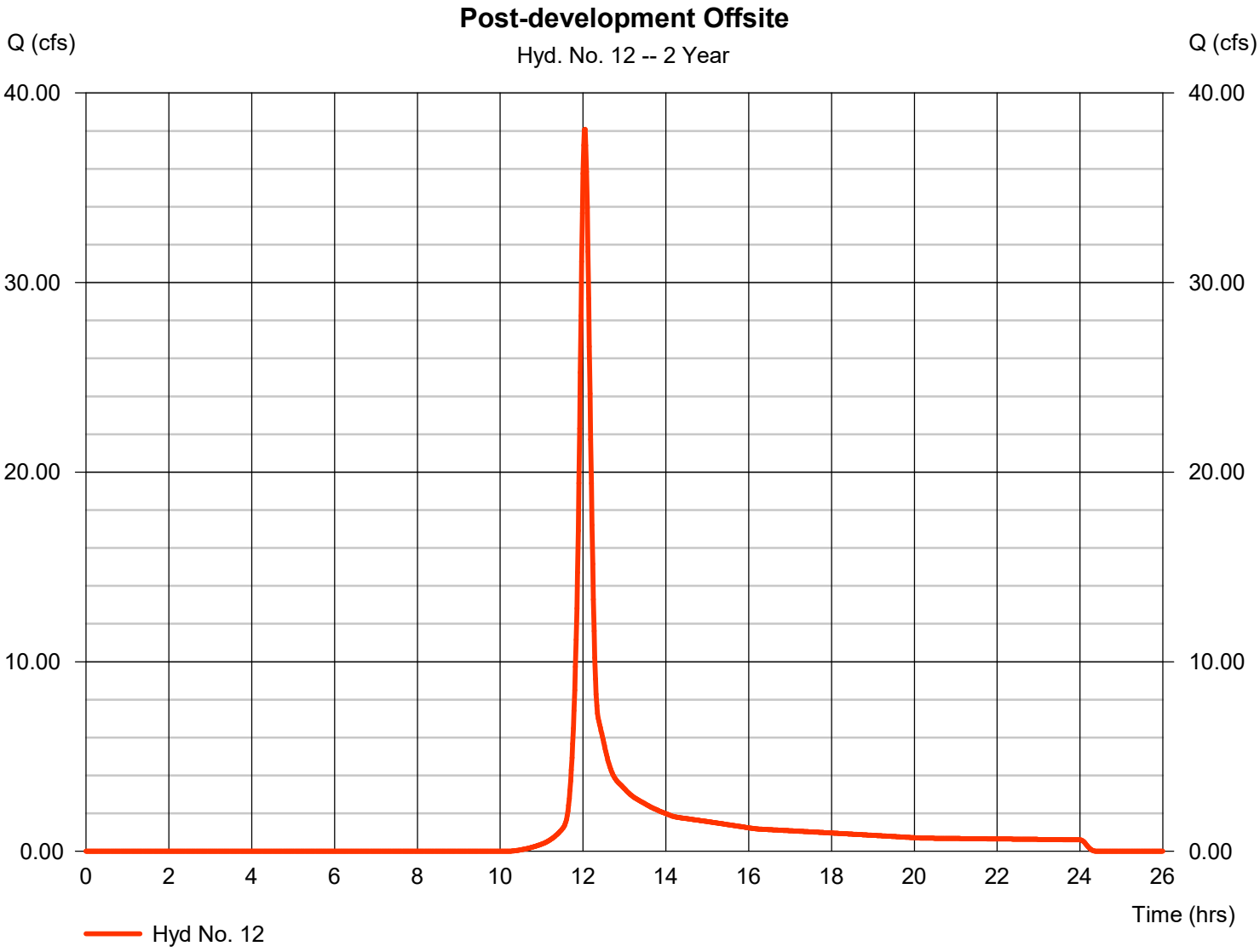


Hydrograph Report

Hyd. No. 12

Post-development Offsite

Hydrograph type	=	SCS Runoff	Peak discharge	=	38.09 cfs
Storm frequency	=	2 yrs	Time to peak	=	12.05 hrs
Time interval	=	1 min	Hyd. volume	=	104,199 cuft
Drainage area	=	18.000 ac	Curve number	=	72
Basin Slope	=	0.0 %	Hydraulic length	=	0 ft
Tc method	=	TR55	Time of conc. (Tc)	=	13.80 min
Total precip.	=	4.19 in	Distribution	=	Type II
Storm duration	=	24 hrs	Shape factor	=	484



TR55 Tc Worksheet

Hyd. No. 12

Post-development Offsite

Description	A		B		C		Totals
Sheet Flow							
Manning's n-value	= 0.150		0.011		0.011		
Flow length (ft)	= 100.0		0.0		0.0		
Two-year 24-hr precip. (in)	= 4.19		0.00		0.00		
Land slope (%)	= 9.00		0.00		0.00		
Travel Time (min)	**= 4.69**	**+**	**0.00**	**+**	**0.00**	**=**	**4.69**
Shallow Concentrated Flow							
Flow length (ft)	= 301.00		0.00		0.00		
Watercourse slope (%)	= 8.00		0.00		0.00		
Surface description	= Unpaved		Paved		Paved		
Average velocity (ft/s)	=4.56		0.00		0.00		
Travel Time (min)	**= 1.10**	**+**	**0.00**	**+**	**0.00**	**=**	**1.10**
Channel Flow							
X sectional flow area (sqft)	= 4.00		0.00		0.00		
Wetted perimeter (ft)	= 16.00		0.00		0.00		
Channel slope (%)	= 2.70		0.00		0.00		
Manning's n-value	= 0.040		0.015		0.015		
Velocity (ft/s)	=2.42						
			0.00				
					0.00		
Flow length (ft)	({0})1157.0		0.0		0.0		
Travel Time (min)	**= 7.98**	**+**	**0.00**	**+**	**0.00**	**=**	**7.98**

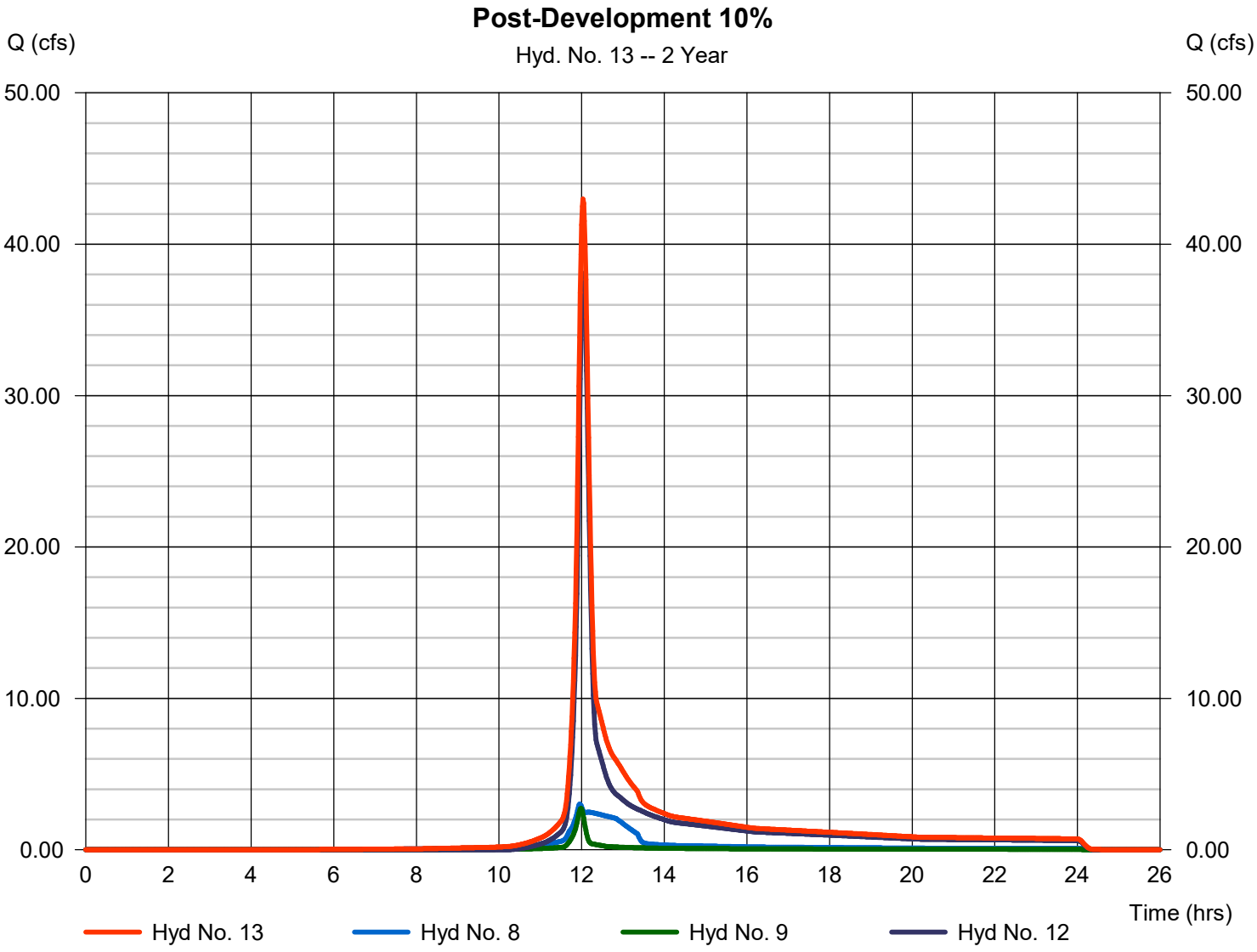
Total Travel Time, Tc ..... 13.80 min

Hydrograph Report

Hyd. No. 13

Post-Development 10%

Hydrograph type	= Combine	Peak discharge	= 42.99 cfs
Storm frequency	= 2 yrs	Time to peak	= 12.03 hrs
Time interval	= 1 min	Hyd. volume	= 132,062 cuft
Inflow hyds.	= 8, 9, 12	Contrib. drain. area	= 18.000 ac



Hydrograph Summary Report

Hydraflow Hydrographs Extension for Autodesk® Civil 3D® by Autodesk, Inc. v2022

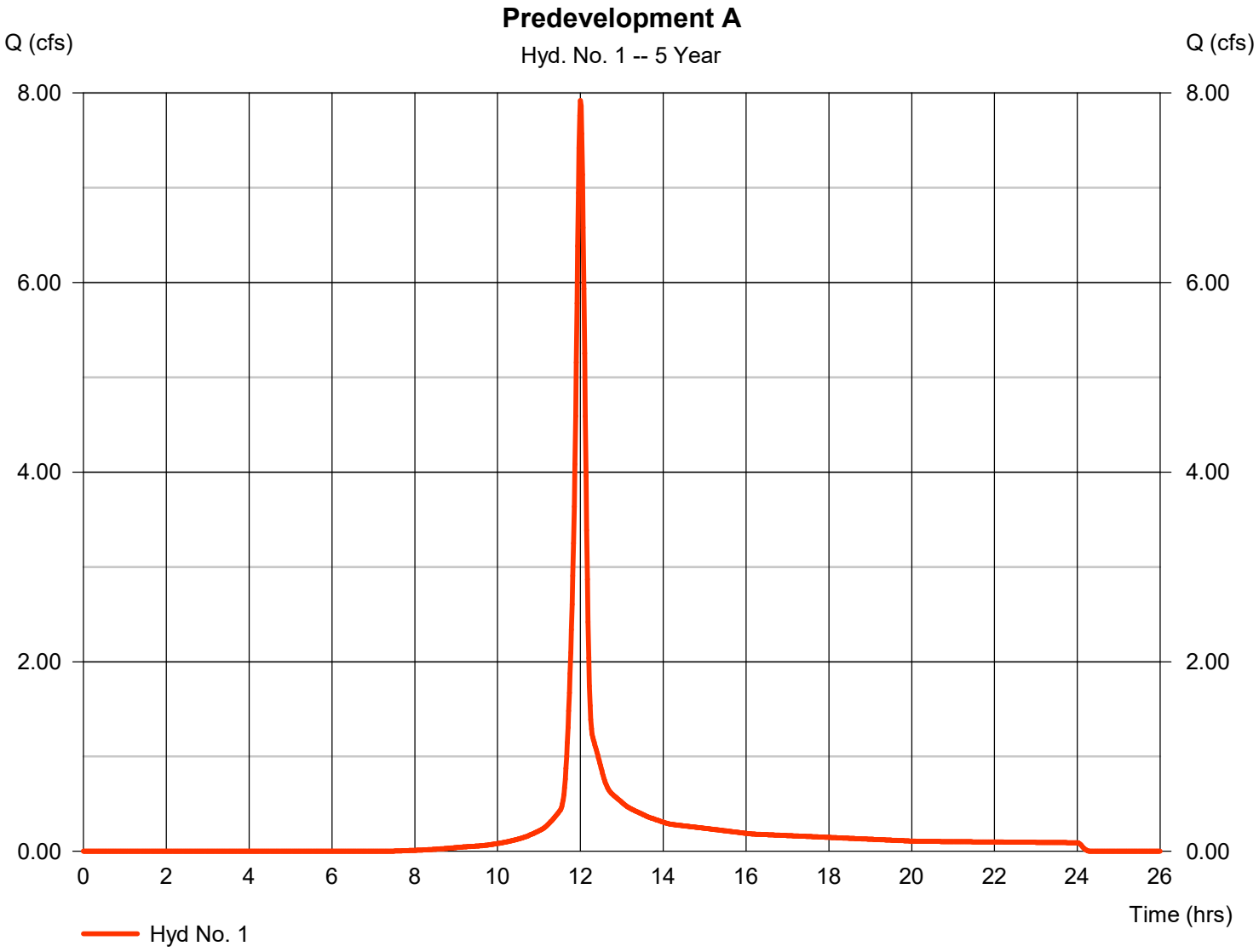
Hyd. No.	Hydrograph type (origin)	Peak flow (cfs)	Time interval (min)	Time to Peak (min)	Hyd. volume (cuft)	Inflow hyd(s)	Maximum elevation (ft)	Total strge used (cuft)	Hydrograph Description
1	SCS Runoff	7.921	1	720	19,214	------	------	------	Predevelopment A
2	SCS Runoff	4.000	1	720	9,772	------	------	------	Predevelopment B
3	SCS Runoff	1.335	1	716	2,581	------	------	------	Undetained A
4	SCS Runoff	9.947	1	720	25,370	------	------	------	Detained A
5	SCS Runoff	3.212	1	719	7,388	------	------	------	Undetained B
6	SCS Runoff	0.427	1	717	945	------	------	------	Undetained B2
7	Reservoir	2.560	1	732	25,369	4	1230.85	7,469	ADS Underground System
8	Combine	3.470	1	717	27,950	3, 7	------	------	Total Post-Development A
9	Combine	3.607	1	719	8,333	5, 6,	------	------	Total Post-Development B
10	SCS Runoff	55.44	1	722	149,673	------	------	------	Pre-development Offsite
11	Combine	66.81	1	722	178,660	1, 2, 10	------	------	Pre-Development 10%
12	SCS Runoff	55.44	1	722	149,673	------	------	------	Post-development Offsite
13	Combine	61.37	1	722	185,957	8, 9, 12	------	------	Post-Development 10%

Hydrograph Report

Hyd. No. 1

Predevelopment A

Hydrograph type	=	SCS Runoff	Peak discharge	=	7.921 cfs
Storm frequency	=	5 yrs	Time to peak	=	12.00 hrs
Time interval	=	1 min	Hyd. volume	=	19,214 cuft
Drainage area	=	1.790 ac	Curve number	=	79
Basin Slope	=	2.0 %	Hydraulic length	=	130 ft
Tc method	=	TR55	Time of conc. (Tc)	=	11.00 min
Total precip.	=	5.12 in	Distribution	=	Type II
Storm duration	=	24 hrs	Shape factor	=	484



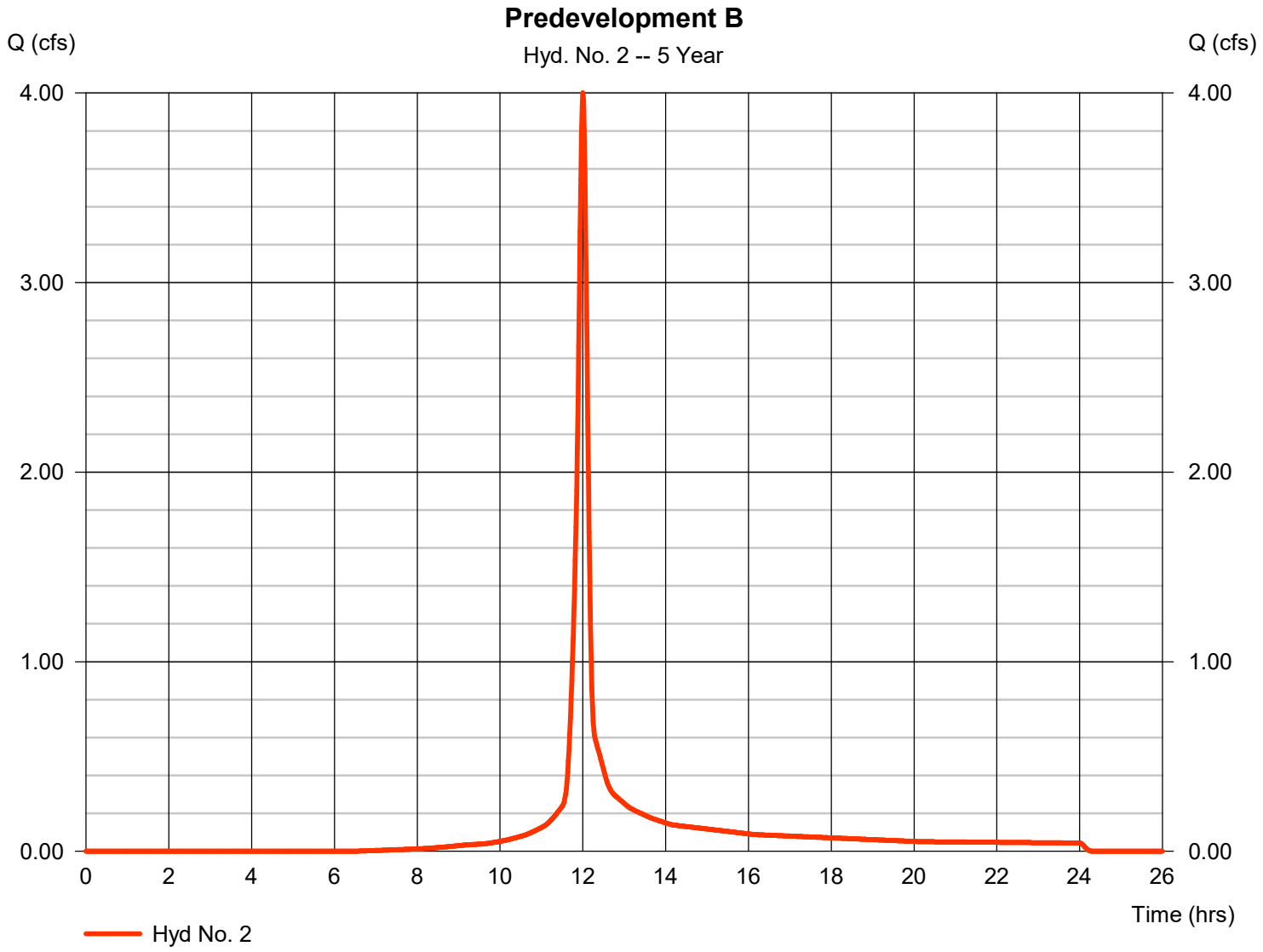
Hydrograph Report

Hyd. No. 2

Predevelopment B

Hydrograph type	=	SCS Runoff	Peak discharge	=	4.000 cfs
Storm frequency	=	5 yrs	Time to peak	=	12.00 hrs
Time interval	=	1 min	Hyd. volume	=	9,772 cuft
Drainage area	=	0.830 ac	Curve number	=	82*
Basin Slope	=	0.0 %	Hydraulic length	=	0 ft
Tc method	=	TR55	Time of conc. (Tc)	=	10.60 min
Total precip.	=	5.12 in	Distribution	=	Type II
Storm duration	=	24 hrs	Shape factor	=	484

* Composite (Area/CN) = [(0.120 x 98) + (0.710 x 79)] / 0.830



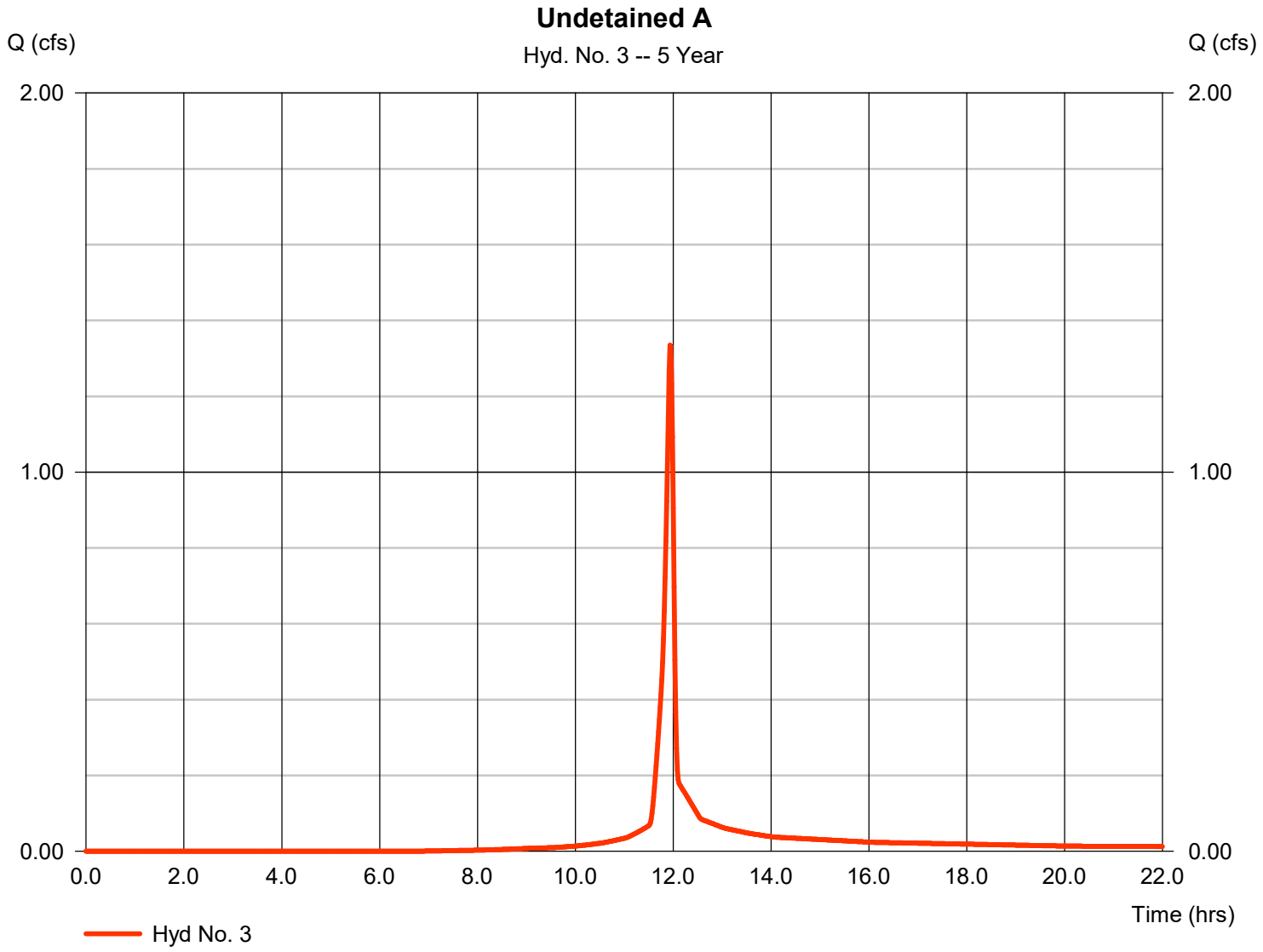
Hydrograph Report

Hyd. No. 3

Undetained A

Hydrograph type	=	SCS Runoff	Peak discharge	=	1.335 cfs
Storm frequency	=	5 yrs	Time to peak	=	11.93 hrs
Time interval	=	1 min	Hyd. volume	=	2,581 cuft
Drainage area	=	0.230 ac	Curve number	=	81*
Basin Slope	=	0.0 %	Hydraulic length	=	0 ft
Tc method	=	TR55	Time of conc. (Tc)	=	4.40 min
Total precip.	=	5.12 in	Distribution	=	Type II
Storm duration	=	24 hrs	Shape factor	=	484

* Composite (Area/CN) = [(0.200 x 79) + (0.030 x 98)] / 0.230



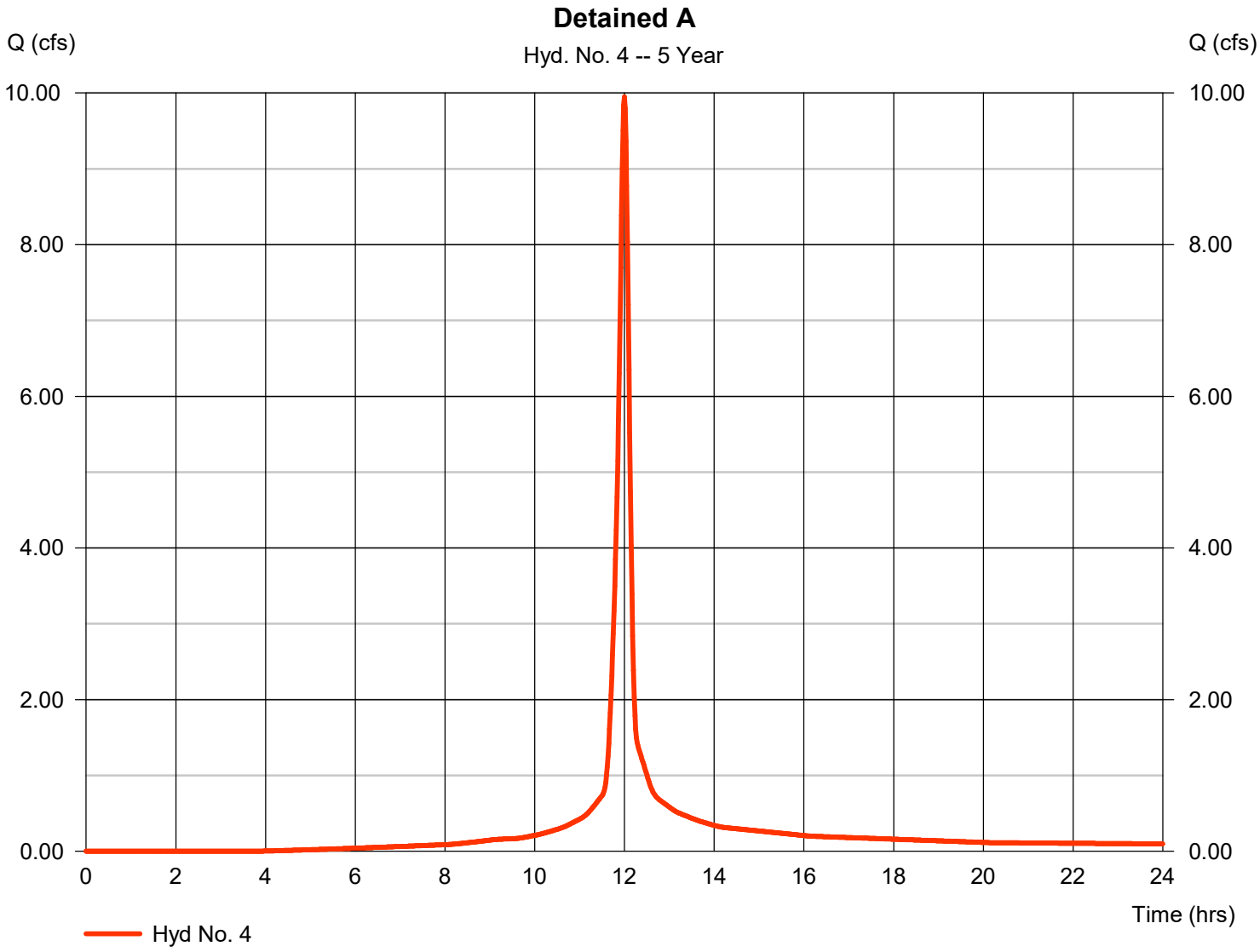
Hydrograph Report

Hyd. No. 4

Detained A

Hydrograph type	=	SCS Runoff	Peak discharge	=	9.947 cfs
Storm frequency	=	5 yrs	Time to peak	=	12.00 hrs
Time interval	=	1 min	Hyd. volume	=	25,370 cuft
Drainage area	=	1.720 ac	Curve number	=	90*
Basin Slope	=	0.0 %	Hydraulic length	=	0 ft
Tc method	=	TR55	Time of conc. (Tc)	=	10.40 min
Total precip.	=	5.12 in	Distribution	=	Type II
Storm duration	=	24 hrs	Shape factor	=	484

* Composite (Area/CN) = [(0.970 x 98) + (0.750 x 79)] / 1.720



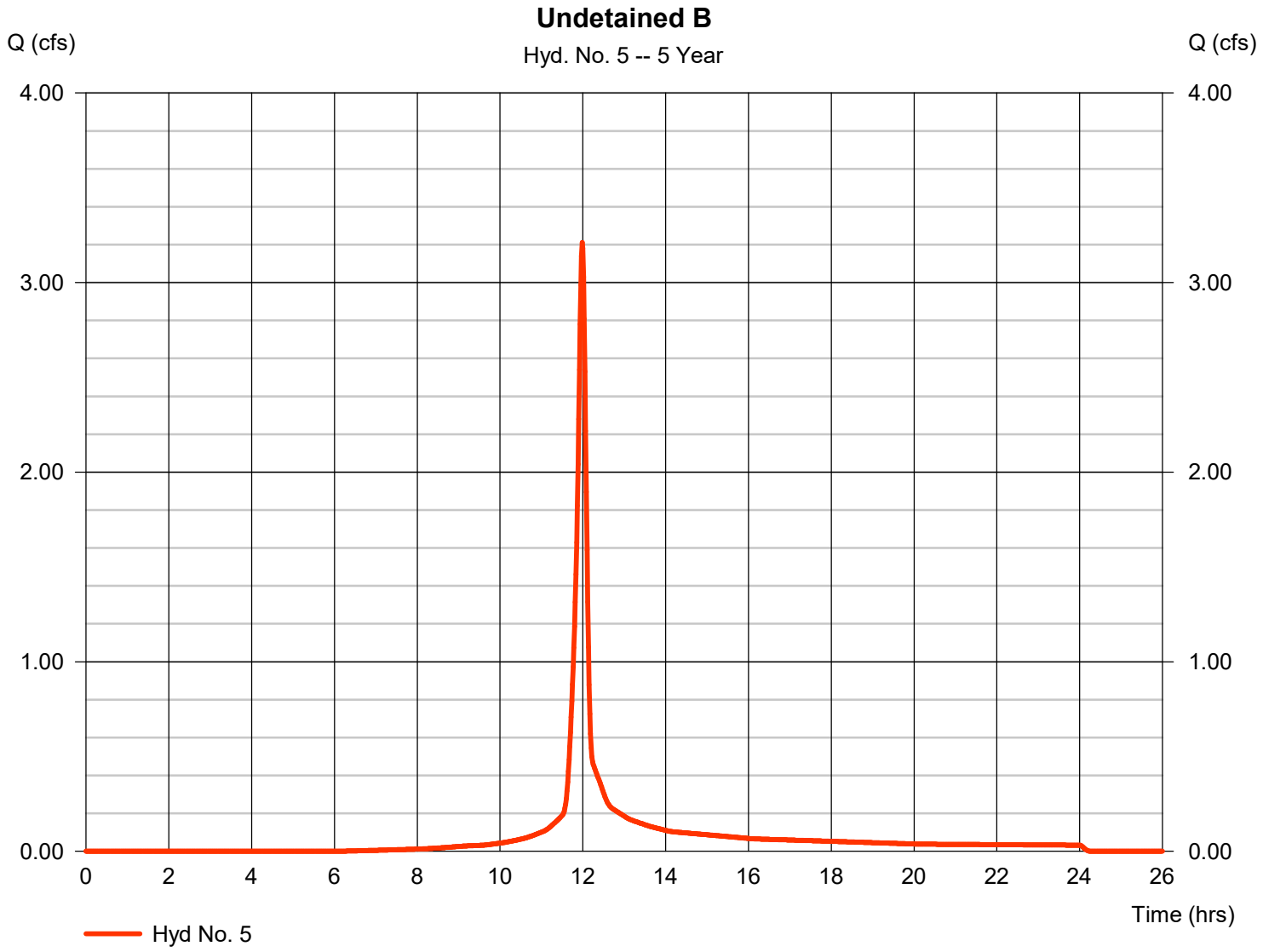
Hydrograph Report

Hyd. No. 5

Undetained B

Hydrograph type	=	SCS Runoff	Peak discharge	=	3.212 cfs
Storm frequency	=	5 yrs	Time to peak	=	11.98 hrs
Time interval	=	1 min	Hyd. volume	=	7,388 cuft
Drainage area	=	0.620 ac	Curve number	=	83*
Basin Slope	=	0.0 %	Hydraulic length	=	0 ft
Tc method	=	TR55	Time of conc. (Tc)	=	10.00 min
Total precip.	=	5.12 in	Distribution	=	Type II
Storm duration	=	24 hrs	Shape factor	=	484

* Composite (Area/CN) = [(0.130 x 98) + (0.490 x 79)] / 0.620



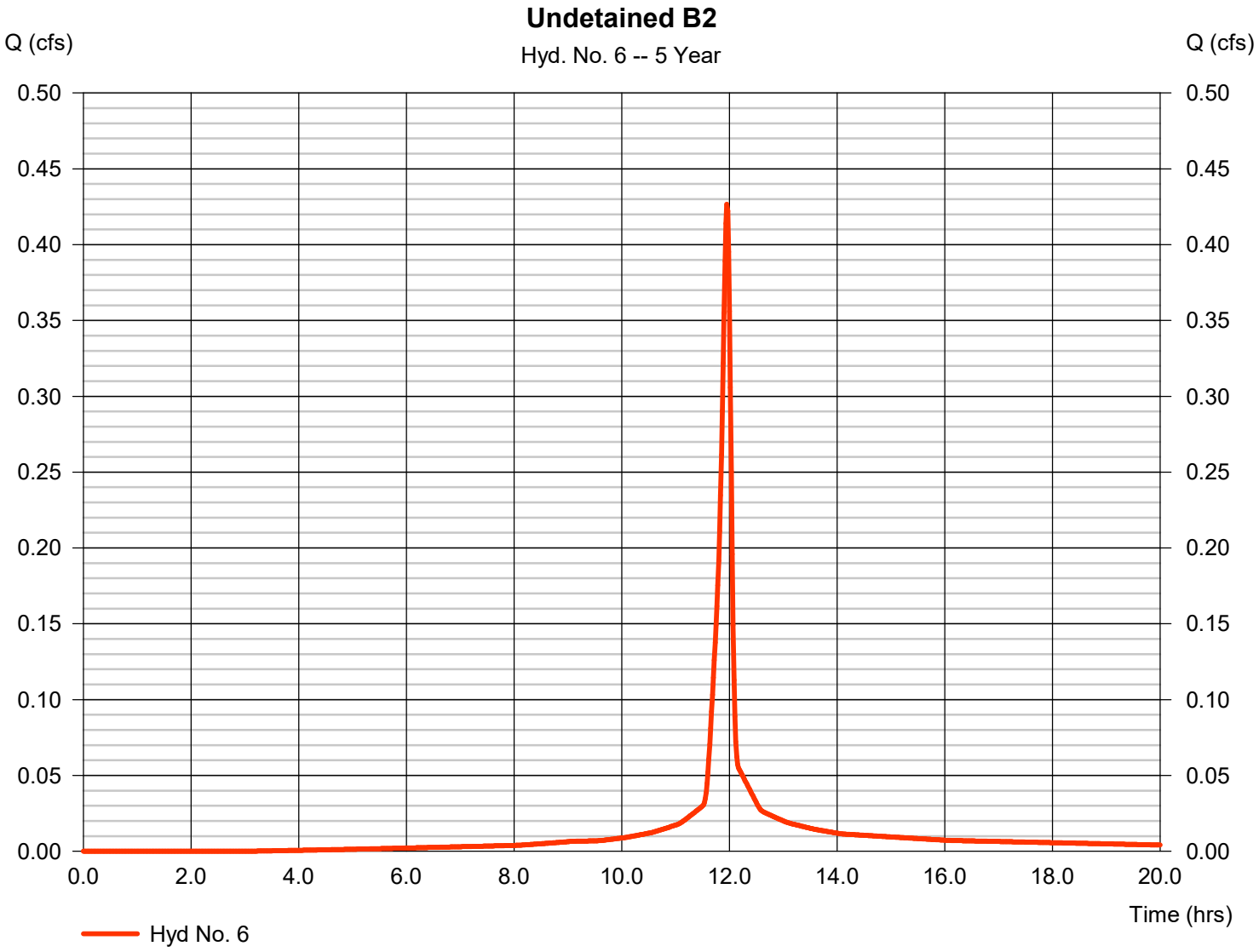
Hydrograph Report

Hyd. No. 6

Undetained B2

Hydrograph type	=	SCS Runoff	Peak discharge	=	0.427 cfs
Storm frequency	=	5 yrs	Time to peak	=	11.95 hrs
Time interval	=	1 min	Hyd. volume	=	945 cuft
Drainage area	=	0.060 ac	Curve number	=	92*
Basin Slope	=	0.0 %	Hydraulic length	=	0 ft
Tc method	=	User	Time of conc. (Tc)	=	5.00 min
Total precip.	=	5.12 in	Distribution	=	Type II
Storm duration	=	24 hrs	Shape factor	=	484

* Composite (Area/CN) = [(0.020 x 79) + (0.040 x 98)] / 0.060



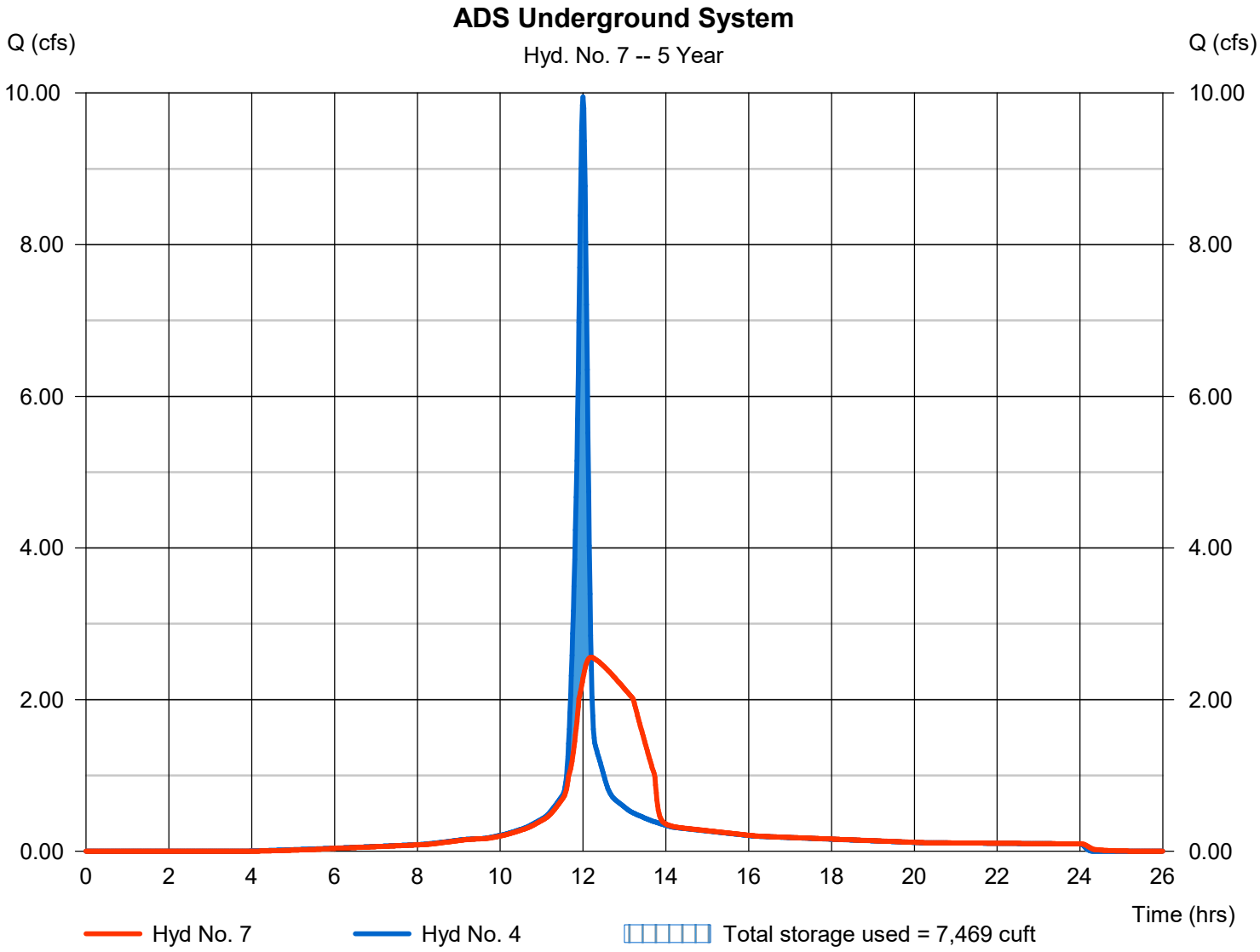
Hydrograph Report

Hyd. No. 7

ADS Underground System

Hydrograph type	=	Reservoir	Peak discharge	=	2.560 cfs
Storm frequency	=	5 yrs	Time to peak	=	12.20 hrs
Time interval	=	1 min	Hyd. volume	=	25,369 cuft
Inflow hyd. No.	=	4 - Detained A	Max. Elevation	=	1230.85 ft
Reservoir name	=	<New Pond>	Max. Storage	=	7,469 cuft

Storage Indication method used.

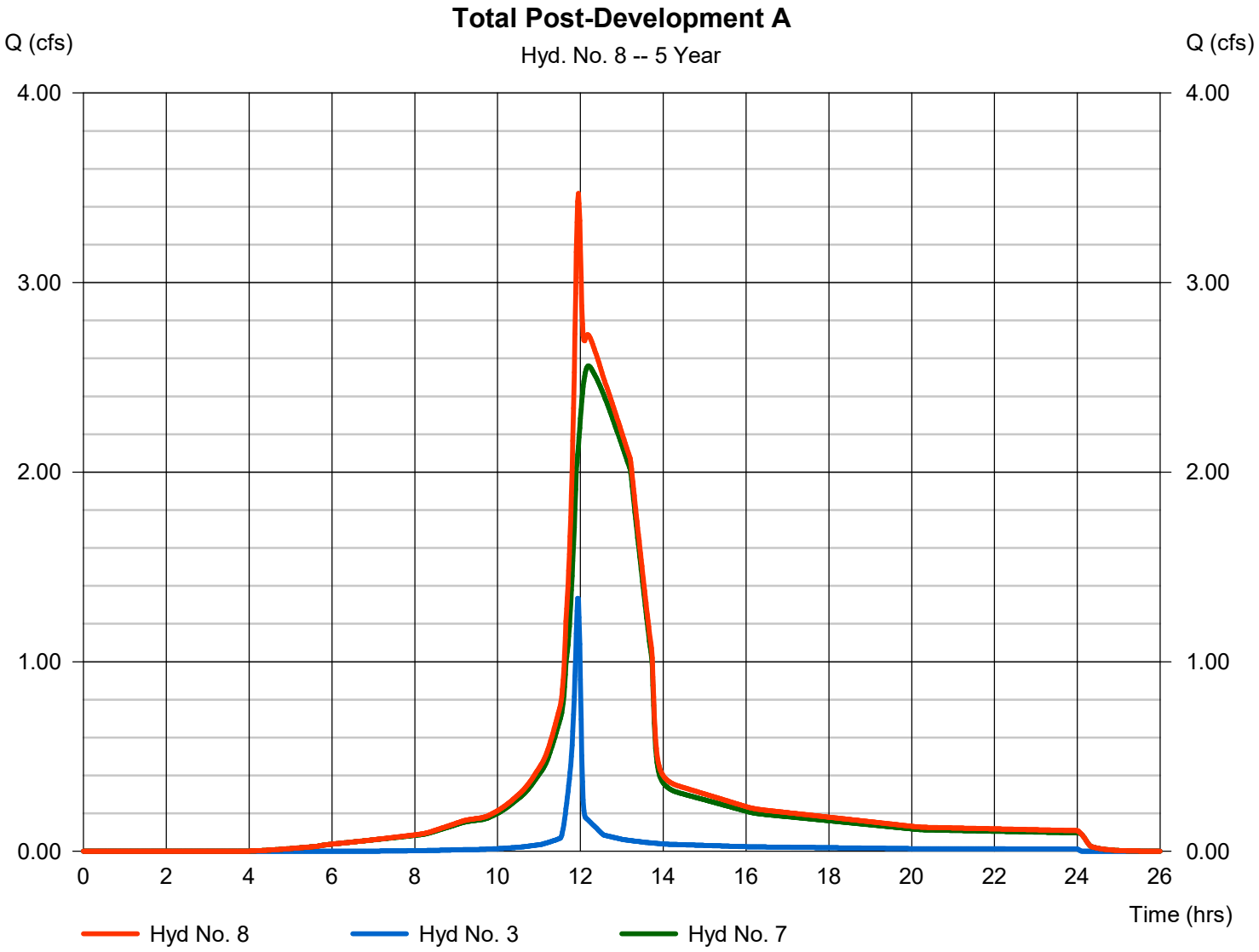


Hydrograph Report

Hyd. No. 8

Total Post-Development A

Hydrograph type	= Combine	Peak discharge	= 3.470 cfs
Storm frequency	= 5 yrs	Time to peak	= 11.95 hrs
Time interval	= 1 min	Hyd. volume	= 27,950 cuft
Inflow hyds.	= 3, 7	Contrib. drain. area	= 0.230 ac

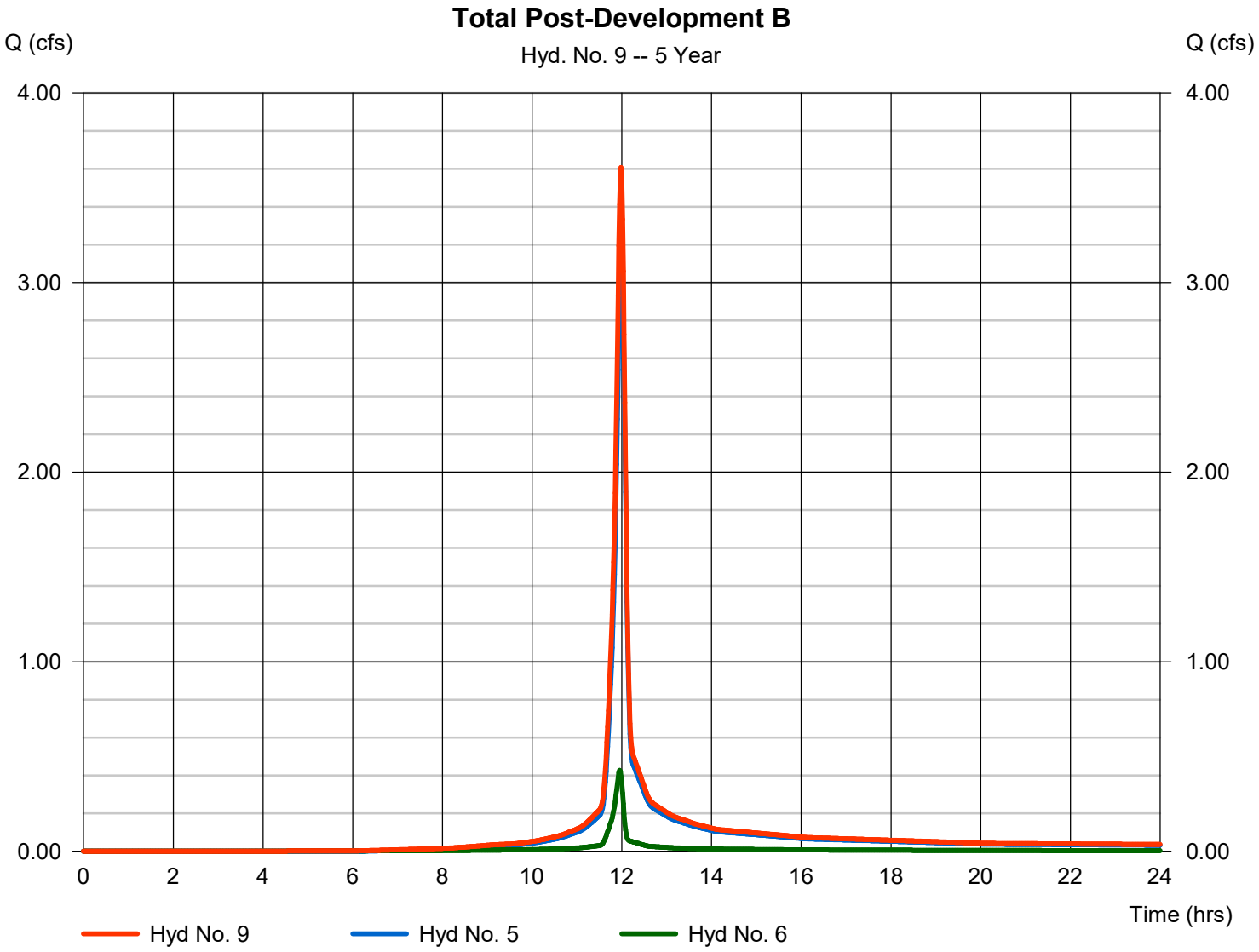


Hydrograph Report

Hyd. No. 9

Total Post-Development B

Hydrograph type	= Combine	Peak discharge	= 3.607 cfs
Storm frequency	= 5 yrs	Time to peak	= 11.98 hrs
Time interval	= 1 min	Hyd. volume	= 8,333 cuft
Inflow hyds.	= 5, 6	Contrib. drain. area	= 0.680 ac



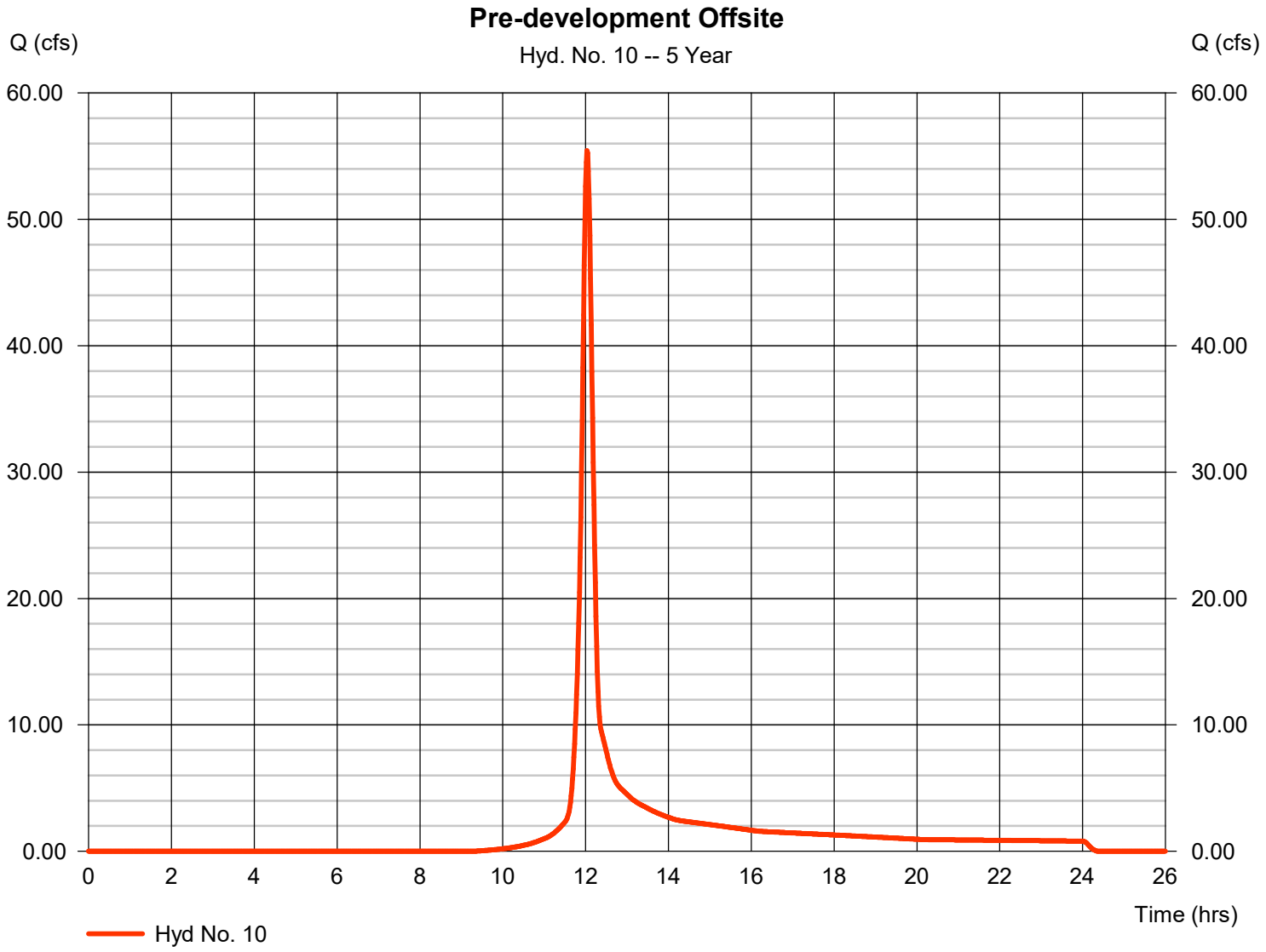
Hydrograph Report

Hyd. No. 10

Pre-development Offsite

Hydrograph type	=	SCS Runoff	Peak discharge	=	55.44 cfs
Storm frequency	=	5 yrs	Time to peak	=	12.03 hrs
Time interval	=	1 min	Hyd. volume	=	149,673 cuft
Drainage area	=	18.000 ac	Curve number	=	72*
Basin Slope	=	0.0 %	Hydraulic length	=	0 ft
Tc method	=	TR55	Time of conc. (Tc)	=	13.80 min
Total precip.	=	5.12 in	Distribution	=	Type II
Storm duration	=	24 hrs	Shape factor	=	484

* Composite (Area/CN) = [(2.140 x 79) + (0.560 x 92) + (15.300 x 70)] / 18.000

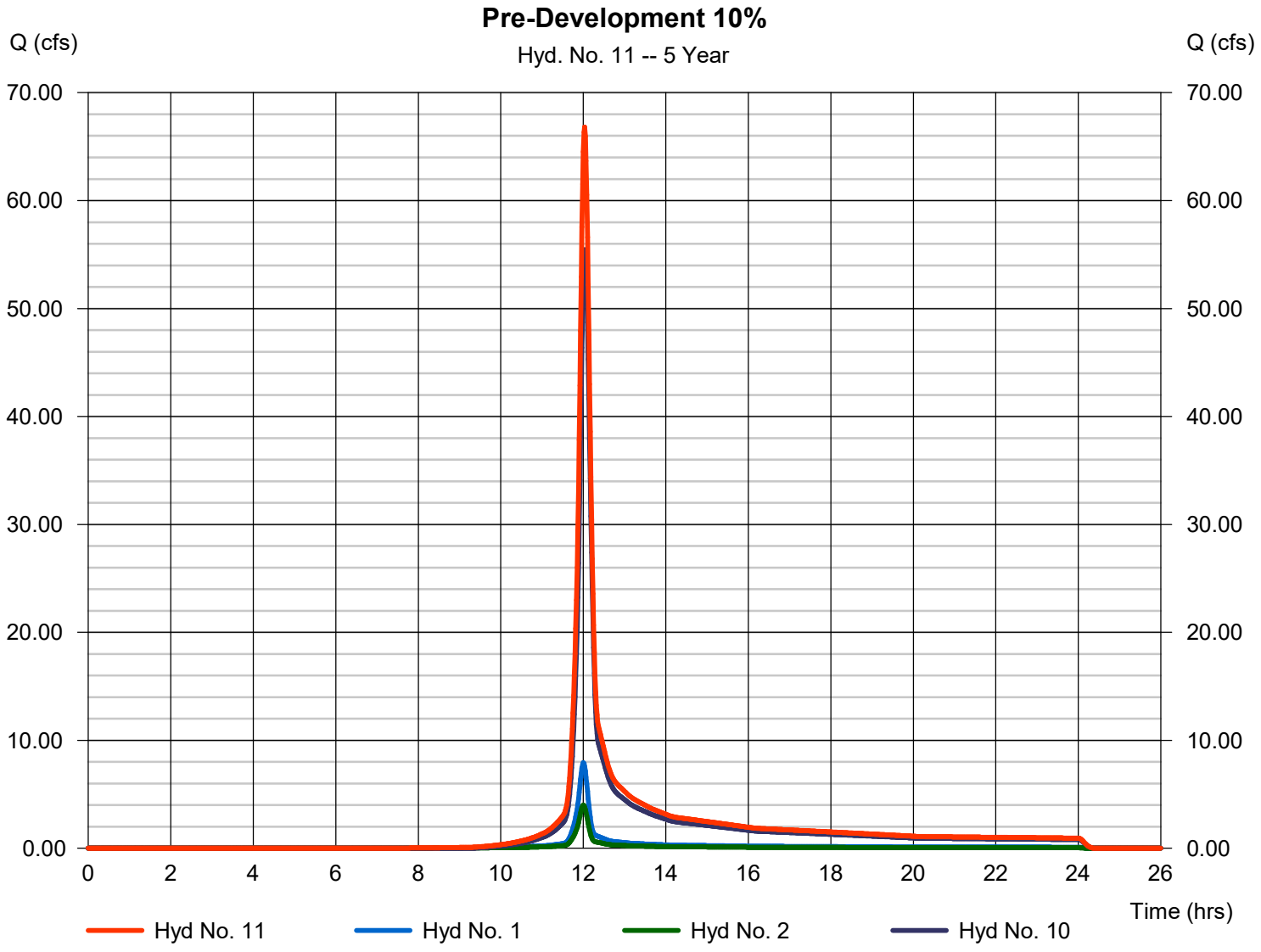


Hydrograph Report

Hyd. No. 11

Pre-Development 10%

Hydrograph type	= Combine	Peak discharge	= 66.81 cfs
Storm frequency	= 5 yrs	Time to peak	= 12.03 hrs
Time interval	= 1 min	Hyd. volume	= 178,660 cuft
Inflow hyds.	= 1, 2, 10	Contrib. drain. area	= 20.620 ac

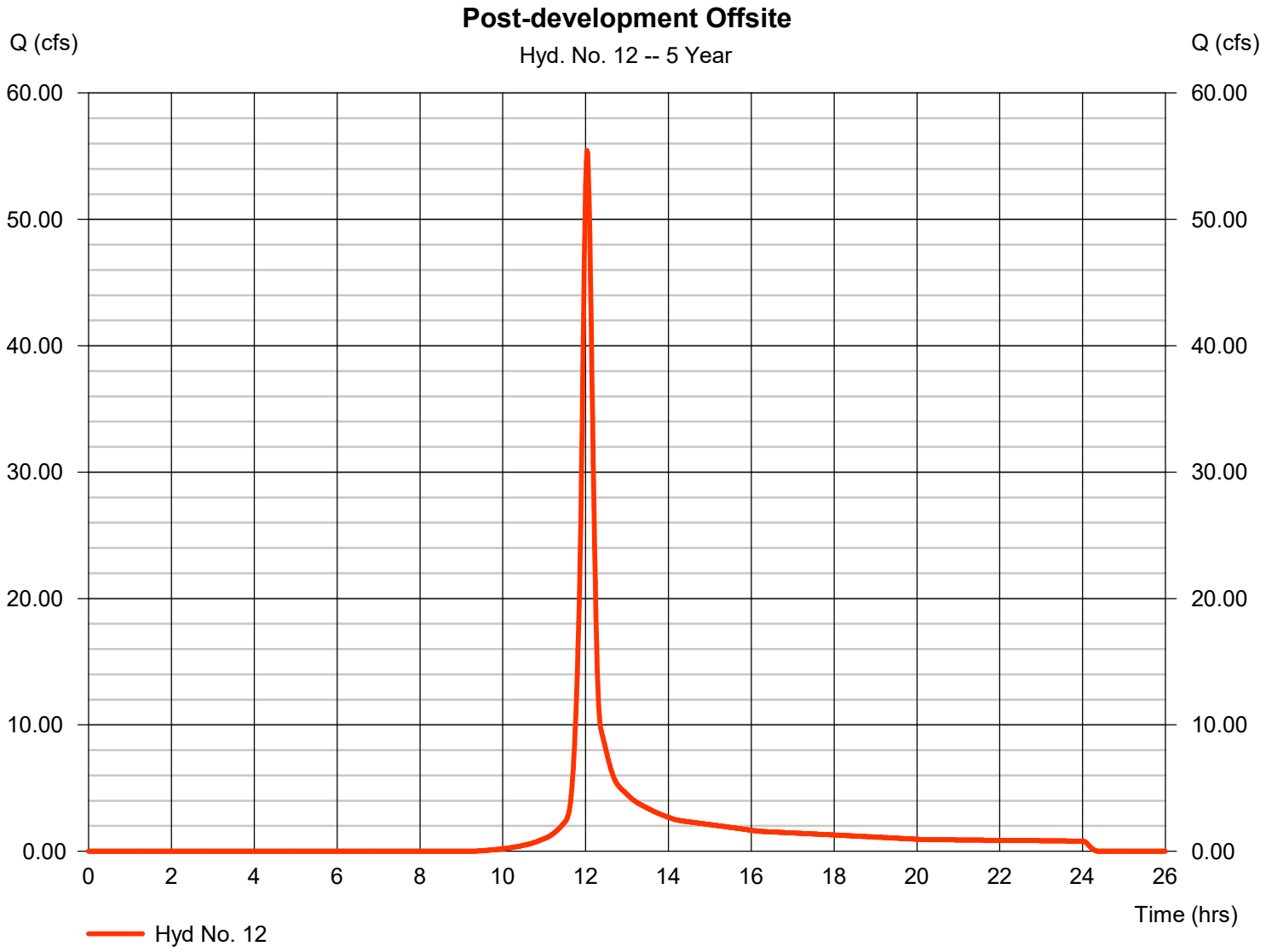


Hydrograph Report

Hyd. No. 12

Post-development Offsite

Hydrograph type	=	SCS Runoff	Peak discharge	=	55.44 cfs
Storm frequency	=	5 yrs	Time to peak	=	12.03 hrs
Time interval	=	1 min	Hyd. volume	=	149,673 cuft
Drainage area	=	18.000 ac	Curve number	=	72
Basin Slope	=	0.0 %	Hydraulic length	=	0 ft
Tc method	=	TR55	Time of conc. (Tc)	=	13.80 min
Total precip.	=	5.12 in	Distribution	=	Type II
Storm duration	=	24 hrs	Shape factor	=	484

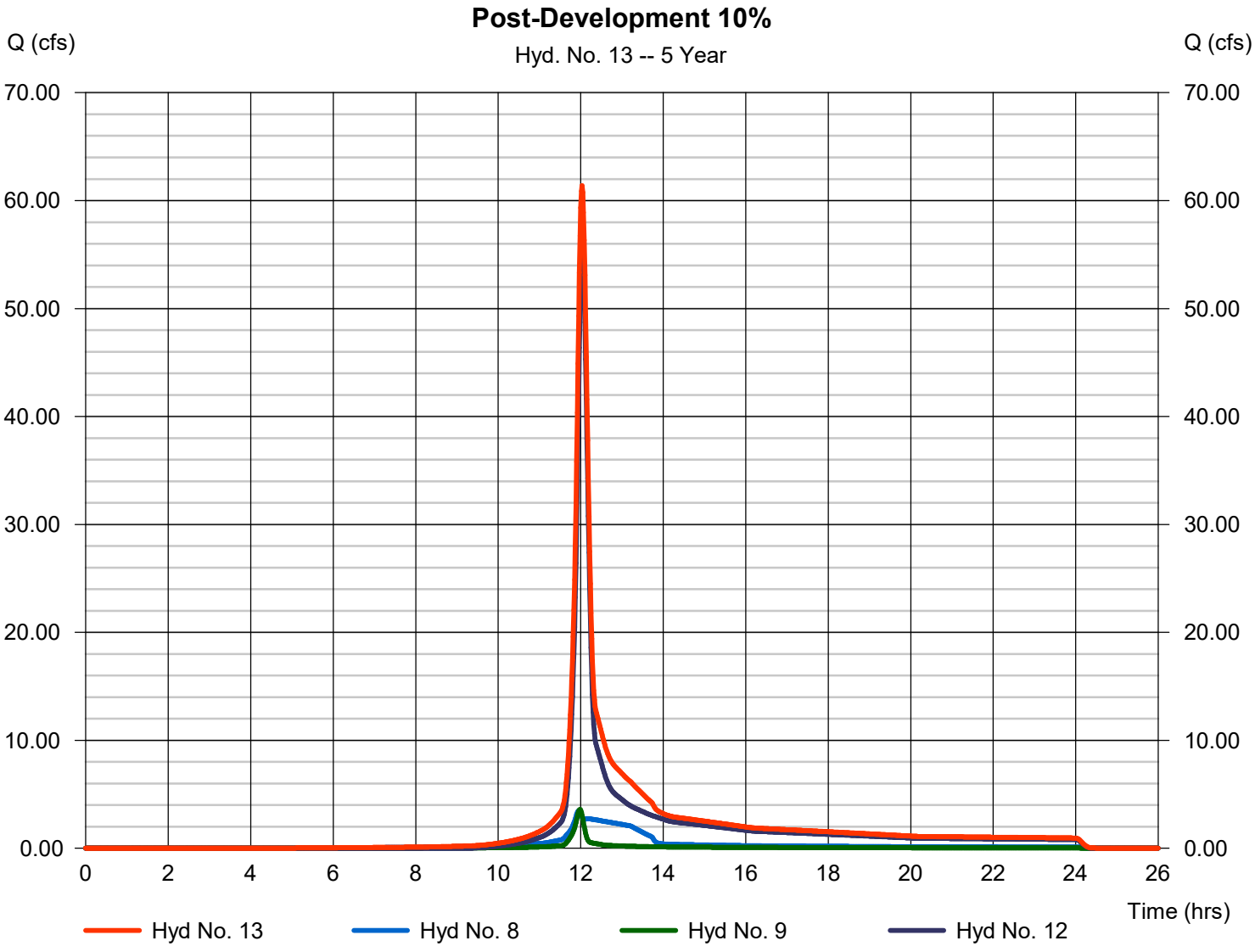


Hydrograph Report

Hyd. No. 13

Post-Development 10%

Hydrograph type	= Combine	Peak discharge	= 61.37 cfs
Storm frequency	= 5 yrs	Time to peak	= 12.03 hrs
Time interval	= 1 min	Hyd. volume	= 185,957 cuft
Inflow hyds.	= 8, 9, 12	Contrib. drain. area	= 18.000 ac



Hydrograph Summary Report

Hydraflow Hydrographs Extension for Autodesk® Civil 3D® by Autodesk, Inc. v2022

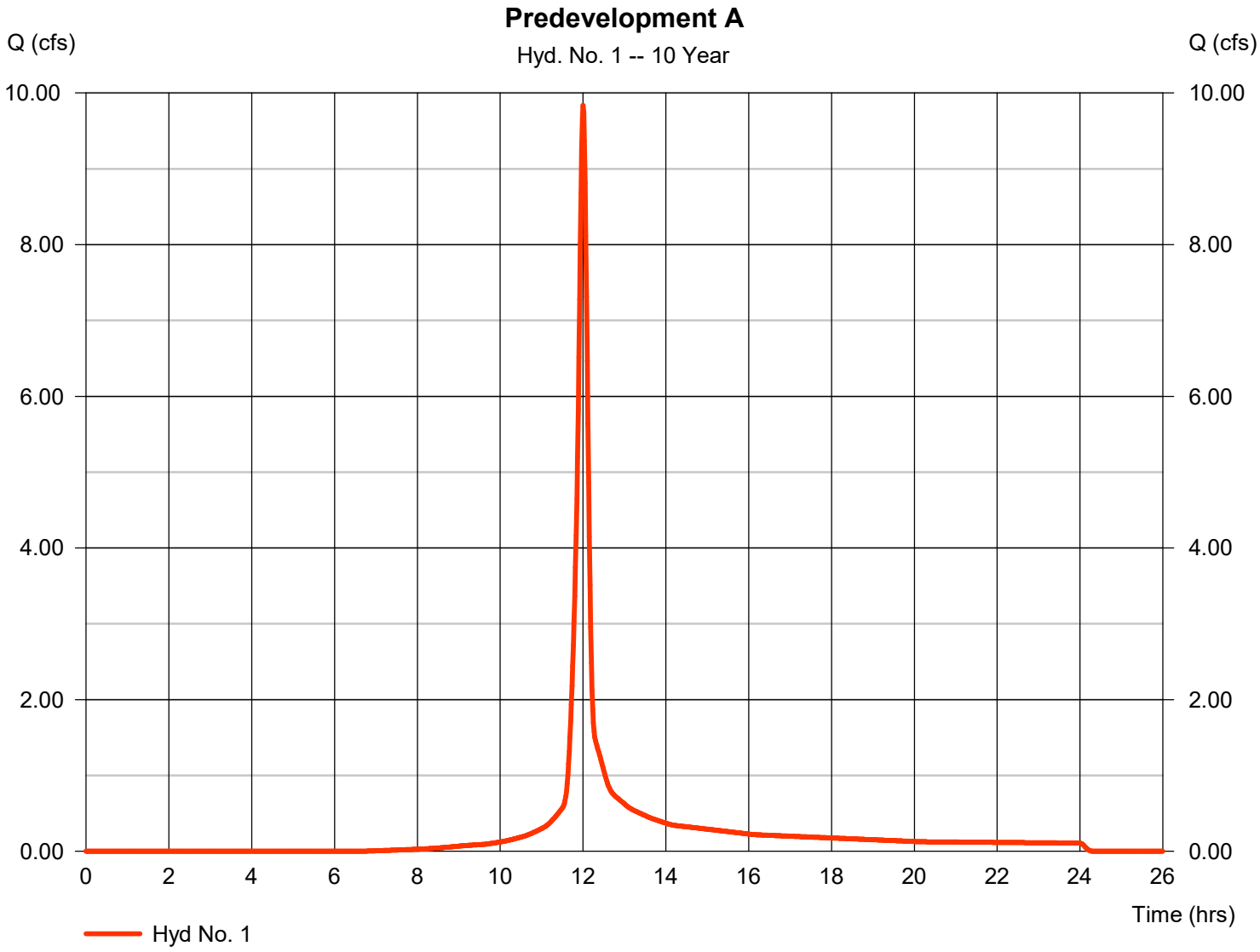
Hyd. No.	Hydrograph type (origin)	Peak flow (cfs)	Time interval (min)	Time to Peak (min)	Hyd. volume (cuft)	Inflow hyd(s)	Maximum elevation (ft)	Total strge used (cuft)	Hydrograph Description
1	SCS Runoff	9.835	1	720	23,982	------	------	------	Predevelopment A
2	SCS Runoff	4.898	1	720	12,058	------	------	------	Predevelopment B
3	SCS Runoff	1.637	1	716	3,197	------	------	------	Undetained A
4	SCS Runoff	11.80	1	720	30,425	------	------	------	Detained A
5	SCS Runoff	3.913	1	719	9,083	------	------	------	Undetained B
6	SCS Runoff	0.502	1	717	1,125	------	------	------	Undetained B2
7	Reservoir	2.696	1	732	30,423	4	1231.09	9,388	ADS Underground System
8	Combine	3.865	1	717	33,620	3, 7	------	------	Total Post-Development A
9	Combine	4.377	1	719	10,208	5, 6,	------	------	Total Post-Development B
10	SCS Runoff	71.53	1	722	192,376	------	------	------	Pre-development Offsite
11	Combine	85.55	1	722	228,416	1, 2, 10	------	------	Pre-Development 10%
12	SCS Runoff	71.53	1	722	192,376	------	------	------	Post-development Offsite
13	Combine	78.36	1	722	236,204	8, 9, 12	------	------	Post-Development 10%

Hydrograph Report

Hyd. No. 1

Predevelopment A

Hydrograph type	=	SCS Runoff	Peak discharge	=	9.835 cfs
Storm frequency	=	10 yrs	Time to peak	=	12.00 hrs
Time interval	=	1 min	Hyd. volume	=	23,982 cuft
Drainage area	=	1.790 ac	Curve number	=	79
Basin Slope	=	2.0 %	Hydraulic length	=	130 ft
Tc method	=	TR55	Time of conc. (Tc)	=	11.00 min
Total precip.	=	5.94 in	Distribution	=	Type II
Storm duration	=	24 hrs	Shape factor	=	484



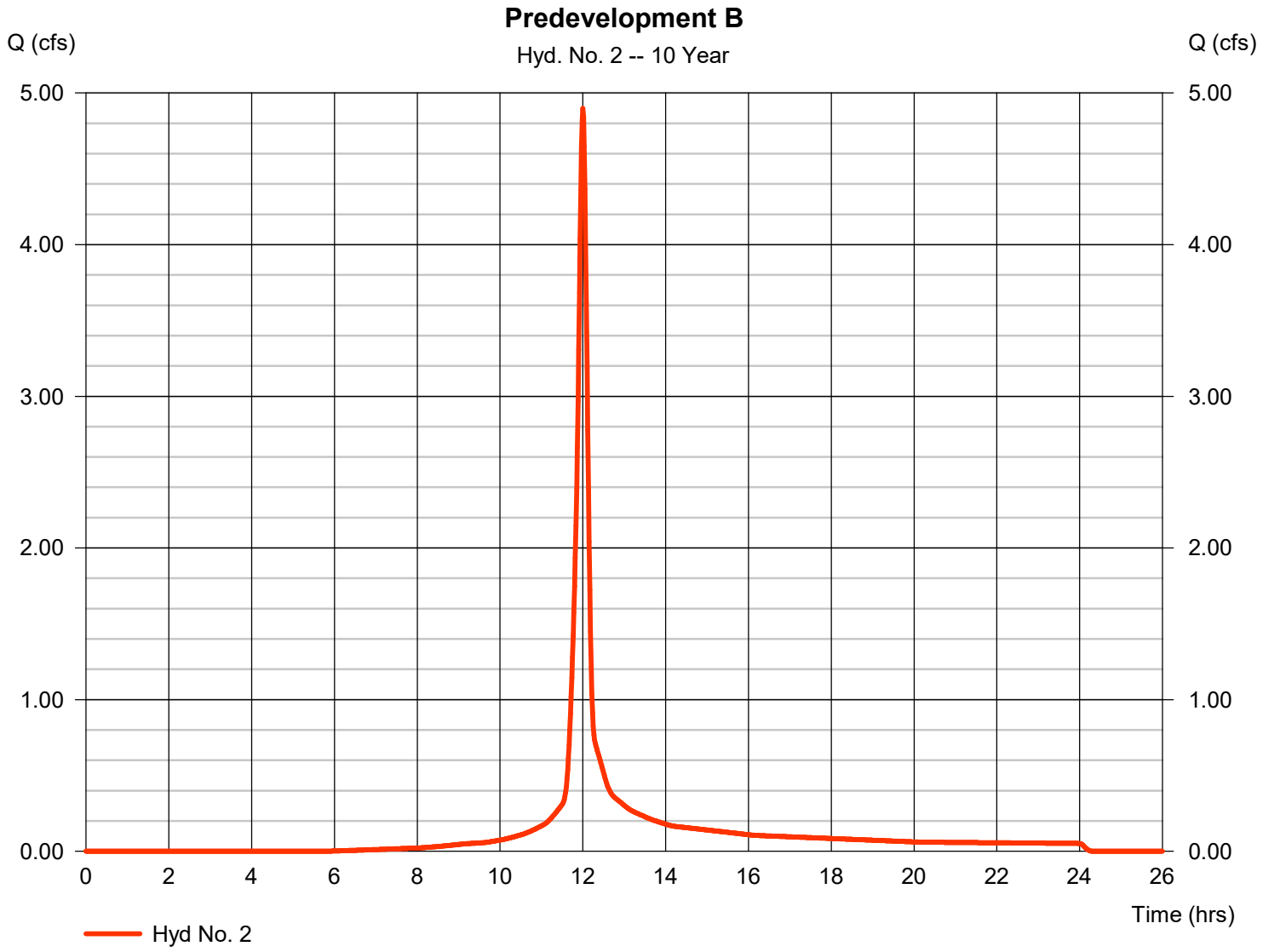
Hydrograph Report

Hyd. No. 2

Predevelopment B

Hydrograph type	=	SCS Runoff	Peak discharge	=	4.898 cfs
Storm frequency	=	10 yrs	Time to peak	=	12.00 hrs
Time interval	=	1 min	Hyd. volume	=	12,058 cuft
Drainage area	=	0.830 ac	Curve number	=	82*
Basin Slope	=	0.0 %	Hydraulic length	=	0 ft
Tc method	=	TR55	Time of conc. (Tc)	=	10.60 min
Total precip.	=	5.94 in	Distribution	=	Type II
Storm duration	=	24 hrs	Shape factor	=	484

* Composite (Area/CN) = [(0.120 x 98) + (0.710 x 79)] / 0.830



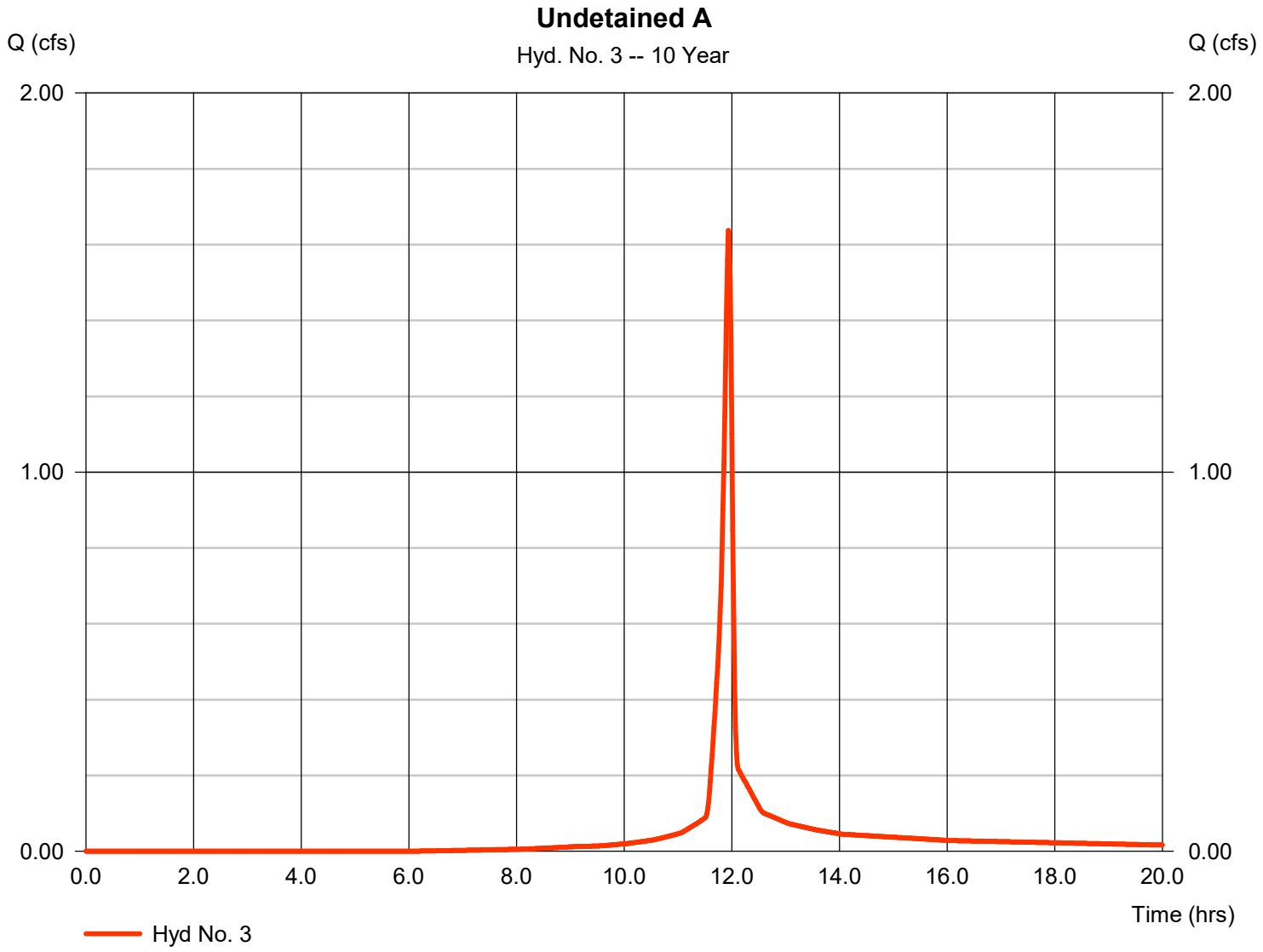
Hydrograph Report

Hyd. No. 3

Undetained A

Hydrograph type	=	SCS Runoff	Peak discharge	=	1.637 cfs
Storm frequency	=	10 yrs	Time to peak	=	11.93 hrs
Time interval	=	1 min	Hyd. volume	=	3,197 cuft
Drainage area	=	0.230 ac	Curve number	=	81*
Basin Slope	=	0.0 %	Hydraulic length	=	0 ft
Tc method	=	TR55	Time of conc. (Tc)	=	4.40 min
Total precip.	=	5.94 in	Distribution	=	Type II
Storm duration	=	24 hrs	Shape factor	=	484

* Composite (Area/CN) = [(0.200 x 79) + (0.030 x 98)] / 0.230



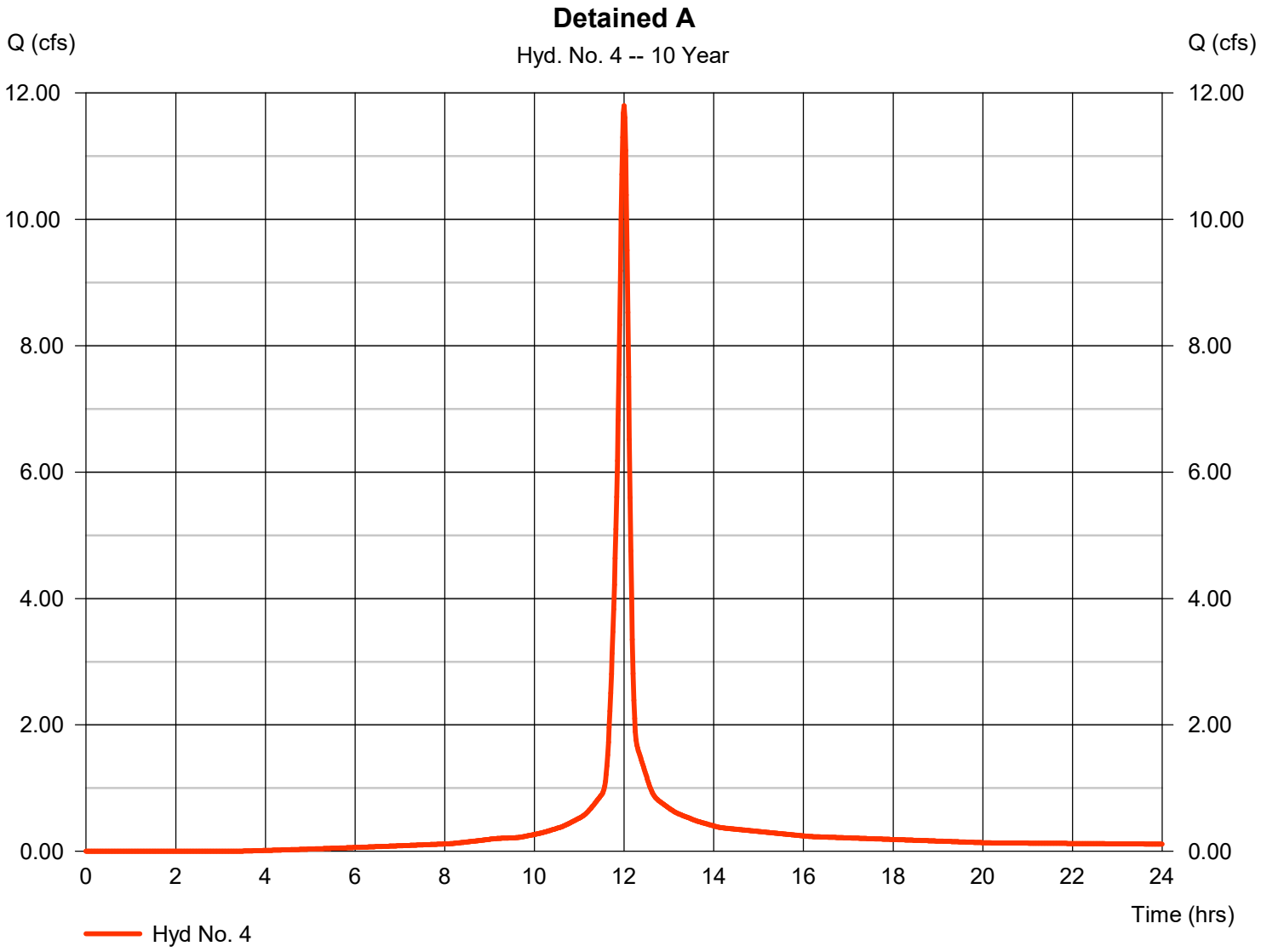
Hydrograph Report

Hyd. No. 4

Detained A

Hydrograph type	=	SCS Runoff	Peak discharge	=	11.80 cfs
Storm frequency	=	10 yrs	Time to peak	=	12.00 hrs
Time interval	=	1 min	Hyd. volume	=	30,425 cuft
Drainage area	=	1.720 ac	Curve number	=	90*
Basin Slope	=	0.0 %	Hydraulic length	=	0 ft
Tc method	=	TR55	Time of conc. (Tc)	=	10.40 min
Total precip.	=	5.94 in	Distribution	=	Type II
Storm duration	=	24 hrs	Shape factor	=	484

* Composite (Area/CN) = [(0.970 x 98) + (0.750 x 79)] / 1.720



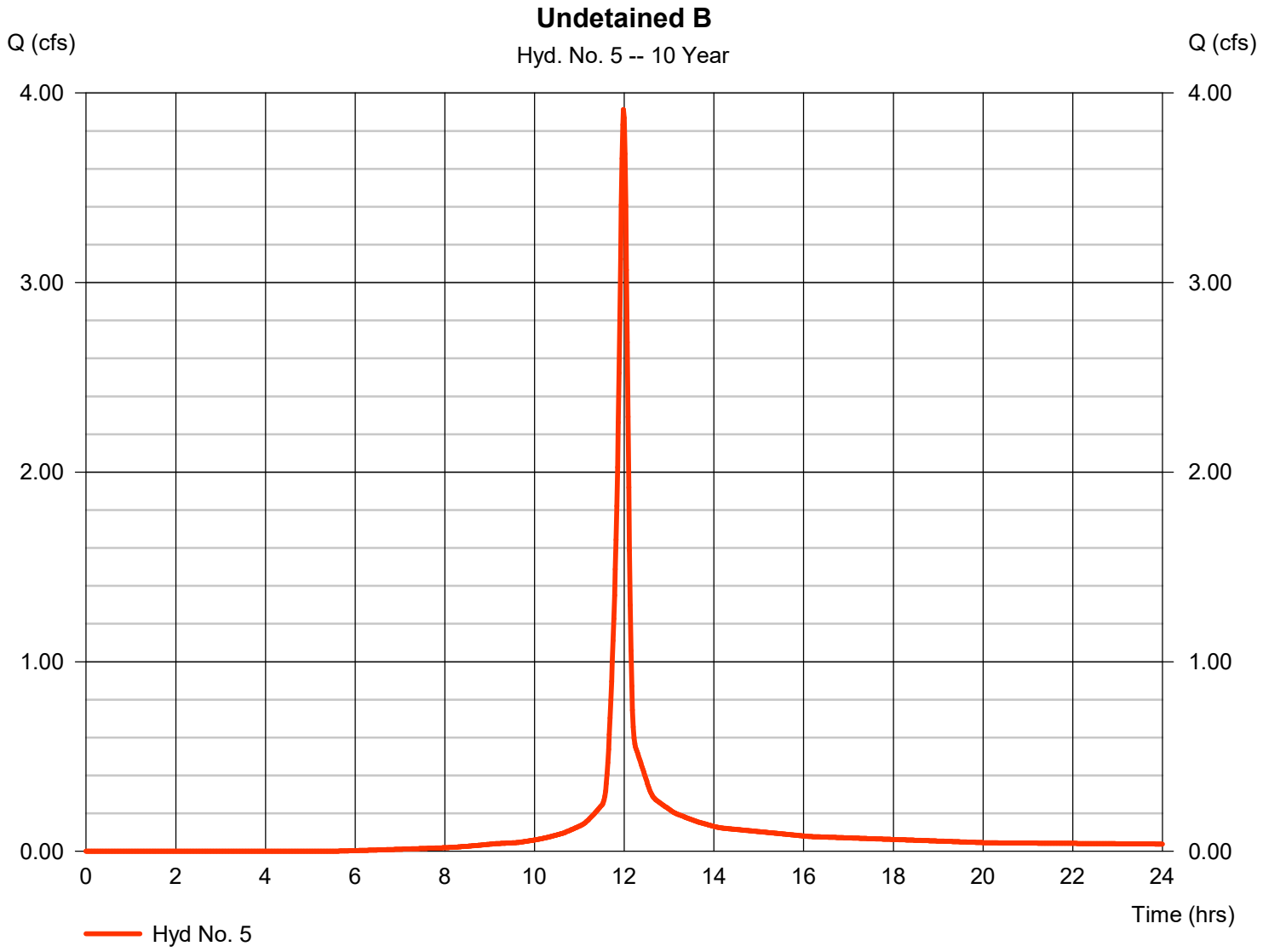
Hydrograph Report

Hyd. No. 5

Undetained B

Hydrograph type	=	SCS Runoff	Peak discharge	=	3.913 cfs
Storm frequency	=	10 yrs	Time to peak	=	11.98 hrs
Time interval	=	1 min	Hyd. volume	=	9,083 cuft
Drainage area	=	0.620 ac	Curve number	=	83*
Basin Slope	=	0.0 %	Hydraulic length	=	0 ft
Tc method	=	TR55	Time of conc. (Tc)	=	10.00 min
Total precip.	=	5.94 in	Distribution	=	Type II
Storm duration	=	24 hrs	Shape factor	=	484

* Composite (Area/CN) = [(0.130 x 98) + (0.490 x 79)] / 0.620



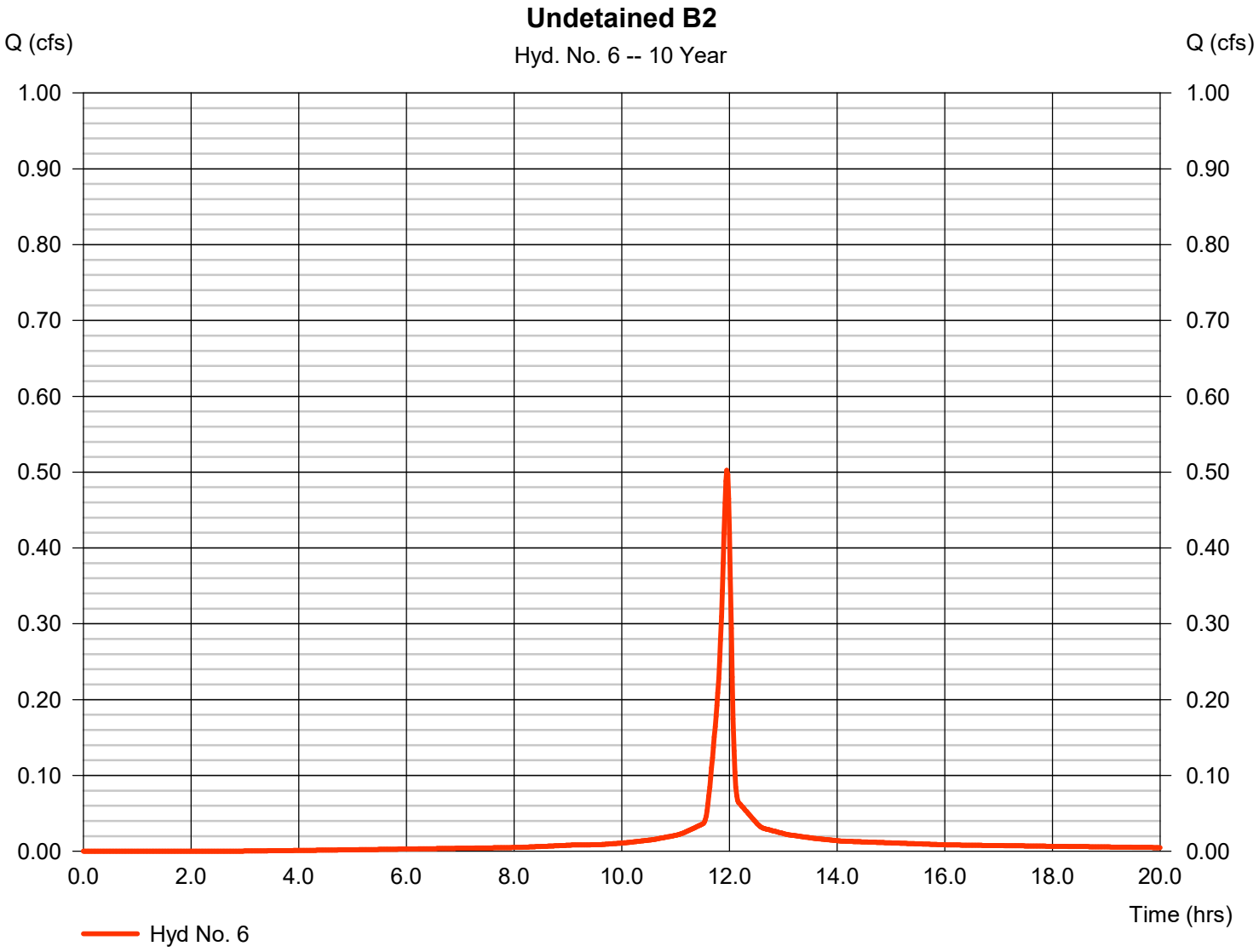
Hydrograph Report

Hyd. No. 6

Undetained B2

Hydrograph type	= SCS Runoff	Peak discharge	= 0.502 cfs
Storm frequency	= 10 yrs	Time to peak	= 11.95 hrs
Time interval	= 1 min	Hyd. volume	= 1,125 cuft
Drainage area	= 0.060 ac	Curve number	= 92*
Basin Slope	= 0.0 %	Hydraulic length	= 0 ft
Tc method	= User	Time of conc. (Tc)	= 5.00 min
Total precip.	= 5.94 in	Distribution	= Type II
Storm duration	= 24 hrs	Shape factor	= 484

* Composite (Area/CN) = [(0.020 x 79) + (0.040 x 98)] / 0.060



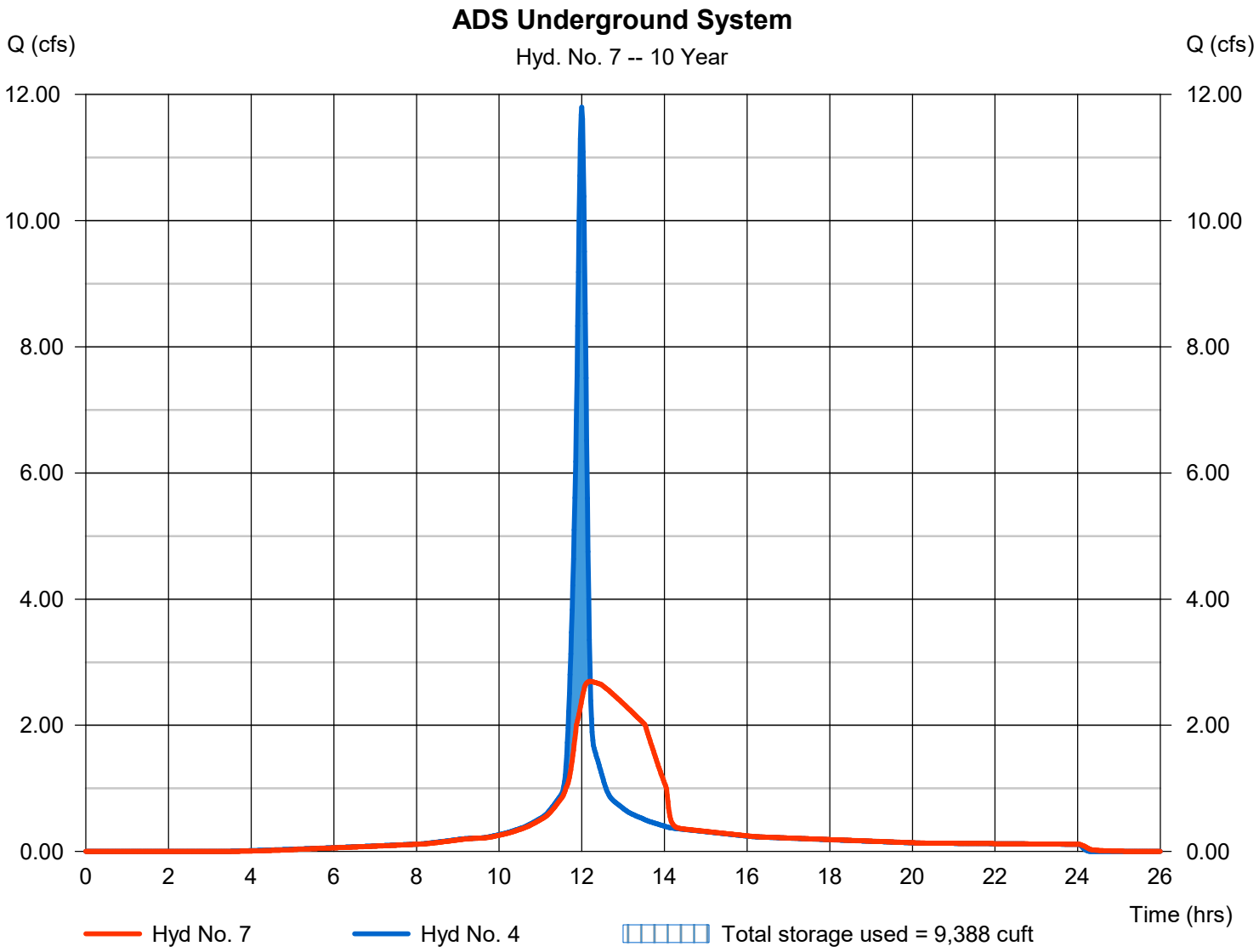
Hydrograph Report

Hyd. No. 7

ADS Underground System

Hydrograph type	=	Reservoir	Peak discharge	=	2.696 cfs
Storm frequency	=	10 yrs	Time to peak	=	12.20 hrs
Time interval	=	1 min	Hyd. volume	=	30,423 cuft
Inflow hyd. No.	=	4 - Detained A	Max. Elevation	=	1231.09 ft
Reservoir name	=	<New Pond>	Max. Storage	=	9,388 cuft

Storage Indication method used.

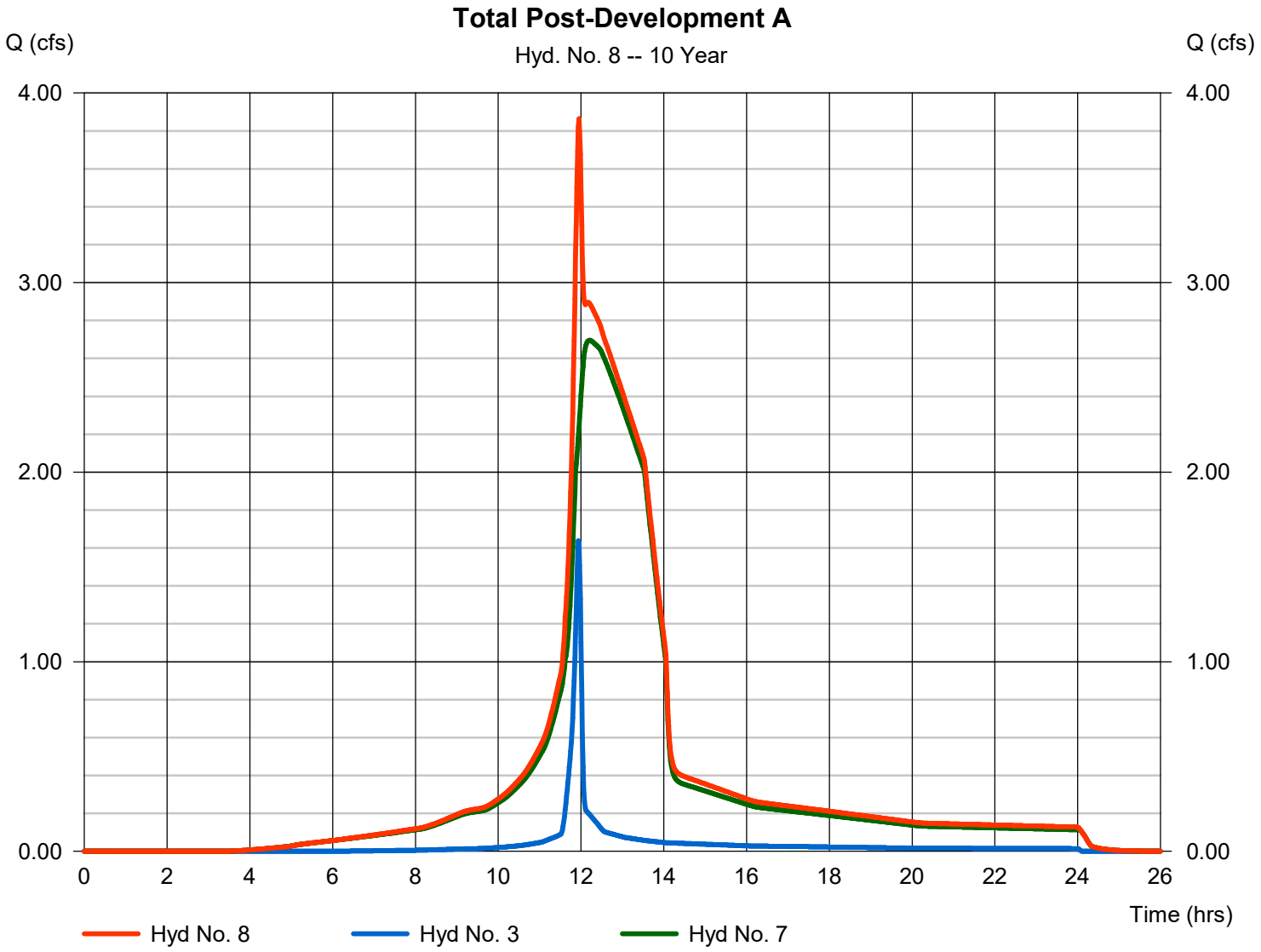


Hydrograph Report

Hyd. No. 8

Total Post-Development A

Hydrograph type	=	Combine	Peak discharge	=	3.865 cfs
Storm frequency	=	10 yrs	Time to peak	=	11.95 hrs
Time interval	=	1 min	Hyd. volume	=	33,620 cuft
Inflow hyds.	=	3, 7	Contrib. drain. area	=	0.230 ac

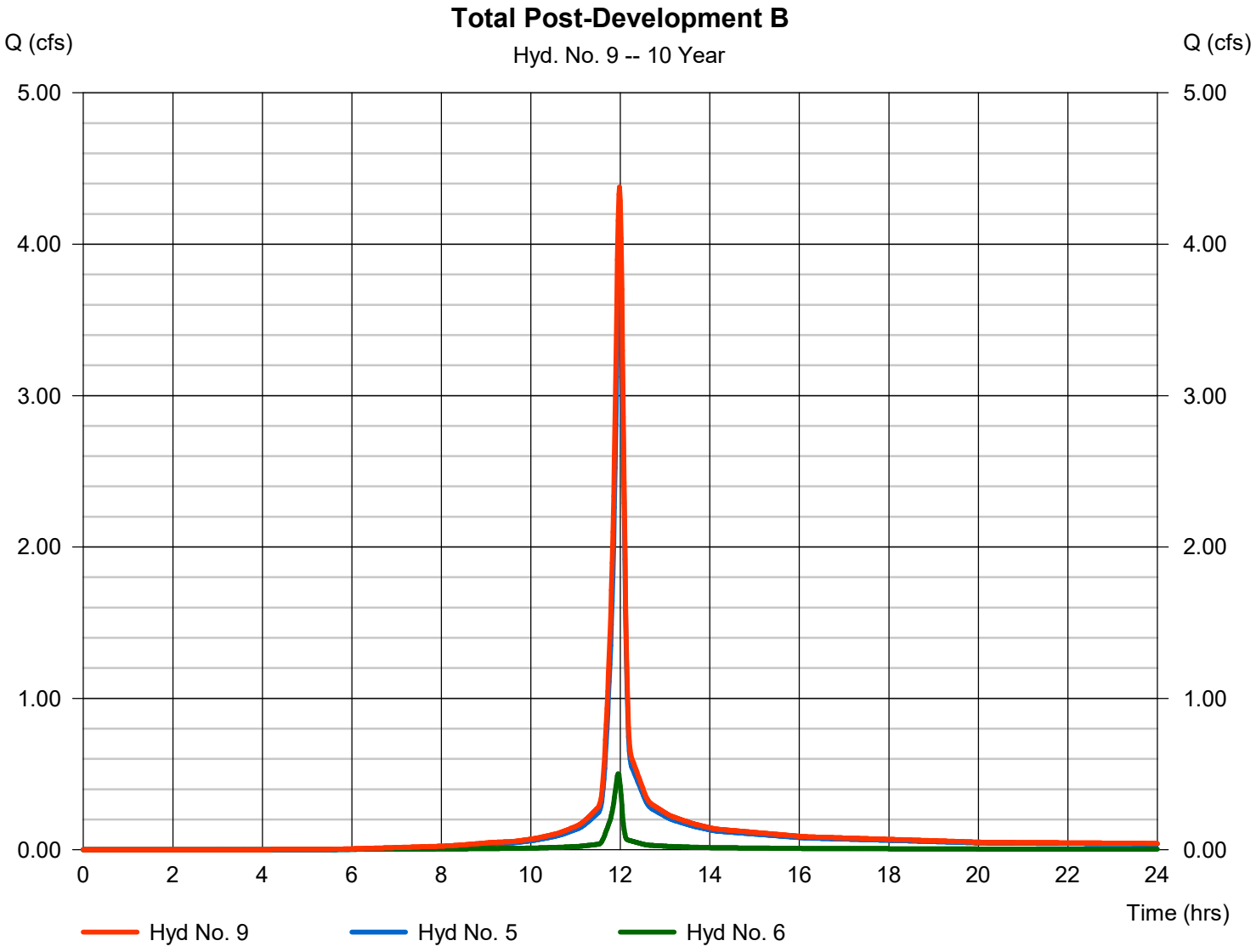


Hydrograph Report

Hyd. No. 9

Total Post-Development B

Hydrograph type	= Combine	Peak discharge	= 4.377 cfs
Storm frequency	= 10 yrs	Time to peak	= 11.98 hrs
Time interval	= 1 min	Hyd. volume	= 10,208 cuft
Inflow hyds.	= 5, 6	Contrib. drain. area	= 0.680 ac



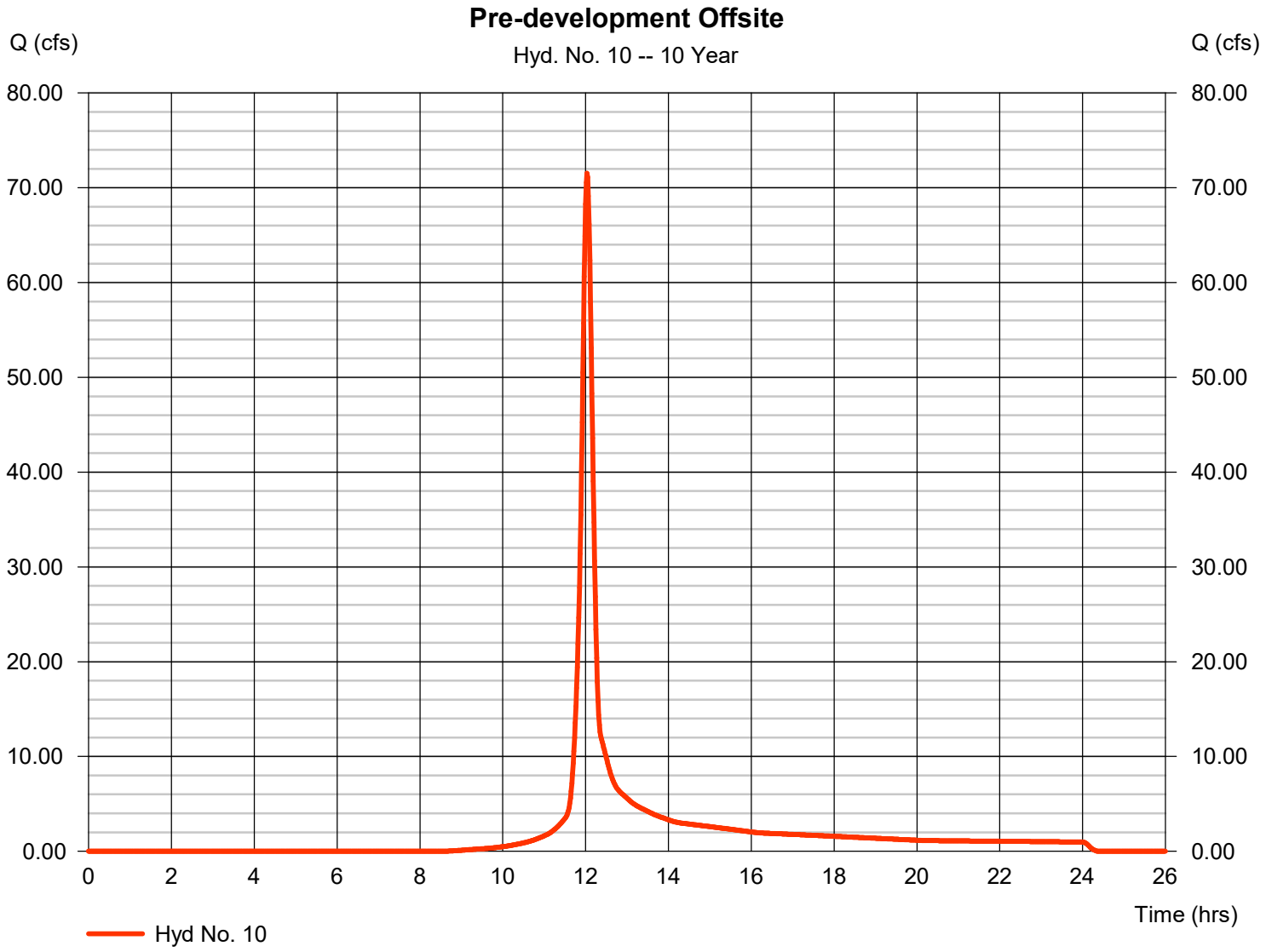
Hydrograph Report

Hyd. No. 10

Pre-development Offsite

Hydrograph type	= SCS Runoff	Peak discharge	= 71.53 cfs
Storm frequency	= 10 yrs	Time to peak	= 12.03 hrs
Time interval	= 1 min	Hyd. volume	= 192,376 cuft
Drainage area	= 18.000 ac	Curve number	= 72*
Basin Slope	= 0.0 %	Hydraulic length	= 0 ft
Tc method	= TR55	Time of conc. (Tc)	= 13.80 min
Total precip.	= 5.94 in	Distribution	= Type II
Storm duration	= 24 hrs	Shape factor	= 484

* Composite (Area/CN) = [(2.140 x 79) + (0.560 x 92) + (15.300 x 70)] / 18.000

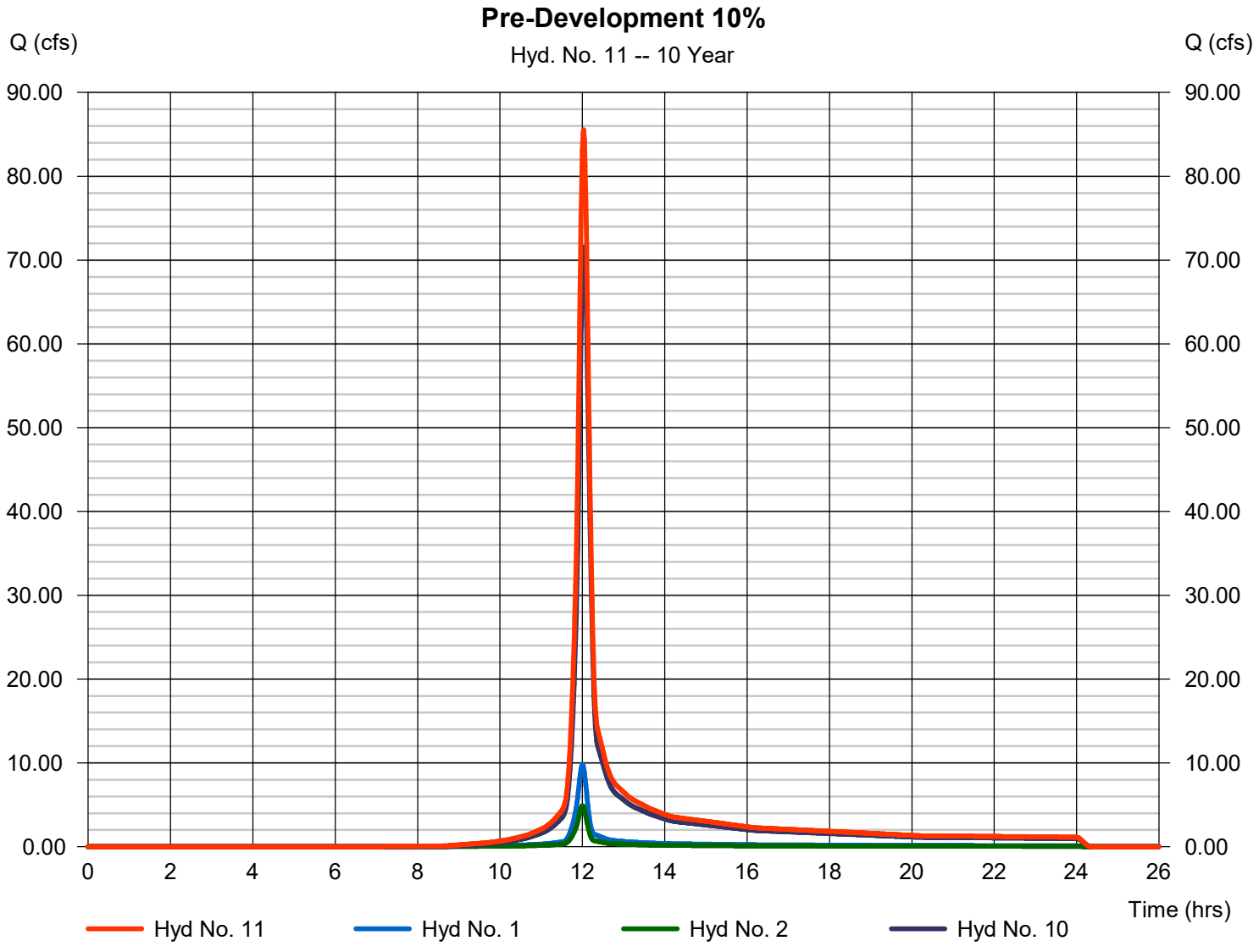


Hydrograph Report

Hyd. No. 11

Pre-Development 10%

Hydrograph type	= Combine	Peak discharge	= 85.55 cfs
Storm frequency	= 10 yrs	Time to peak	= 12.03 hrs
Time interval	= 1 min	Hyd. volume	= 228,416 cuft
Inflow hyds.	= 1, 2, 10	Contrib. drain. area	= 20.620 ac

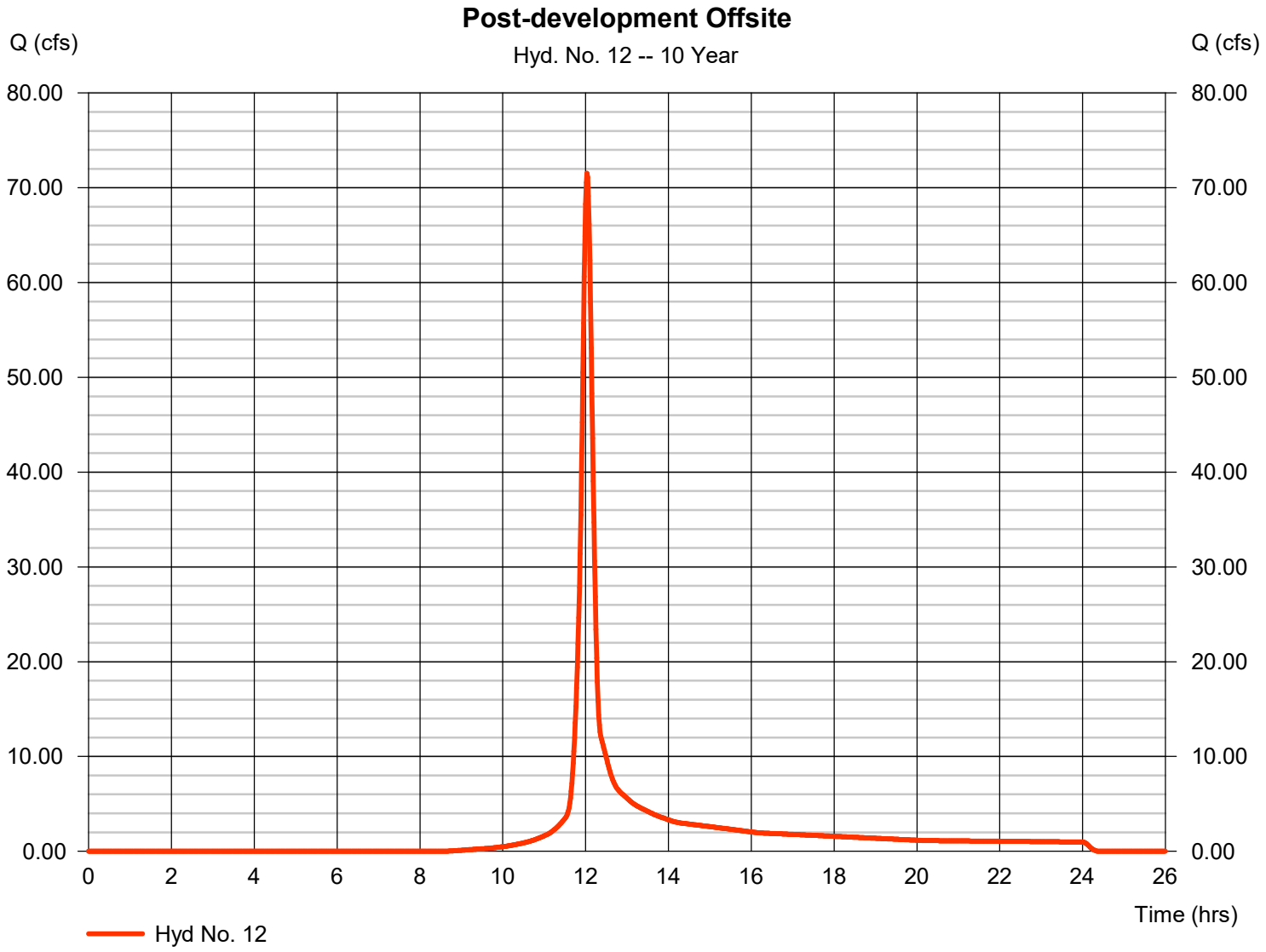


Hydrograph Report

Hyd. No. 12

Post-development Offsite

Hydrograph type	=	SCS Runoff	Peak discharge	=	71.53 cfs
Storm frequency	=	10 yrs	Time to peak	=	12.03 hrs
Time interval	=	1 min	Hyd. volume	=	192,376 cuft
Drainage area	=	18.000 ac	Curve number	=	72
Basin Slope	=	0.0 %	Hydraulic length	=	0 ft
Tc method	=	TR55	Time of conc. (Tc)	=	13.80 min
Total precip.	=	5.94 in	Distribution	=	Type II
Storm duration	=	24 hrs	Shape factor	=	484

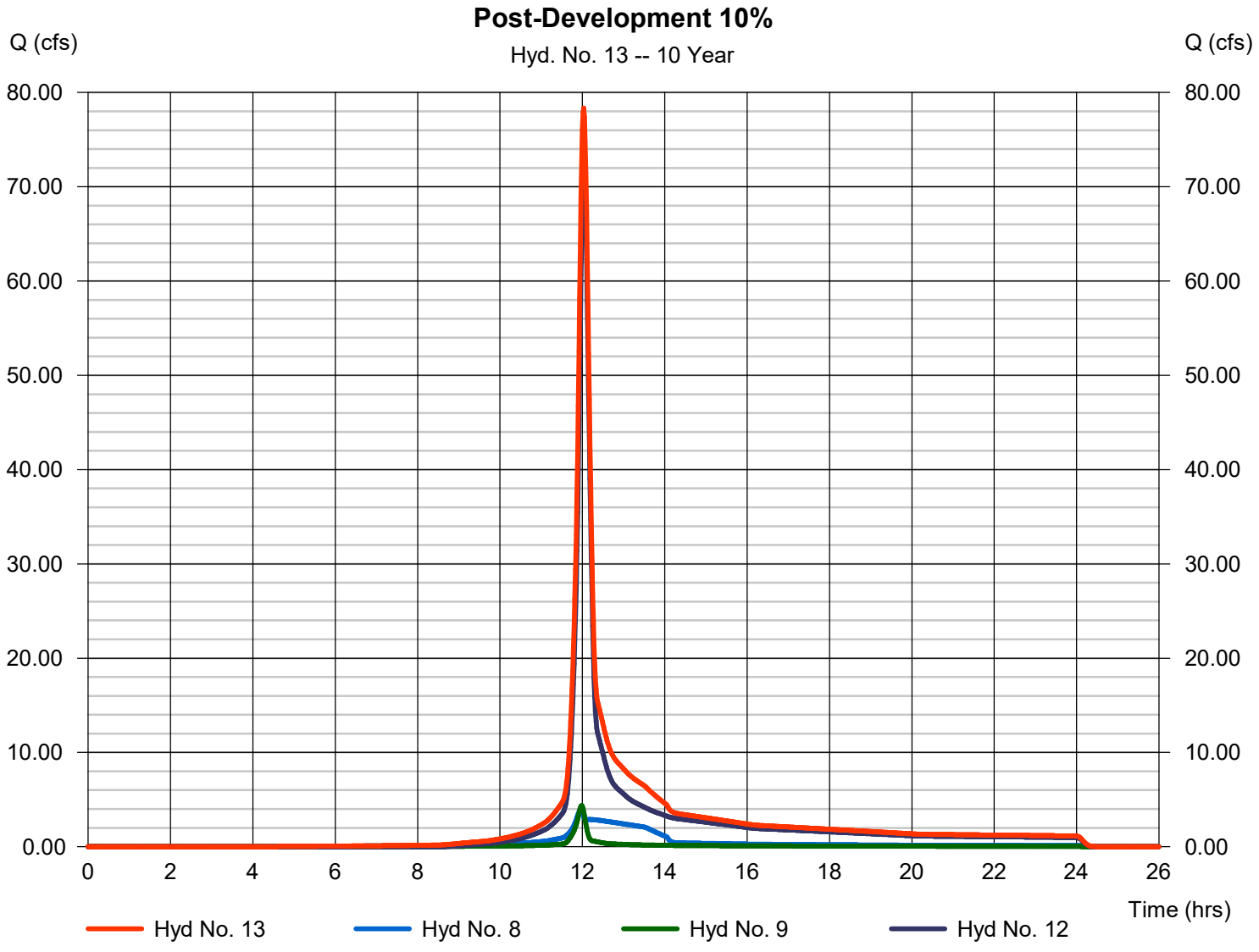


Hydrograph Report

Hyd. No. 13

Post-Development 10%

Hydrograph type	= Combine	Peak discharge	= 78.36 cfs
Storm frequency	= 10 yrs	Time to peak	= 12.03 hrs
Time interval	= 1 min	Hyd. volume	= 236,204 cuft
Inflow hyds.	= 8, 9, 12	Contrib. drain. area	= 18.000 ac



Hydrograph Summary Report

Hydraflow Hydrographs Extension for Autodesk® Civil 3D® by Autodesk, Inc. v2022

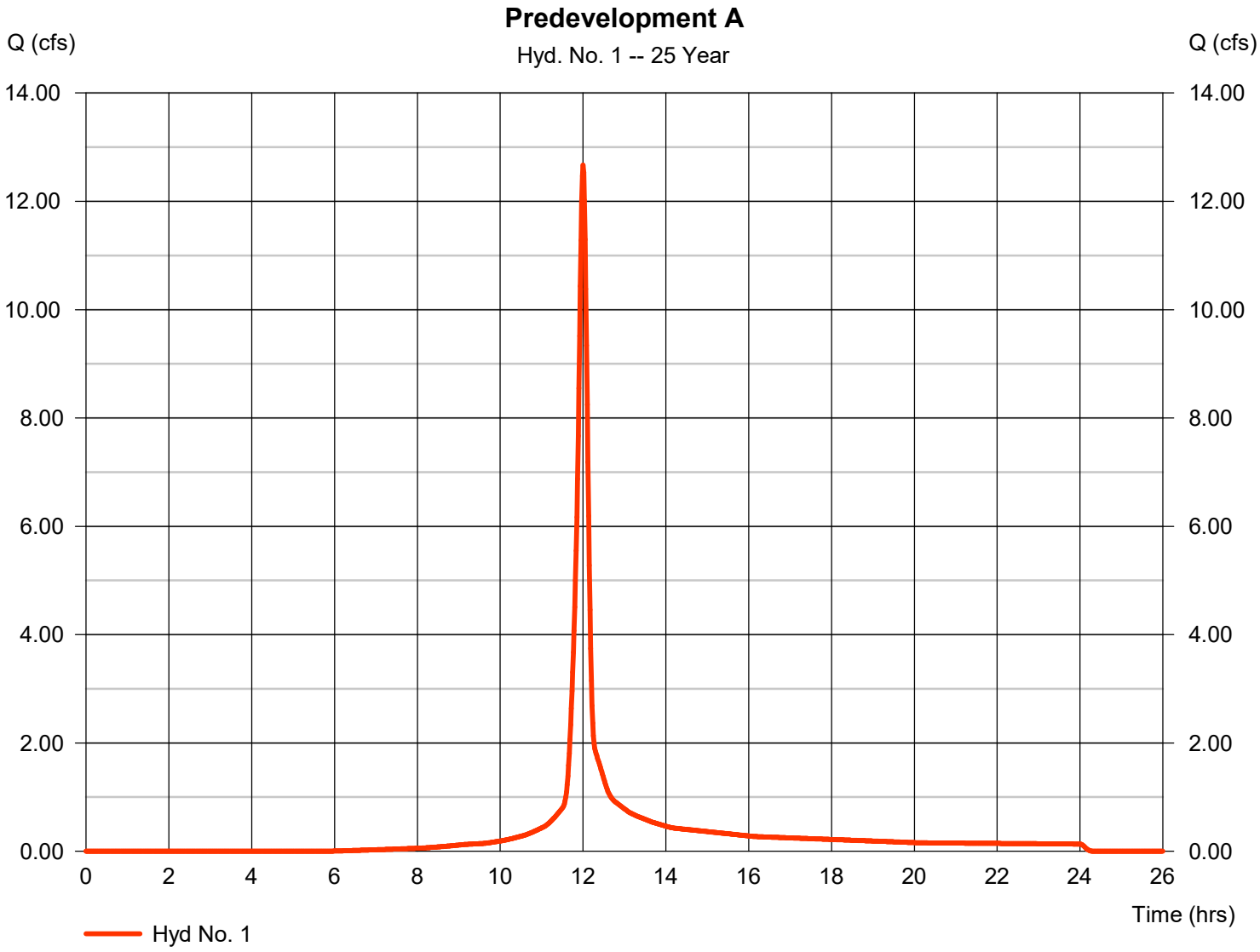
Hyd. No.	Hydrograph type (origin)	Peak flow (cfs)	Time interval (min)	Time to Peak (min)	Hyd. volume (cuft)	Inflow hyd(s)	Maximum elevation (ft)	Total strge used (cuft)	Hydrograph Description
1	SCS Runoff	12.67	1	720	31,168	------	------	------	Predevelopment A
2	SCS Runoff	6.219	1	720	15,479	------	------	------	Predevelopment B
3	SCS Runoff	2.082	1	716	4,121	------	------	------	Undetained A
4	SCS Runoff	14.49	1	720	37,879	------	------	------	Detained A
5	SCS Runoff	4.941	1	719	11,613	------	------	------	Undetained B
6	SCS Runoff	0.613	1	717	1,391	------	------	------	Undetained B2
7	Reservoir	2.836	1	733	37,878	4	1231.35	12,321	ADS Underground System
8	Combine	4.444	1	717	41,998	3, 7	------	------	Total Post-Development A
9	Combine	5.506	1	719	13,004	5, 6,	------	------	Total Post-Development B
10	SCS Runoff	95.91	1	722	258,004	------	------	------	Pre-development Offsite
11	Combine	113.81	1	722	304,651	1, 2, 10	------	------	Pre-Development 10%
12	SCS Runoff	95.91	1	722	258,004	------	------	------	Post-development Offsite
13	Combine	104.00	1	722	313,006	8, 9, 12	------	------	Post-Development 10%

Hydrograph Report

Hyd. No. 1

Predevelopment A

Hydrograph type	=	SCS Runoff	Peak discharge	=	12.67 cfs
Storm frequency	=	25 yrs	Time to peak	=	12.00 hrs
Time interval	=	1 min	Hyd. volume	=	31,168 cuft
Drainage area	=	1.790 ac	Curve number	=	79
Basin Slope	=	2.0 %	Hydraulic length	=	130 ft
Tc method	=	TR55	Time of conc. (Tc)	=	11.00 min
Total precip.	=	7.14 in	Distribution	=	Type II
Storm duration	=	24 hrs	Shape factor	=	484



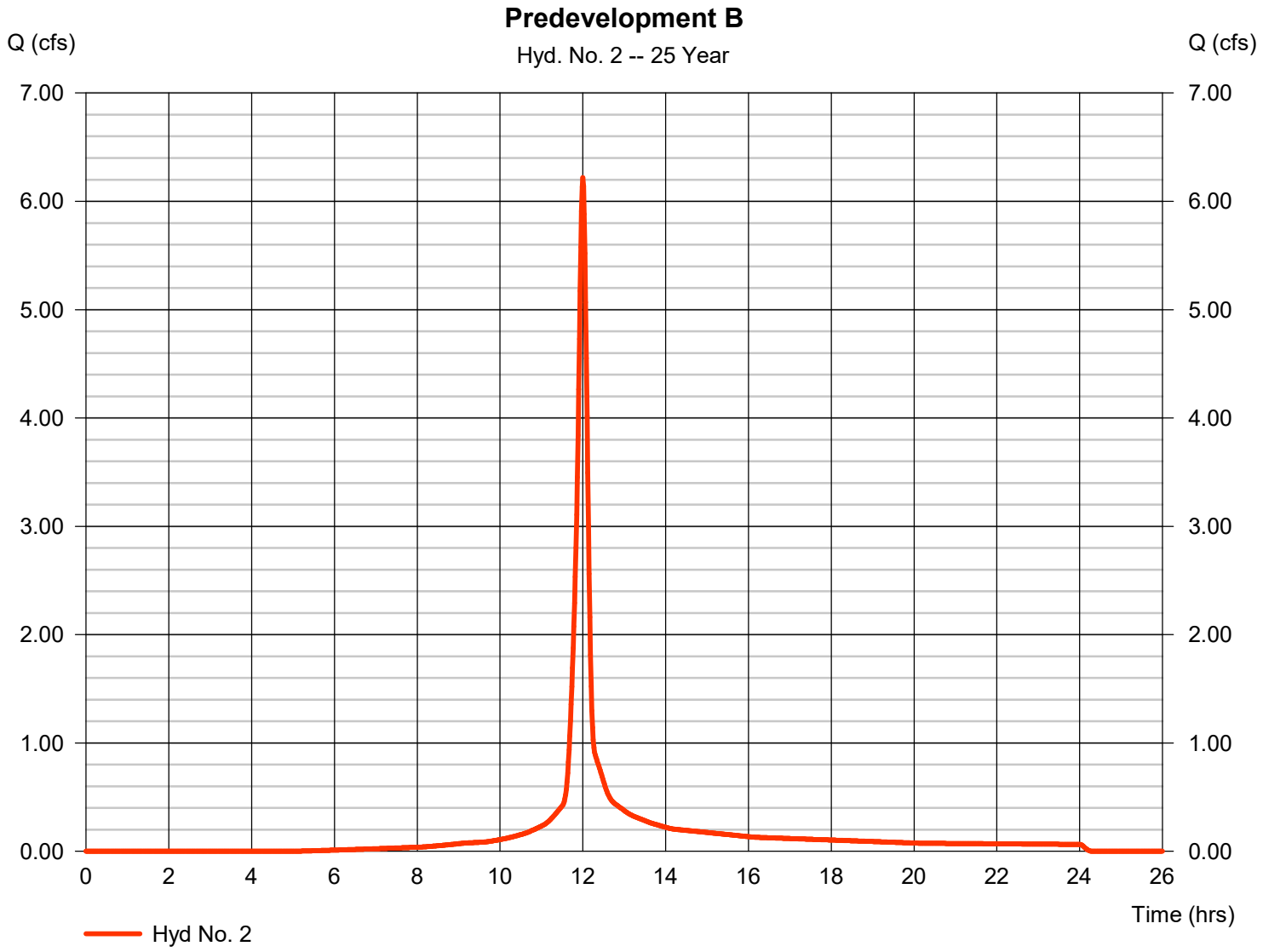
Hydrograph Report

Hyd. No. 2

Predevelopment B

Hydrograph type	=	SCS Runoff	Peak discharge	=	6.219 cfs
Storm frequency	=	25 yrs	Time to peak	=	12.00 hrs
Time interval	=	1 min	Hyd. volume	=	15,479 cuft
Drainage area	=	0.830 ac	Curve number	=	82*
Basin Slope	=	0.0 %	Hydraulic length	=	0 ft
Tc method	=	TR55	Time of conc. (Tc)	=	10.60 min
Total precip.	=	7.14 in	Distribution	=	Type II
Storm duration	=	24 hrs	Shape factor	=	484

* Composite (Area/CN) = [(0.120 x 98) + (0.710 x 79)] / 0.830



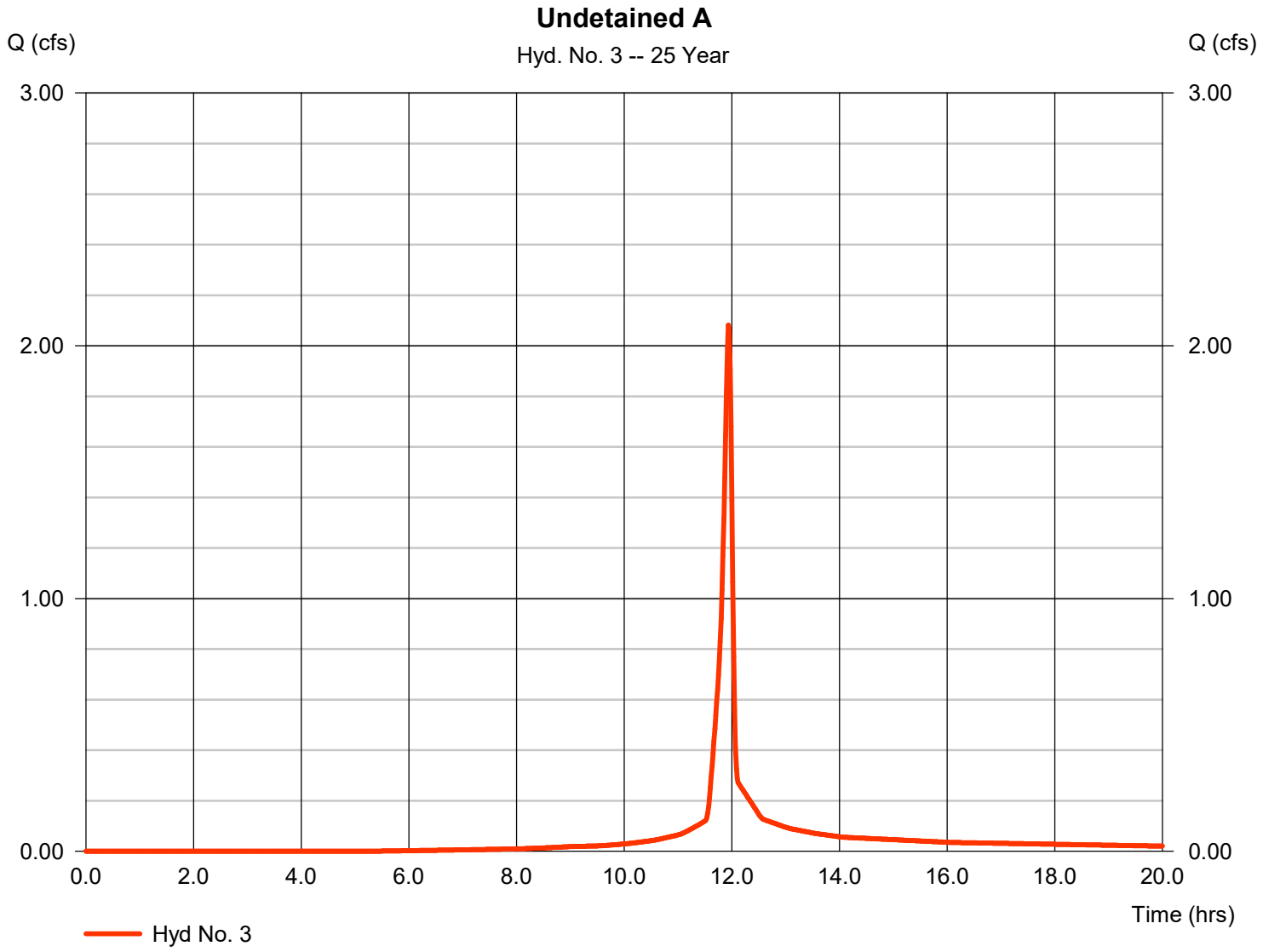
Hydrograph Report

Hyd. No. 3

Undetained A

Hydrograph type	=	SCS Runoff	Peak discharge	=	2.082 cfs
Storm frequency	=	25 yrs	Time to peak	=	11.93 hrs
Time interval	=	1 min	Hyd. volume	=	4,121 cuft
Drainage area	=	0.230 ac	Curve number	=	81*
Basin Slope	=	0.0 %	Hydraulic length	=	0 ft
Tc method	=	TR55	Time of conc. (Tc)	=	4.40 min
Total precip.	=	7.14 in	Distribution	=	Type II
Storm duration	=	24 hrs	Shape factor	=	484

* Composite (Area/CN) = [(0.200 x 79) + (0.030 x 98)] / 0.230



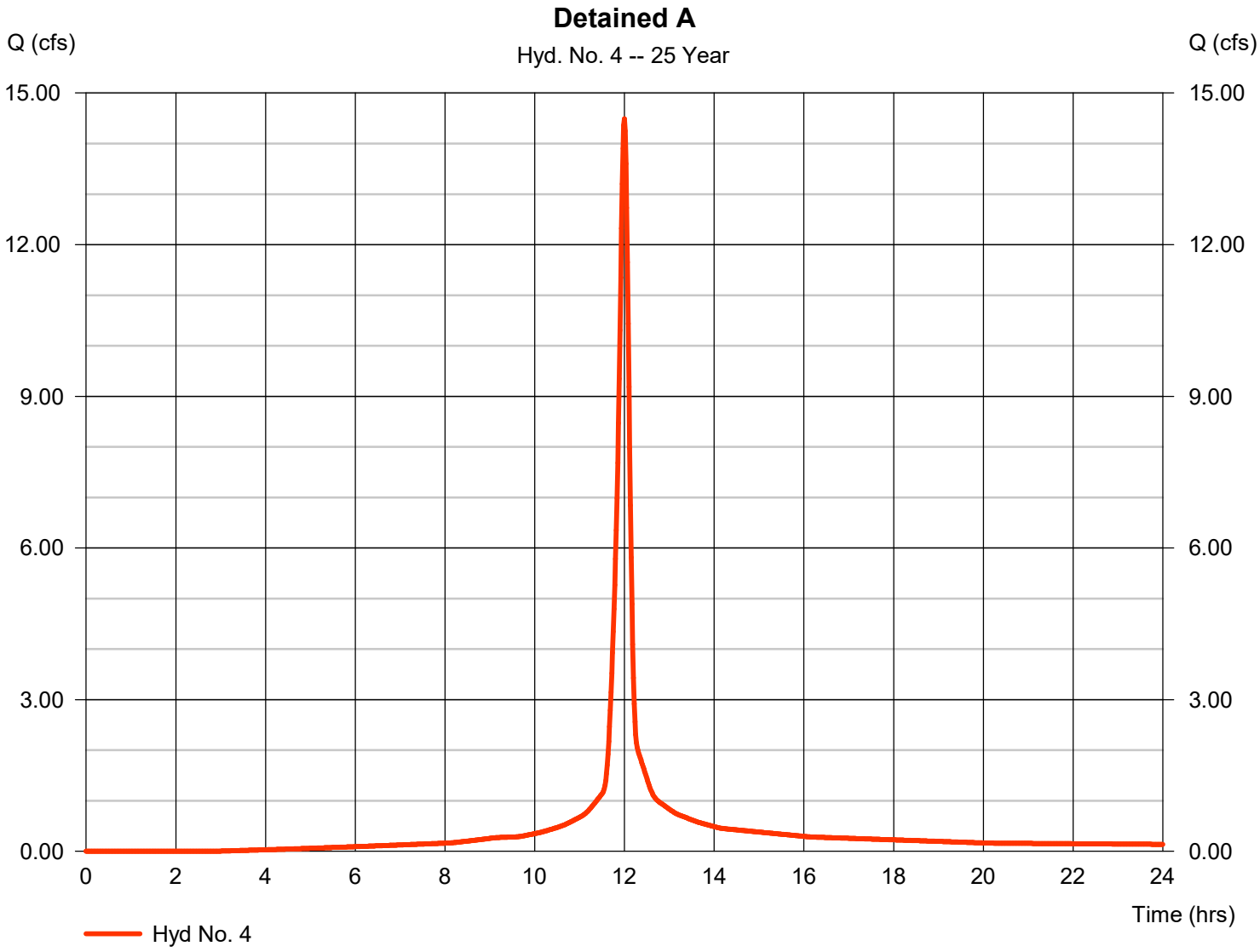
Hydrograph Report

Hyd. No. 4

Detained A

Hydrograph type	=	SCS Runoff	Peak discharge	=	14.49 cfs
Storm frequency	=	25 yrs	Time to peak	=	12.00 hrs
Time interval	=	1 min	Hyd. volume	=	37,879 cuft
Drainage area	=	1.720 ac	Curve number	=	90*
Basin Slope	=	0.0 %	Hydraulic length	=	0 ft
Tc method	=	TR55	Time of conc. (Tc)	=	10.40 min
Total precip.	=	7.14 in	Distribution	=	Type II
Storm duration	=	24 hrs	Shape factor	=	484

* Composite (Area/CN) = [(0.970 x 98) + (0.750 x 79)] / 1.720



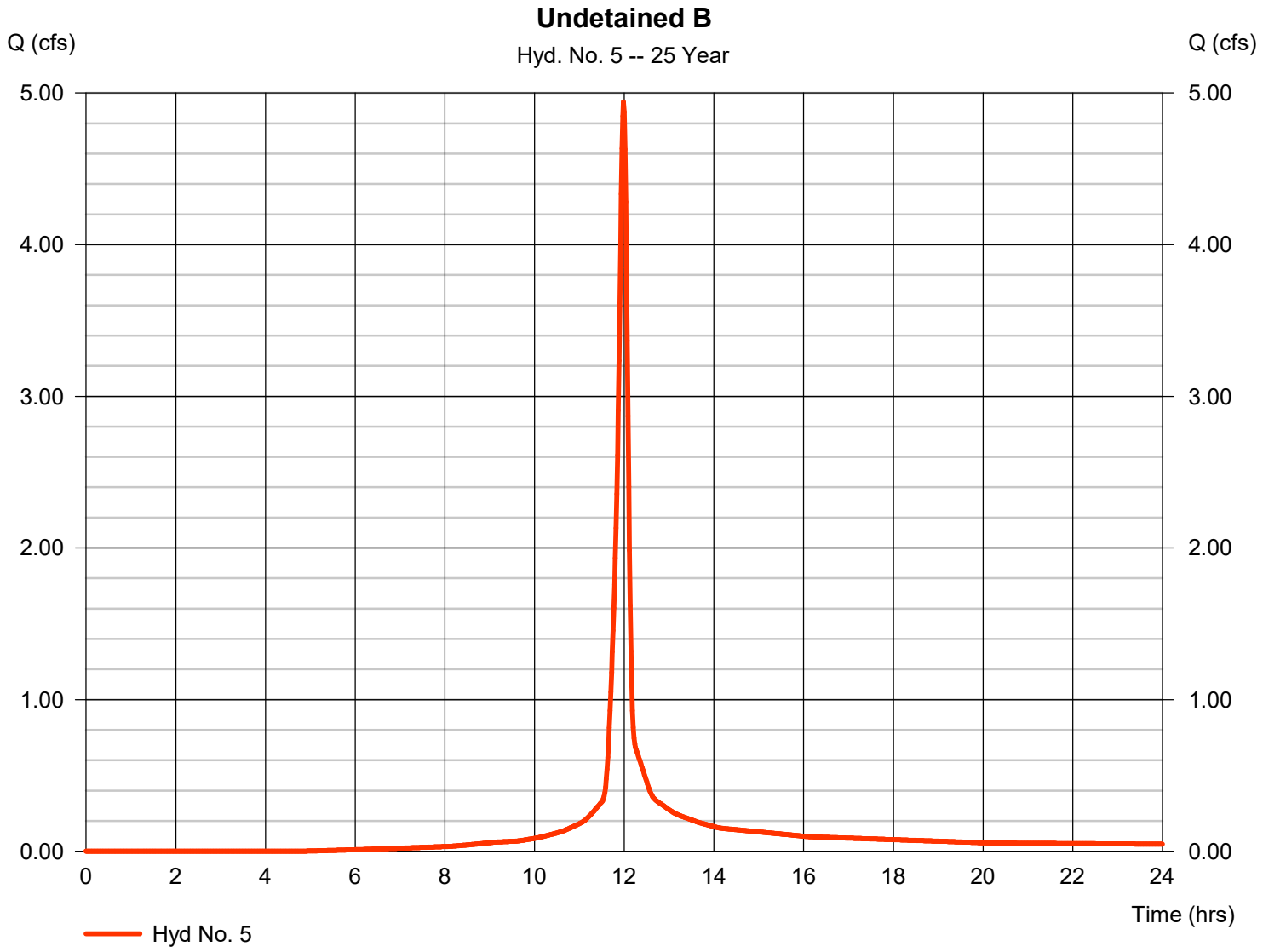
Hydrograph Report

Hyd. No. 5

Undetained B

Hydrograph type	=	SCS Runoff	Peak discharge	=	4.941 cfs
Storm frequency	=	25 yrs	Time to peak	=	11.98 hrs
Time interval	=	1 min	Hyd. volume	=	11,613 cuft
Drainage area	=	0.620 ac	Curve number	=	83*
Basin Slope	=	0.0 %	Hydraulic length	=	0 ft
Tc method	=	TR55	Time of conc. (Tc)	=	10.00 min
Total precip.	=	7.14 in	Distribution	=	Type II
Storm duration	=	24 hrs	Shape factor	=	484

* Composite (Area/CN) = [(0.130 x 98) + (0.490 x 79)] / 0.620



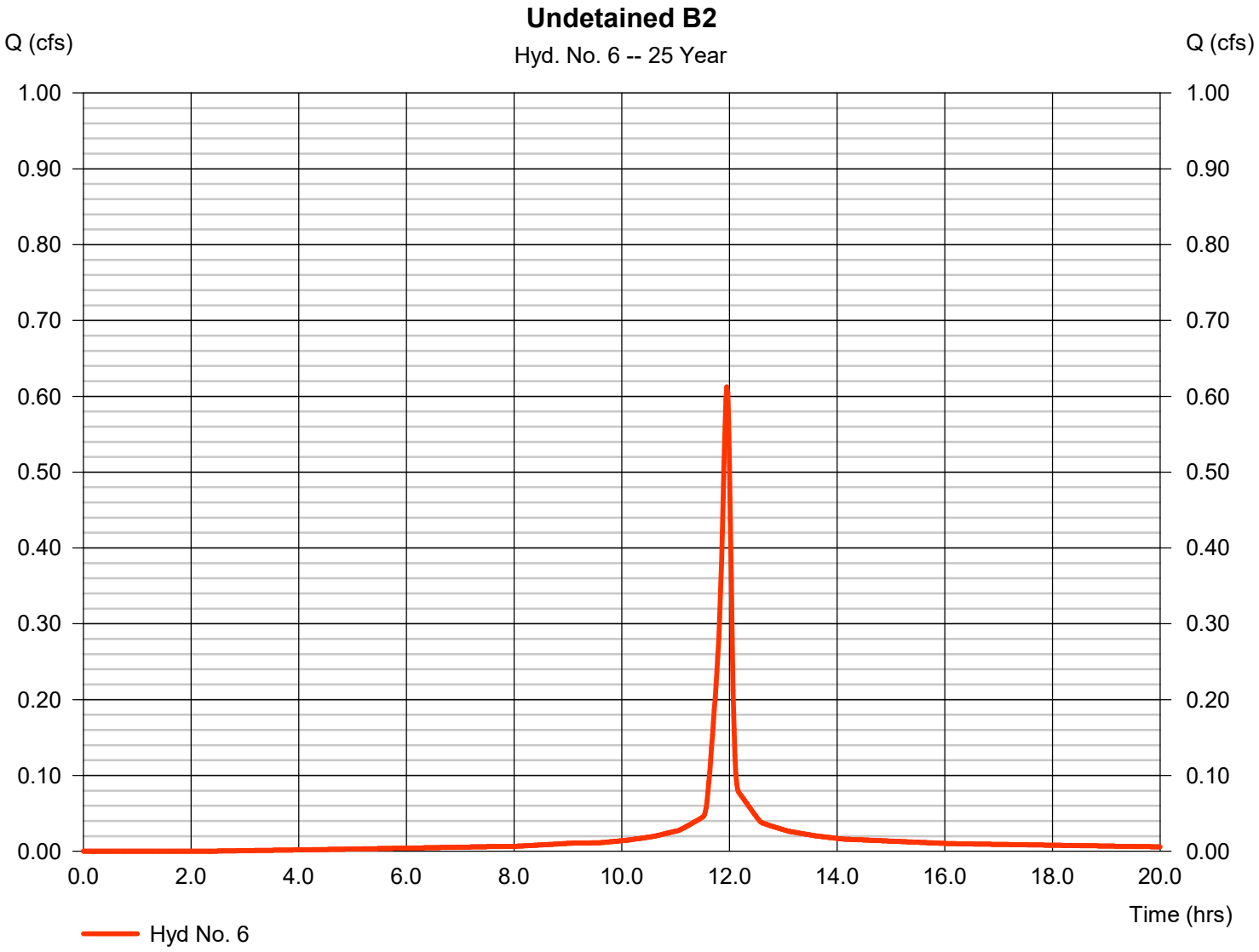
Hydrograph Report

Hyd. No. 6

Undetained B2

Hydrograph type	=	SCS Runoff	Peak discharge	=	0.613 cfs
Storm frequency	=	25 yrs	Time to peak	=	11.95 hrs
Time interval	=	1 min	Hyd. volume	=	1,391 cuft
Drainage area	=	0.060 ac	Curve number	=	92*
Basin Slope	=	0.0 %	Hydraulic length	=	0 ft
Tc method	=	User	Time of conc. (Tc)	=	5.00 min
Total precip.	=	7.14 in	Distribution	=	Type II
Storm duration	=	24 hrs	Shape factor	=	484

* Composite (Area/CN) = [(0.020 x 79) + (0.040 x 98)] / 0.060



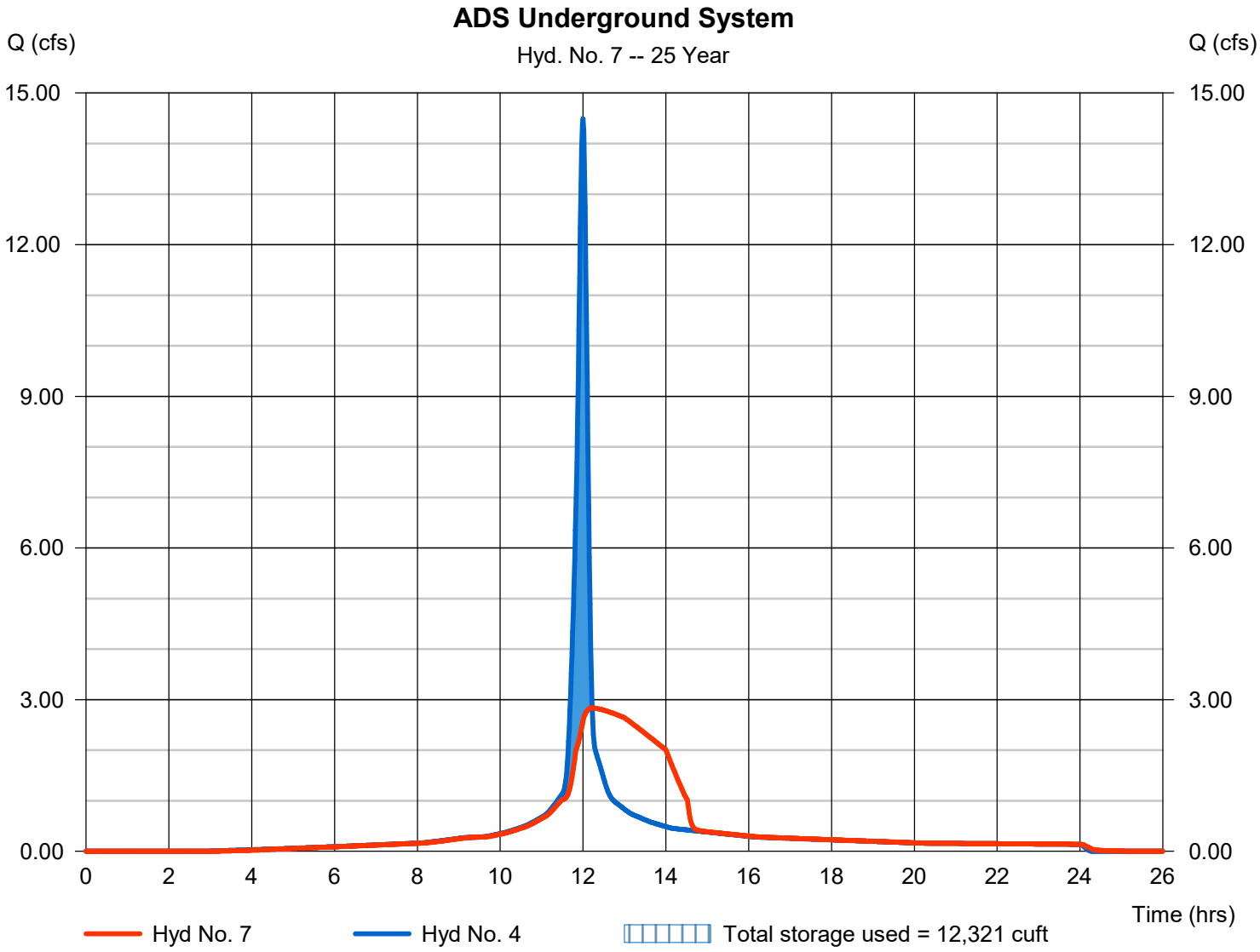
Hydrograph Report

Hyd. No. 7

ADS Underground System

Hydrograph type	=	Reservoir	Peak discharge	=	2.836 cfs
Storm frequency	=	25 yrs	Time to peak	=	12.22 hrs
Time interval	=	1 min	Hyd. volume	=	37,878 cuft
Inflow hyd. No.	=	4 - Detained A	Max. Elevation	=	1231.35 ft
Reservoir name	=	<New Pond>	Max. Storage	=	12,321 cuft

Storage Indication method used.

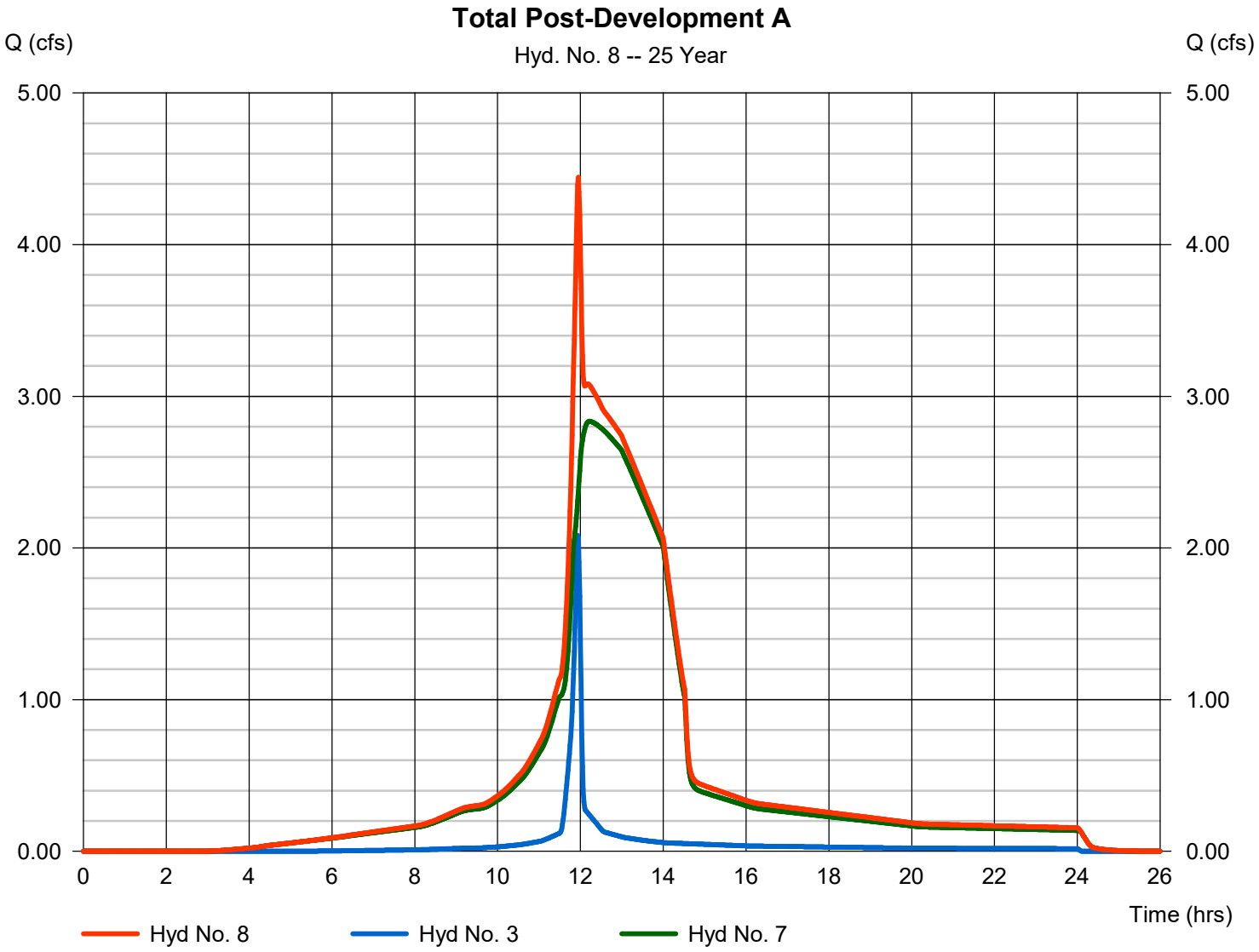


Hydrograph Report

Hyd. No. 8

Total Post-Development A

Hydrograph type	= Combine	Peak discharge	= 4.444 cfs
Storm frequency	= 25 yrs	Time to peak	= 11.95 hrs
Time interval	= 1 min	Hyd. volume	= 41,998 cuft
Inflow hyds.	= 3, 7	Contrib. drain. area	= 0.230 ac

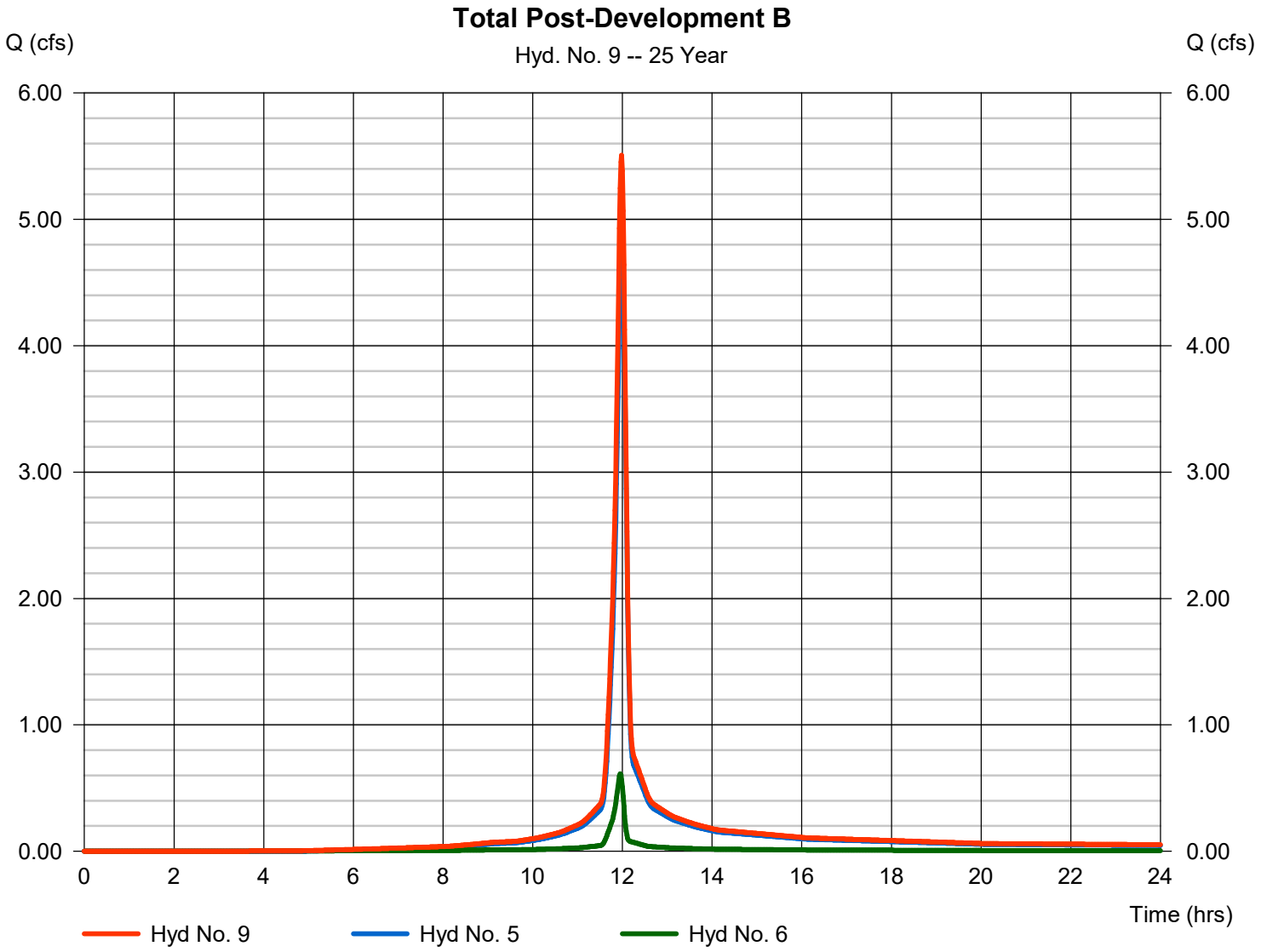


Hydrograph Report

Hyd. No. 9

Total Post-Development B

Hydrograph type	= Combine	Peak discharge	= 5.506 cfs
Storm frequency	= 25 yrs	Time to peak	= 11.98 hrs
Time interval	= 1 min	Hyd. volume	= 13,004 cuft
Inflow hyds.	= 5, 6	Contrib. drain. area	= 0.680 ac



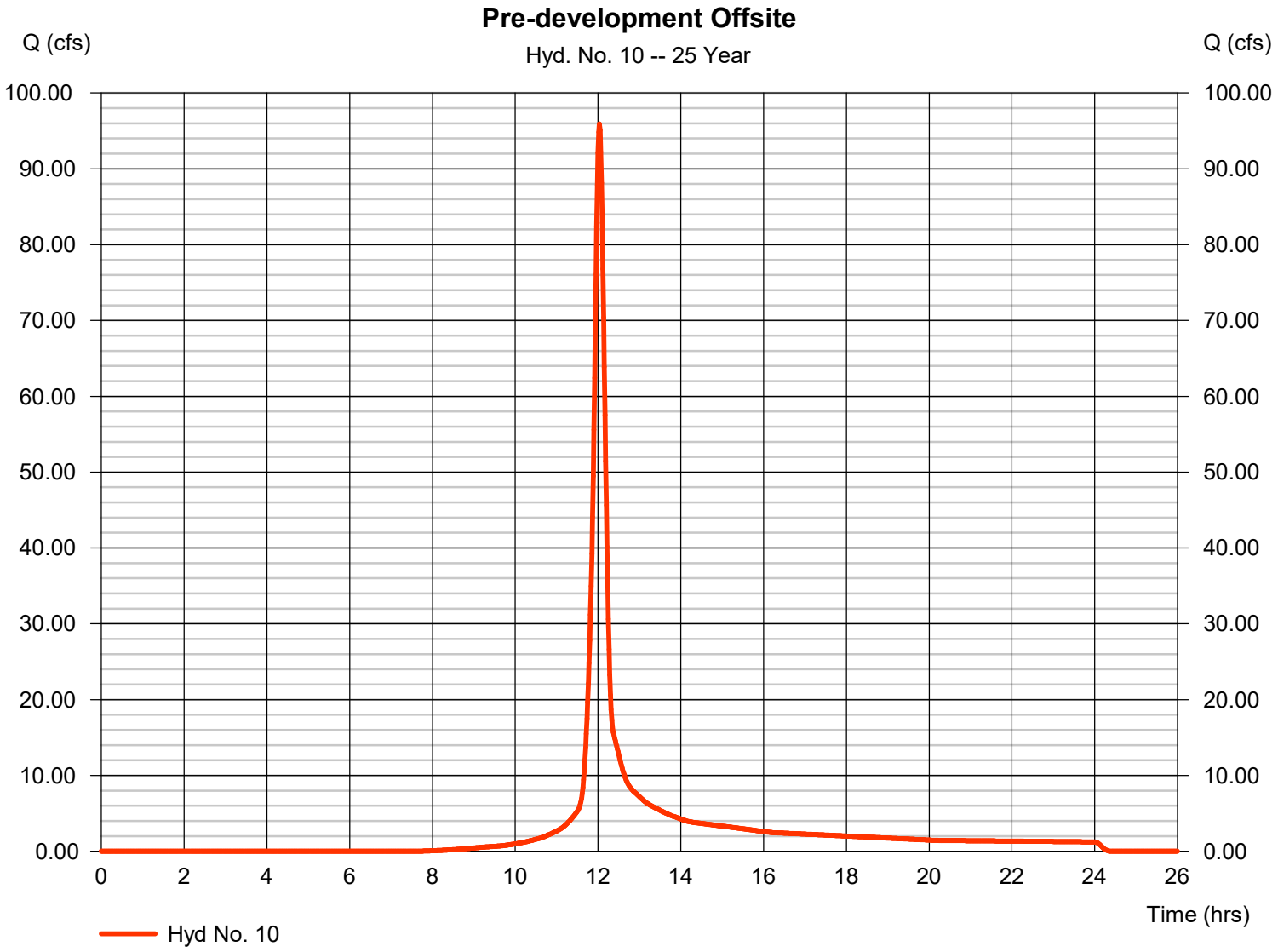
Hydrograph Report

Hyd. No. 10

Pre-development Offsite

Hydrograph type	=	SCS Runoff	Peak discharge	=	95.91 cfs
Storm frequency	=	25 yrs	Time to peak	=	12.03 hrs
Time interval	=	1 min	Hyd. volume	=	258,004 cuft
Drainage area	=	18.000 ac	Curve number	=	72*
Basin Slope	=	0.0 %	Hydraulic length	=	0 ft
Tc method	=	TR55	Time of conc. (Tc)	=	13.80 min
Total precip.	=	7.14 in	Distribution	=	Type II
Storm duration	=	24 hrs	Shape factor	=	484

* Composite (Area/CN) = [(2.140 x 79) + (0.560 x 92) + (15.300 x 70)] / 18.000

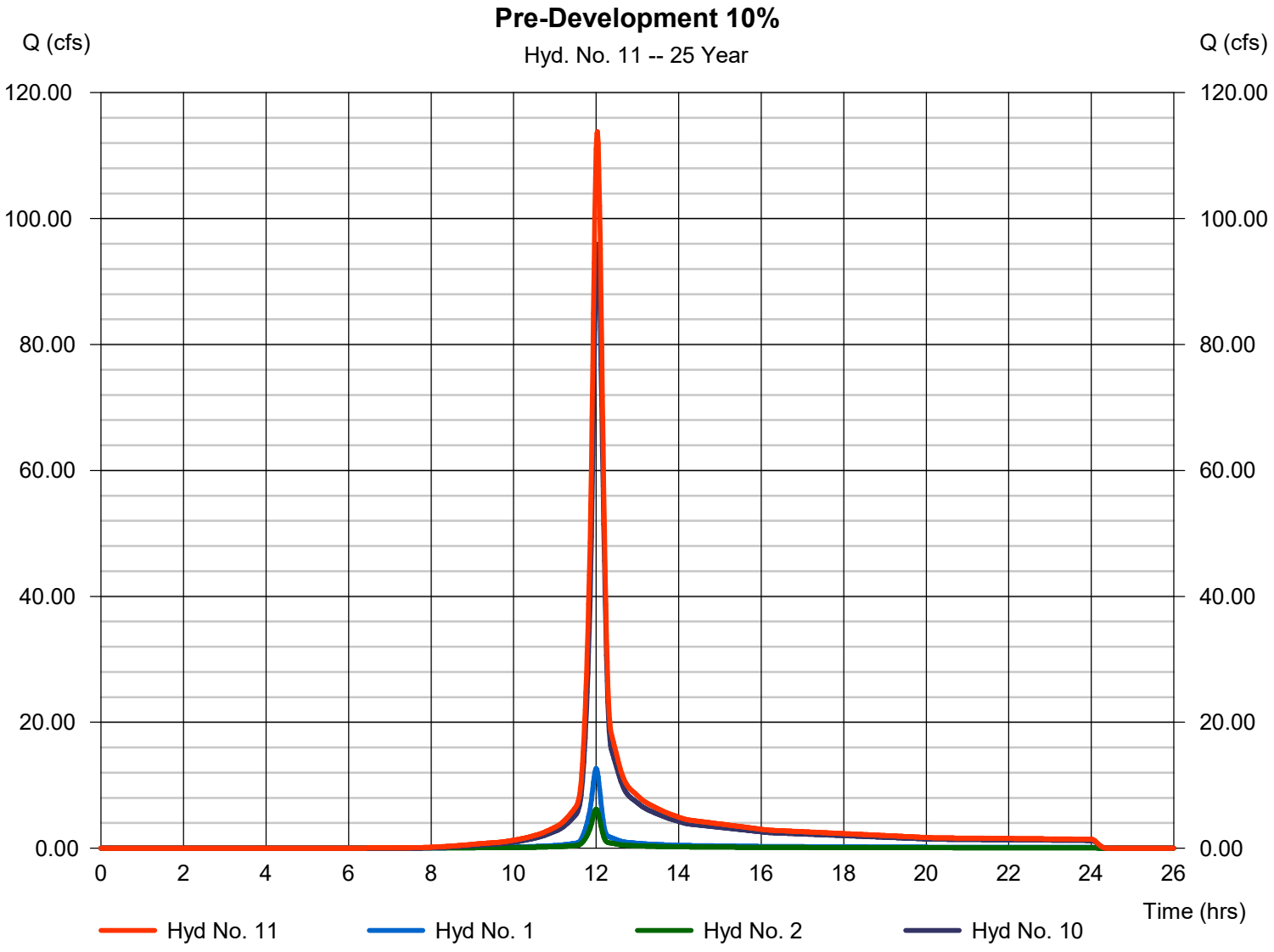


Hydrograph Report

Hyd. No. 11

Pre-Development 10%

Hydrograph type	= Combine	Peak discharge	= 113.81 cfs
Storm frequency	= 25 yrs	Time to peak	= 12.03 hrs
Time interval	= 1 min	Hyd. volume	= 304,651 cuft
Inflow hyds.	= 1, 2, 10	Contrib. drain. area	= 20.620 ac

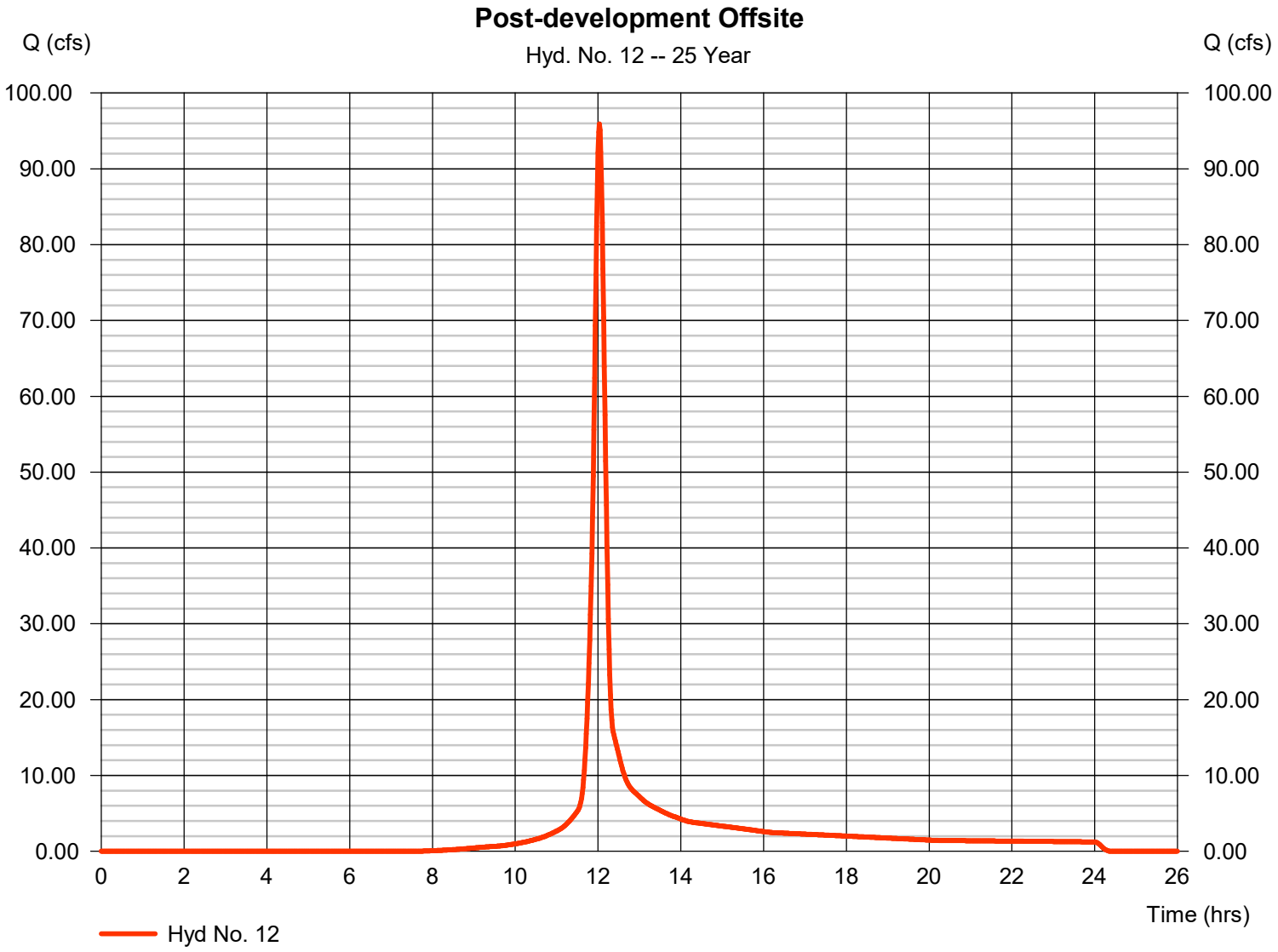


Hydrograph Report

Hyd. No. 12

Post-development Offsite

Hydrograph type	=	SCS Runoff	Peak discharge	=	95.91 cfs
Storm frequency	=	25 yrs	Time to peak	=	12.03 hrs
Time interval	=	1 min	Hyd. volume	=	258,004 cuft
Drainage area	=	18.000 ac	Curve number	=	72
Basin Slope	=	0.0 %	Hydraulic length	=	0 ft
Tc method	=	TR55	Time of conc. (Tc)	=	13.80 min
Total precip.	=	7.14 in	Distribution	=	Type II
Storm duration	=	24 hrs	Shape factor	=	484

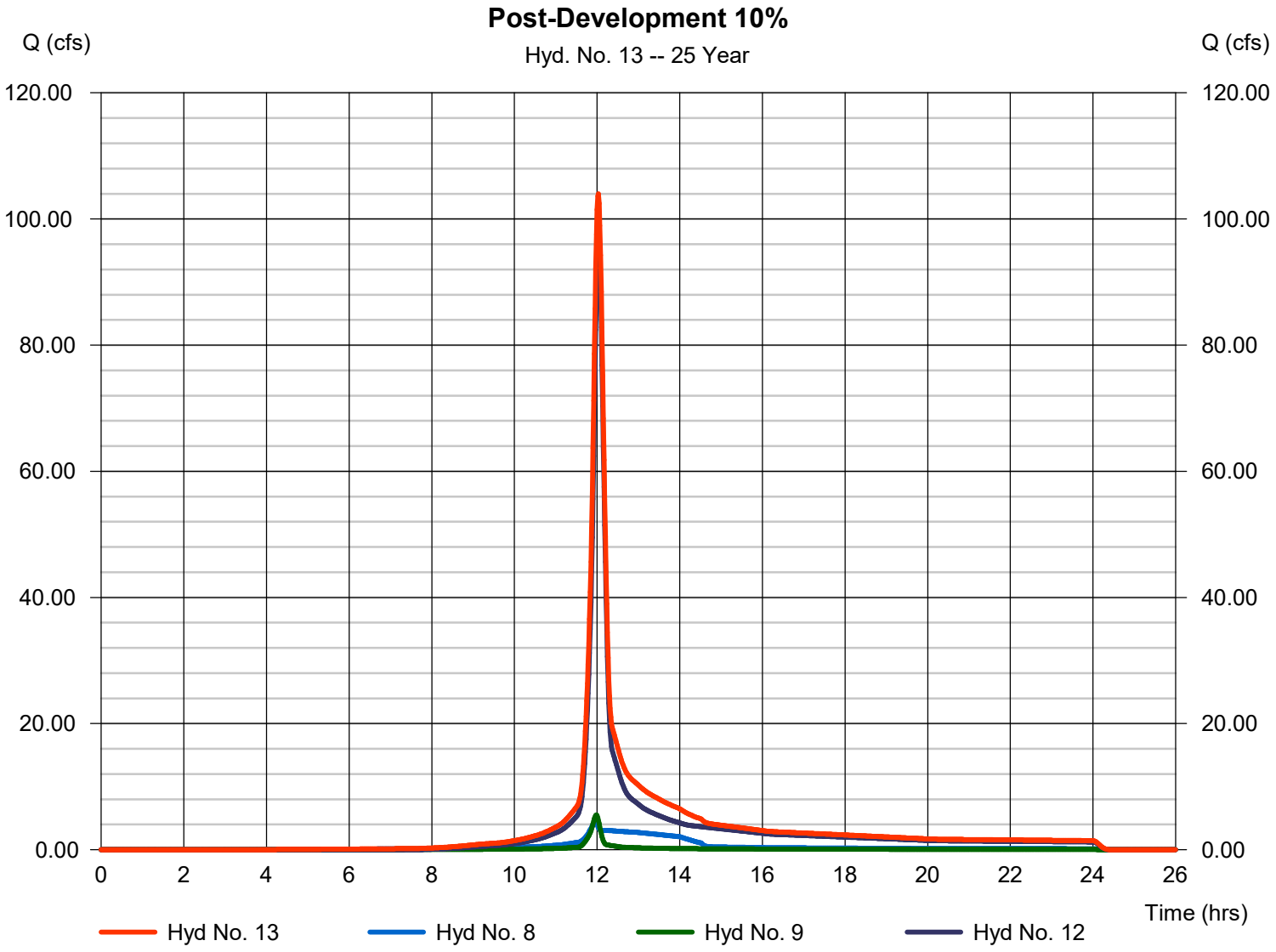


Hydrograph Report

Hyd. No. 13

Post-Development 10%

Hydrograph type	=	Combine	Peak discharge	=	104.00 cfs
Storm frequency	=	25 yrs	Time to peak	=	12.03 hrs
Time interval	=	1 min	Hyd. volume	=	313,006 cuft
Inflow hyds.	=	8, 9, 12	Contrib. drain. area	=	18.000 ac



Hydrograph Summary Report

Hydraflow Hydrographs Extension for Autodesk® Civil 3D® by Autodesk, Inc. v2022

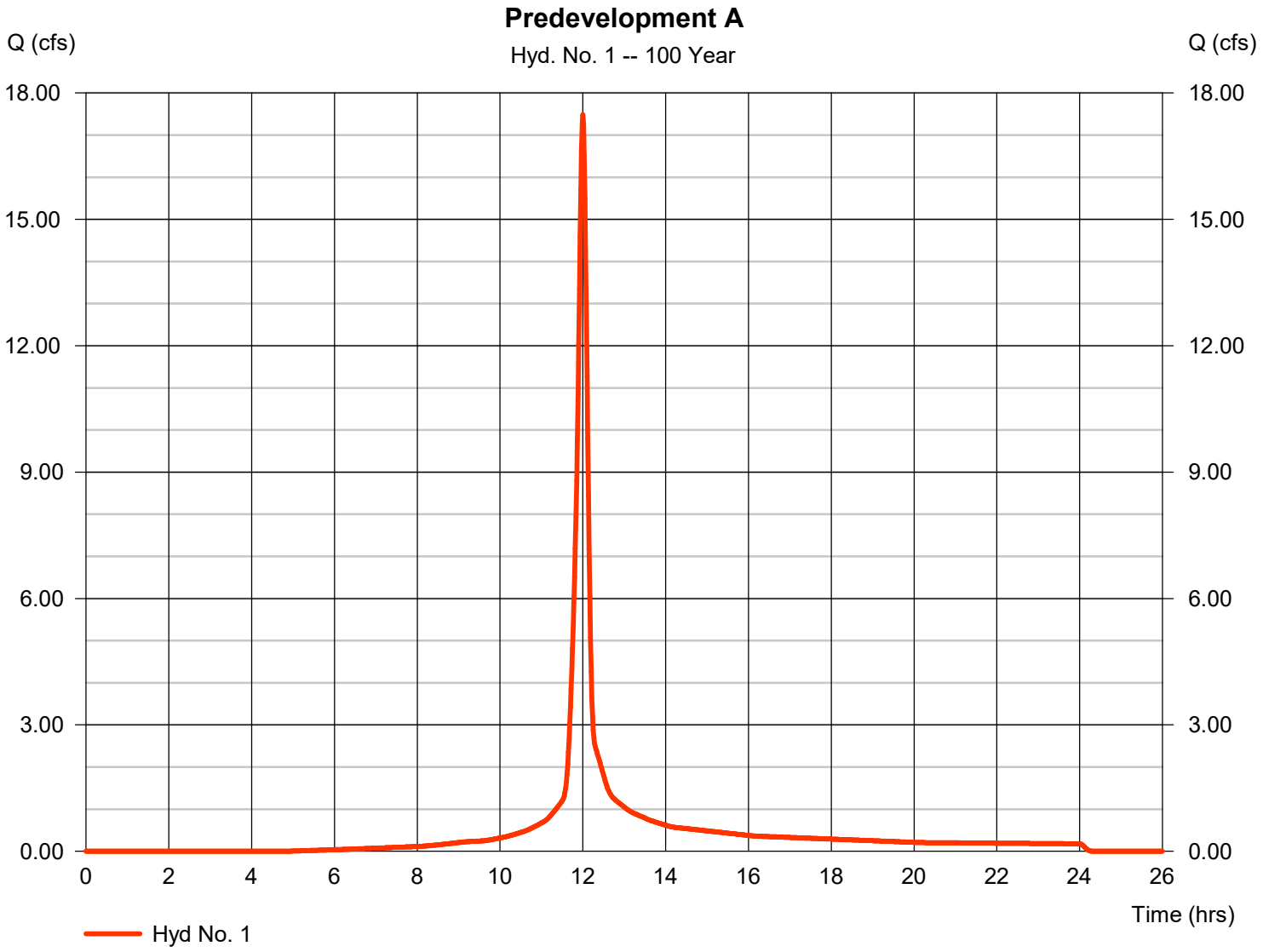
Hyd. No.	Hydrograph type (origin)	Peak flow (cfs)	Time interval (min)	Time to Peak (min)	Hyd. volume (cuft)	Inflow hyd(s)	Maximum elevation (ft)	Total strge used (cuft)	Hydrograph Description
1	SCS Runoff	17.48	1	720	43,688	------	------	------	Predevelopment A
2	SCS Runoff	8.449	1	720	21,396	------	------	------	Predevelopment B
3	SCS Runoff	2.834	1	716	5,722	------	------	------	Undetained A
4	SCS Runoff	19.01	1	720	50,583	------	------	------	Detained A
5	SCS Runoff	6.674	1	719	15,980	------	------	------	Undetained B
6	SCS Runoff	0.797	1	717	1,842	------	------	------	Undetained B2
7	Reservoir	3.068	1	735	50,581	4	1231.81	17,500	ADS Underground System
8	Combine	5.420	1	717	56,303	3, 7	------	------	Total Post-Development A
9	Combine	7.409	1	719	17,822	5, 6,	------	------	Total Post-Development B
10	SCS Runoff	138.33	1	722	374,710	------	------	------	Pre-development Offsite
11	Combine	162.82	1	721	439,794	1, 2, 10	------	------	Pre-Development 10%
12	SCS Runoff	138.33	1	722	374,710	------	------	------	Post-development Offsite
13	Combine	148.45	1	722	448,835	8, 9, 12	------	------	Post-Development 10%

Hydrograph Report

Hyd. No. 1

Predevelopment A

Hydrograph type	=	SCS Runoff	Peak discharge	=	17.48 cfs
Storm frequency	=	100 yrs	Time to peak	=	12.00 hrs
Time interval	=	1 min	Hyd. volume	=	43,688 cuft
Drainage area	=	1.790 ac	Curve number	=	79
Basin Slope	=	2.0 %	Hydraulic length	=	130 ft
Tc method	=	TR55	Time of conc. (Tc)	=	11.00 min
Total precip.	=	9.17 in	Distribution	=	Type II
Storm duration	=	24 hrs	Shape factor	=	484



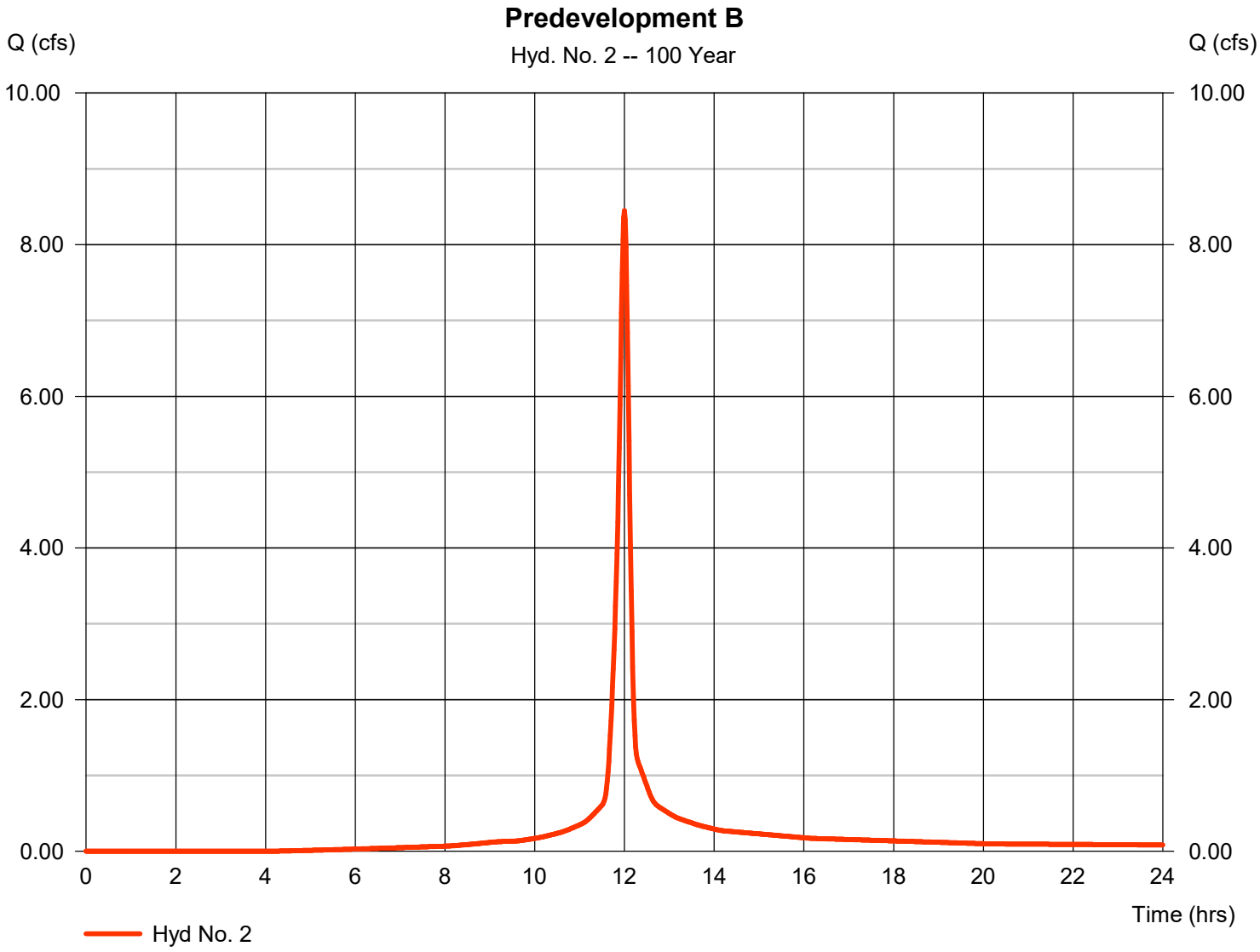
Hydrograph Report

Hyd. No. 2

Predevelopment B

Hydrograph type	=	SCS Runoff	Peak discharge	=	8.449 cfs
Storm frequency	=	100 yrs	Time to peak	=	12.00 hrs
Time interval	=	1 min	Hyd. volume	=	21,396 cuft
Drainage area	=	0.830 ac	Curve number	=	82*
Basin Slope	=	0.0 %	Hydraulic length	=	0 ft
Tc method	=	TR55	Time of conc. (Tc)	=	10.60 min
Total precip.	=	9.17 in	Distribution	=	Type II
Storm duration	=	24 hrs	Shape factor	=	484

* Composite (Area/CN) = [(0.120 x 98) + (0.710 x 79)] / 0.830



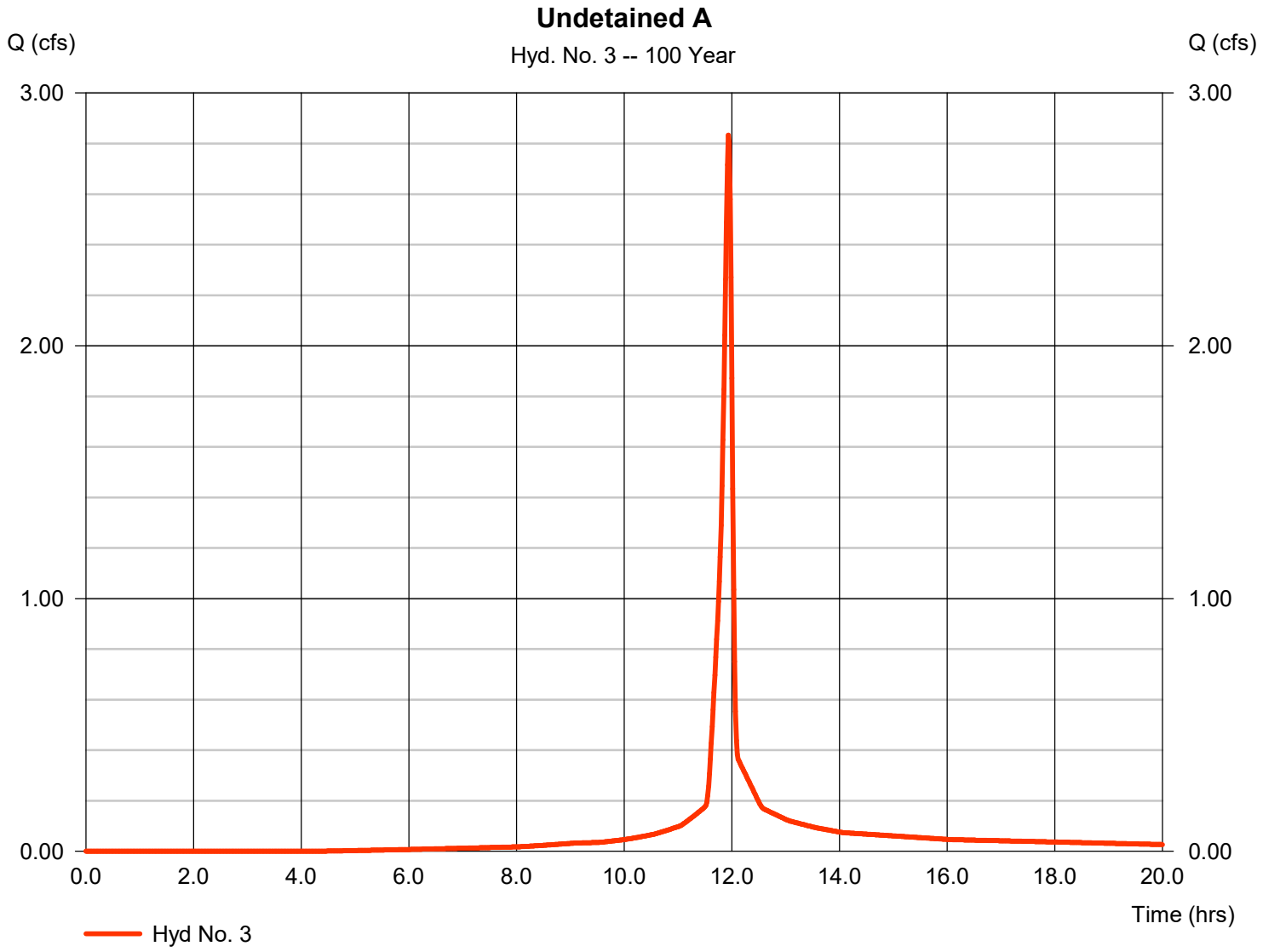
Hydrograph Report

Hyd. No. 3

Undetained A

Hydrograph type	=	SCS Runoff	Peak discharge	=	2.834 cfs
Storm frequency	=	100 yrs	Time to peak	=	11.93 hrs
Time interval	=	1 min	Hyd. volume	=	5,722 cuft
Drainage area	=	0.230 ac	Curve number	=	81*
Basin Slope	=	0.0 %	Hydraulic length	=	0 ft
Tc method	=	TR55	Time of conc. (Tc)	=	4.40 min
Total precip.	=	9.17 in	Distribution	=	Type II
Storm duration	=	24 hrs	Shape factor	=	484

* Composite (Area/CN) = [(0.200 x 79) + (0.030 x 98)] / 0.230



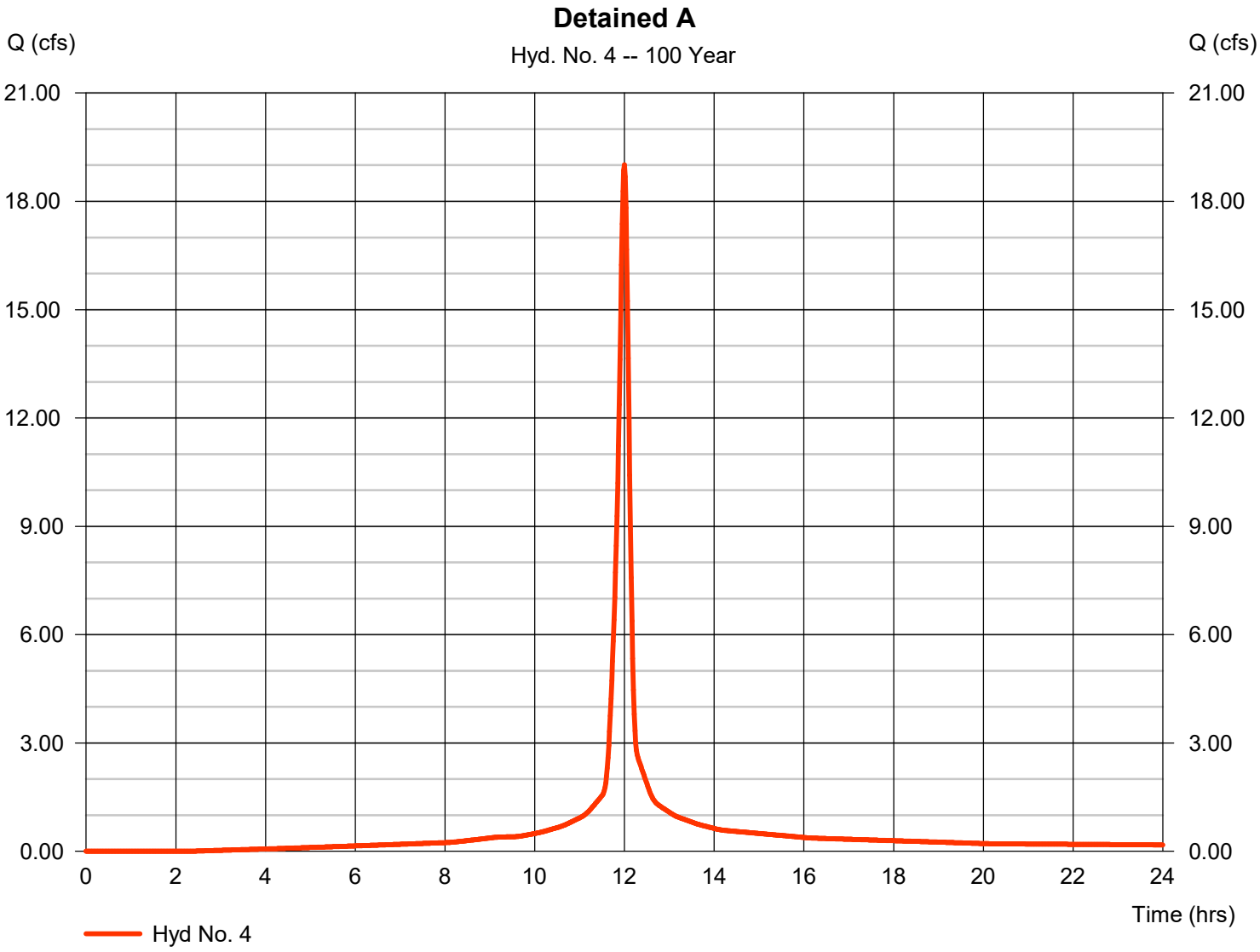
Hydrograph Report

Hyd. No. 4

Detained A

Hydrograph type	=	SCS Runoff	Peak discharge	=	19.01 cfs
Storm frequency	=	100 yrs	Time to peak	=	12.00 hrs
Time interval	=	1 min	Hyd. volume	=	50,583 cuft
Drainage area	=	1.720 ac	Curve number	=	90*
Basin Slope	=	0.0 %	Hydraulic length	=	0 ft
Tc method	=	TR55	Time of conc. (Tc)	=	10.40 min
Total precip.	=	9.17 in	Distribution	=	Type II
Storm duration	=	24 hrs	Shape factor	=	484

* Composite (Area/CN) = [(0.970 x 98) + (0.750 x 79)] / 1.720



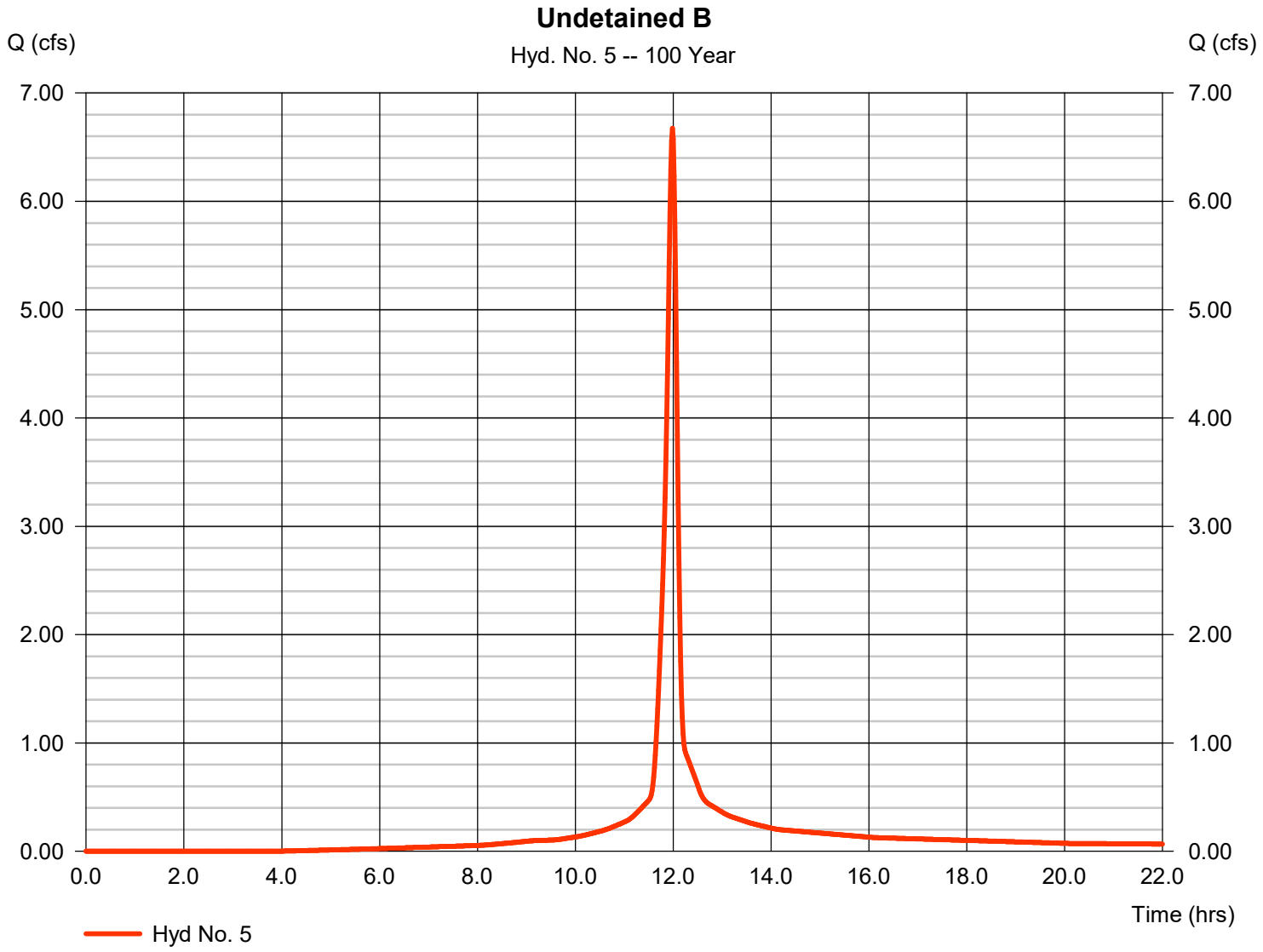
Hydrograph Report

Hyd. No. 5

Undetained B

Hydrograph type	=	SCS Runoff	Peak discharge	=	6.674 cfs
Storm frequency	=	100 yrs	Time to peak	=	11.98 hrs
Time interval	=	1 min	Hyd. volume	=	15,980 cuft
Drainage area	=	0.620 ac	Curve number	=	83*
Basin Slope	=	0.0 %	Hydraulic length	=	0 ft
Tc method	=	TR55	Time of conc. (Tc)	=	10.00 min
Total precip.	=	9.17 in	Distribution	=	Type II
Storm duration	=	24 hrs	Shape factor	=	484

* Composite (Area/CN) = [(0.130 x 98) + (0.490 x 79)] / 0.620



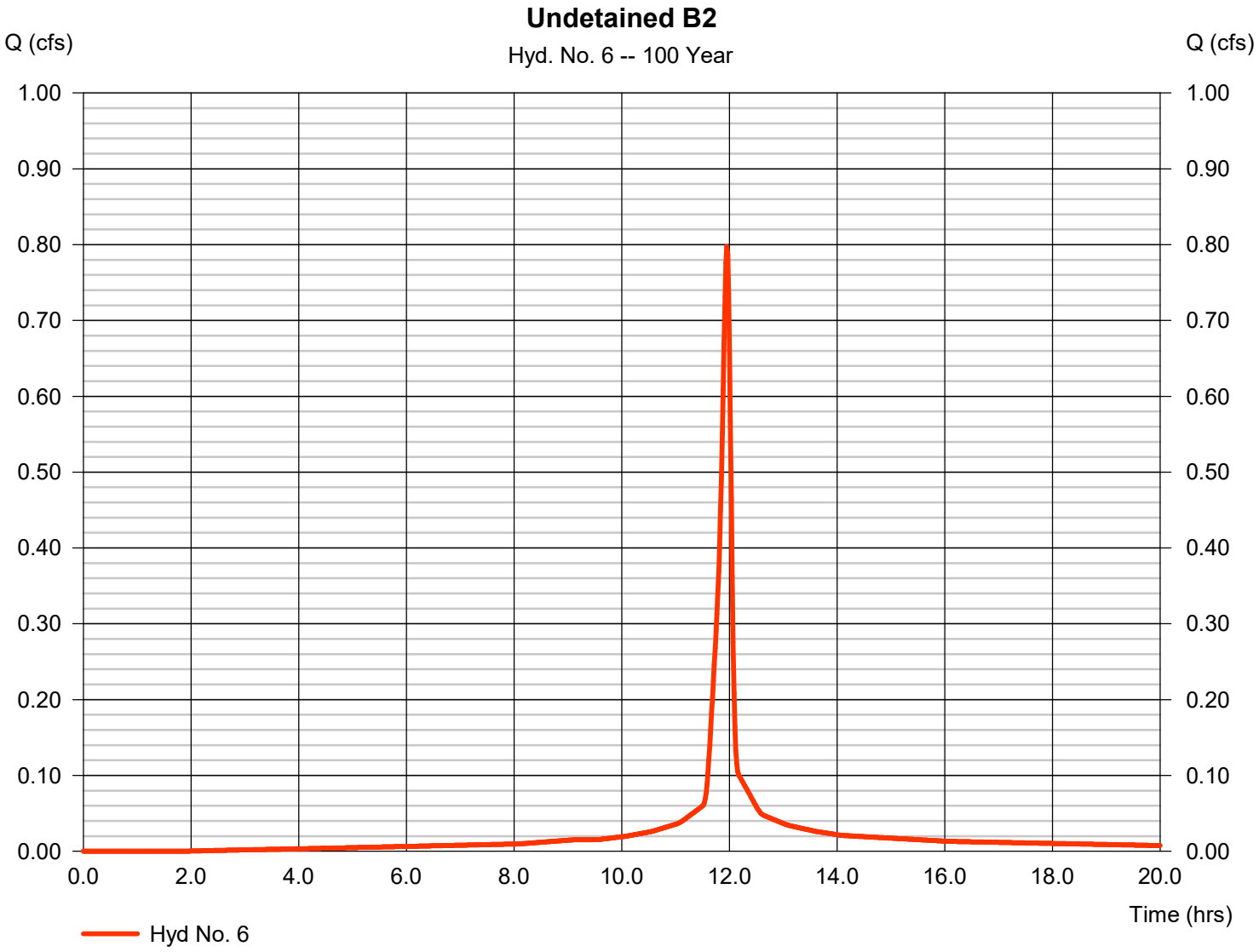
Hydrograph Report

Hyd. No. 6

Undetained B2

Hydrograph type	=	SCS Runoff	Peak discharge	=	0.797 cfs
Storm frequency	=	100 yrs	Time to peak	=	11.95 hrs
Time interval	=	1 min	Hyd. volume	=	1,842 cuft
Drainage area	=	0.060 ac	Curve number	=	92*
Basin Slope	=	0.0 %	Hydraulic length	=	0 ft
Tc method	=	User	Time of conc. (Tc)	=	5.00 min
Total precip.	=	9.17 in	Distribution	=	Type II
Storm duration	=	24 hrs	Shape factor	=	484

* Composite (Area/CN) = [(0.020 x 79) + (0.040 x 98)] / 0.060



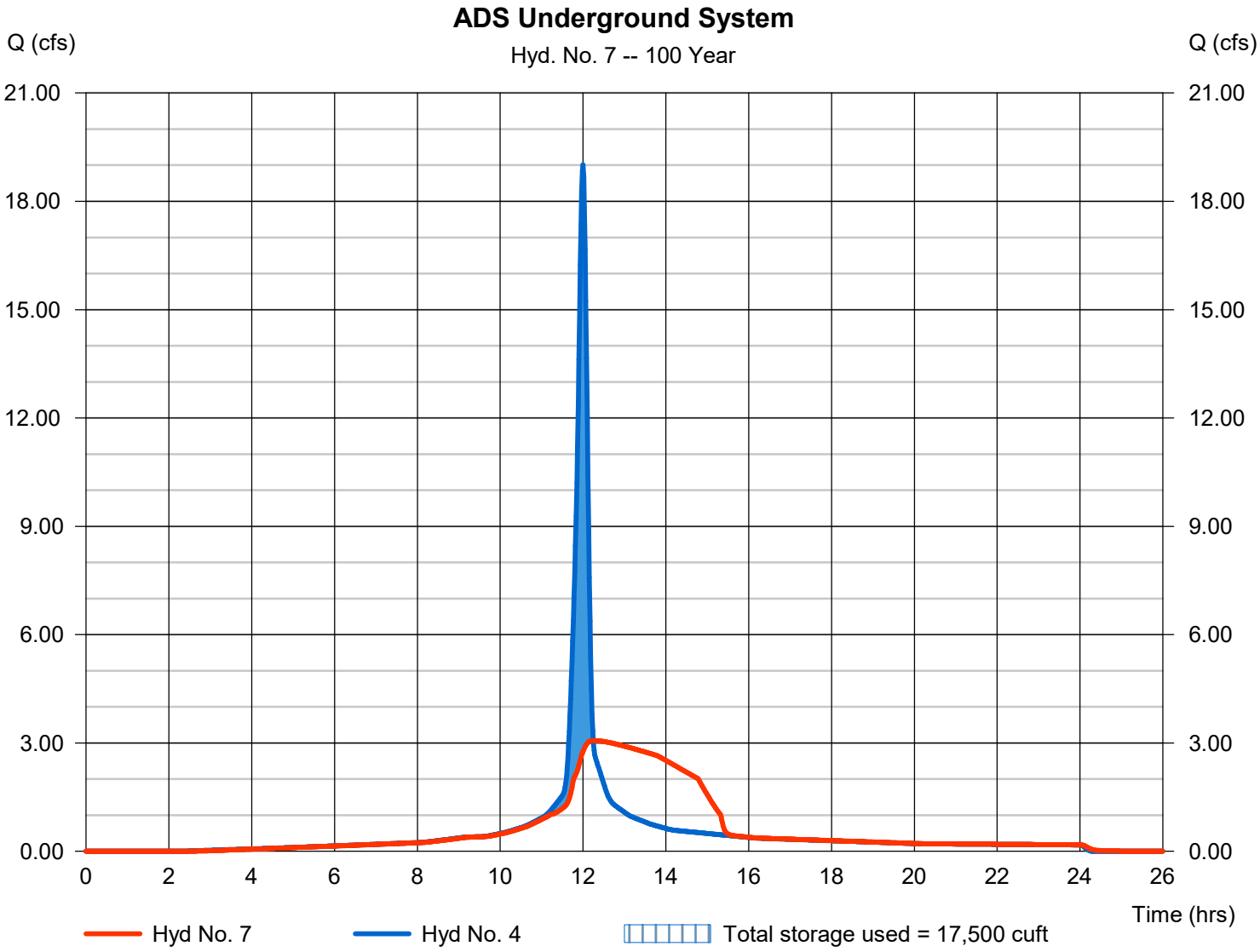
Hydrograph Report

Hyd. No. 7

ADS Underground System

Hydrograph type	=	Reservoir	Peak discharge	=	3.068 cfs
Storm frequency	=	100 yrs	Time to peak	=	12.25 hrs
Time interval	=	1 min	Hyd. volume	=	50,581 cuft
Inflow hyd. No.	=	4 - Detained A	Max. Elevation	=	1231.81 ft
Reservoir name	=	<New Pond>	Max. Storage	=	17,500 cuft

Storage Indication method used.

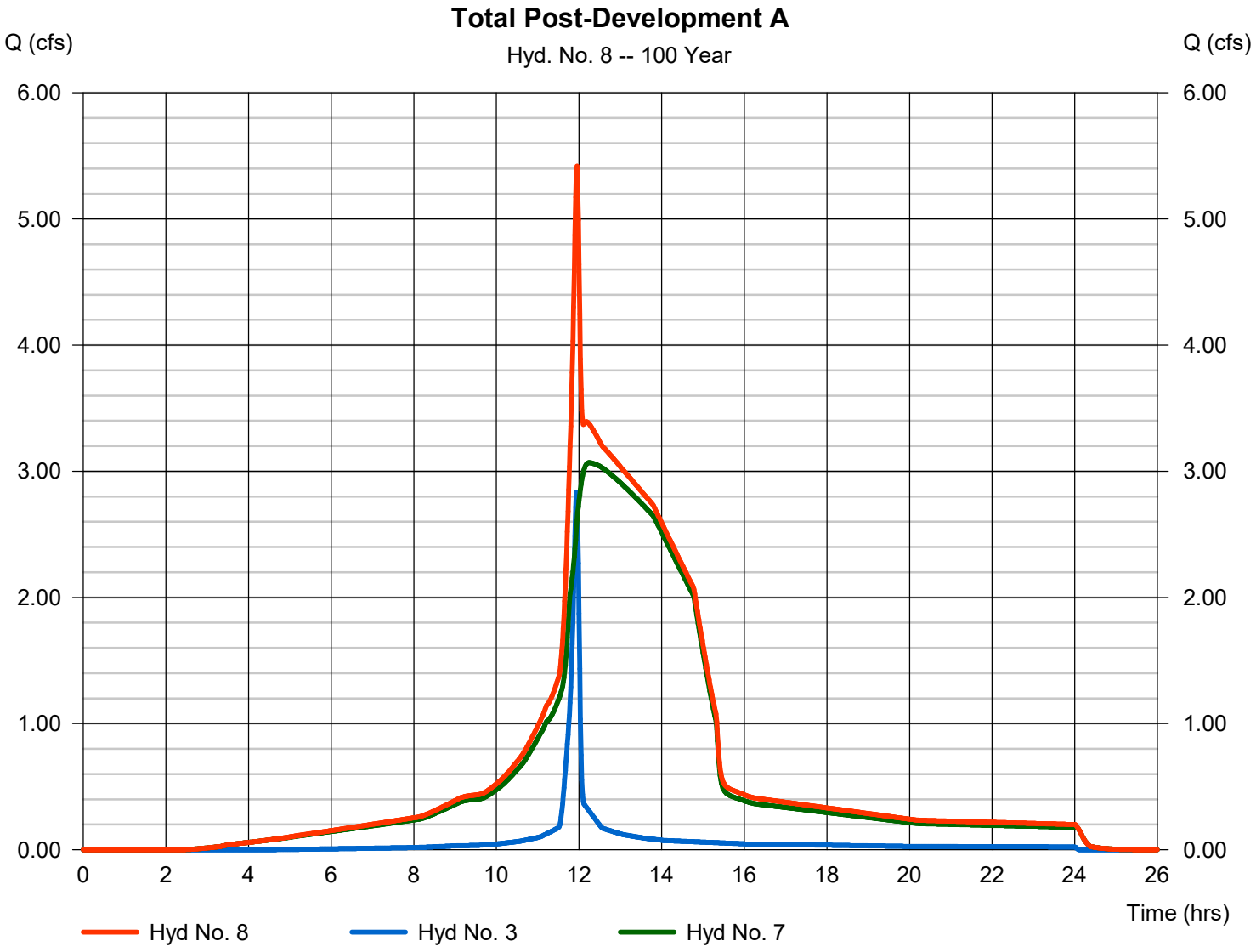


Hydrograph Report

Hyd. No. 8

Total Post-Development A

Hydrograph type	= Combine	Peak discharge	= 5.420 cfs
Storm frequency	= 100 yrs	Time to peak	= 11.95 hrs
Time interval	= 1 min	Hyd. volume	= 56,303 cuft
Inflow hyds.	= 3, 7	Contrib. drain. area	= 0.230 ac

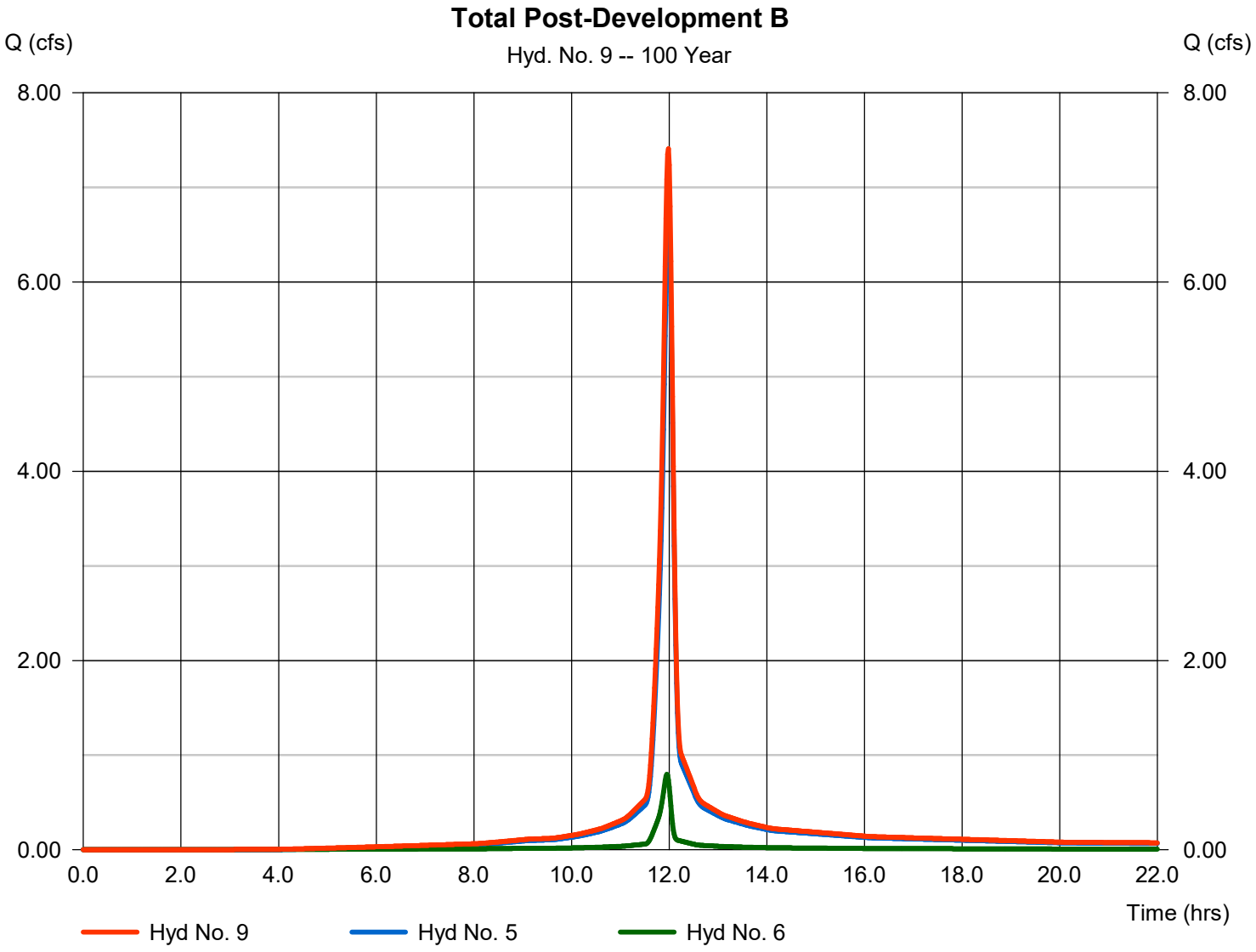


Hydrograph Report

Hyd. No. 9

Total Post-Development B

Hydrograph type	= Combine	Peak discharge	= 7.409 cfs
Storm frequency	= 100 yrs	Time to peak	= 11.98 hrs
Time interval	= 1 min	Hyd. volume	= 17,822 cuft
Inflow hyds.	= 5, 6	Contrib. drain. area	= 0.680 ac



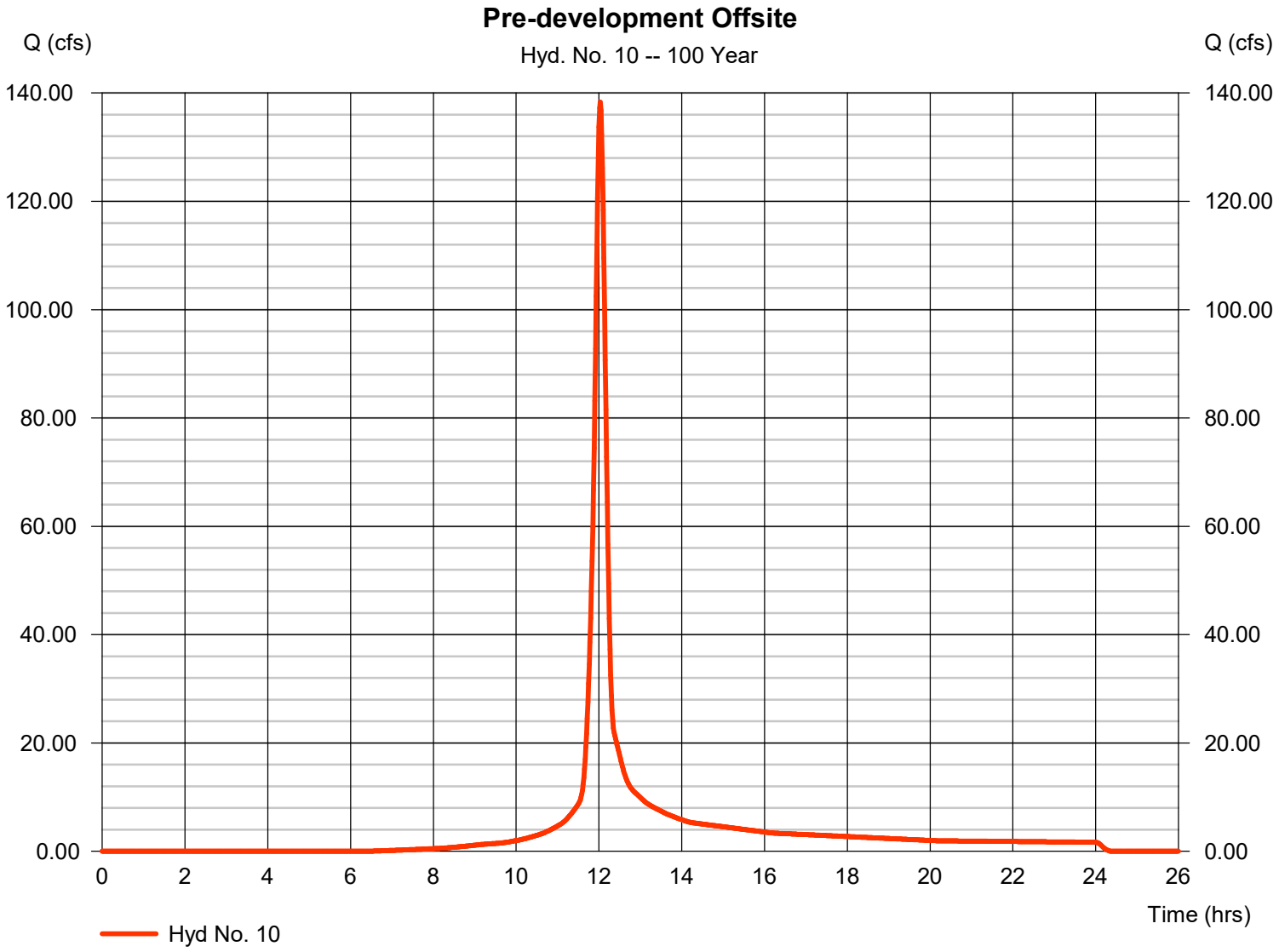
Hydrograph Report

Hyd. No. 10

Pre-development Offsite

Hydrograph type	=	SCS Runoff	Peak discharge	=	138.33 cfs
Storm frequency	=	100 yrs	Time to peak	=	12.03 hrs
Time interval	=	1 min	Hyd. volume	=	374,710 cuft
Drainage area	=	18.000 ac	Curve number	=	72*
Basin Slope	=	0.0 %	Hydraulic length	=	0 ft
Tc method	=	TR55	Time of conc. (Tc)	=	13.80 min
Total precip.	=	9.17 in	Distribution	=	Type II
Storm duration	=	24 hrs	Shape factor	=	484

* Composite (Area/CN) = [(2.140 x 79) + (0.560 x 92) + (15.300 x 70)] / 18.000

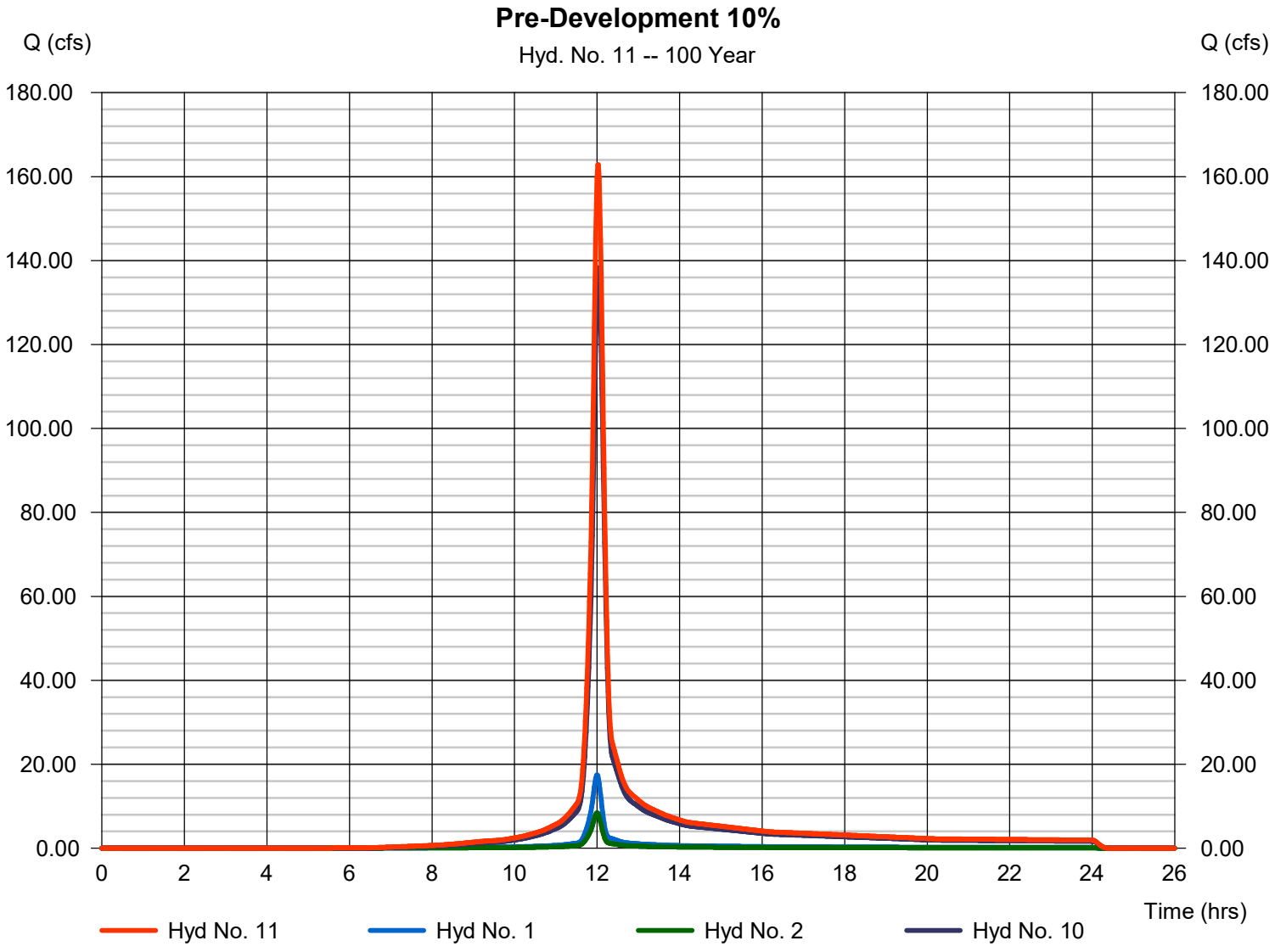


Hydrograph Report

Hyd. No. 11

Pre-Development 10%

Hydrograph type	= Combine	Peak discharge	= 162.82 cfs
Storm frequency	= 100 yrs	Time to peak	= 12.02 hrs
Time interval	= 1 min	Hyd. volume	= 439,794 cuft
Inflow hyds.	= 1, 2, 10	Contrib. drain. area	= 20.620 ac

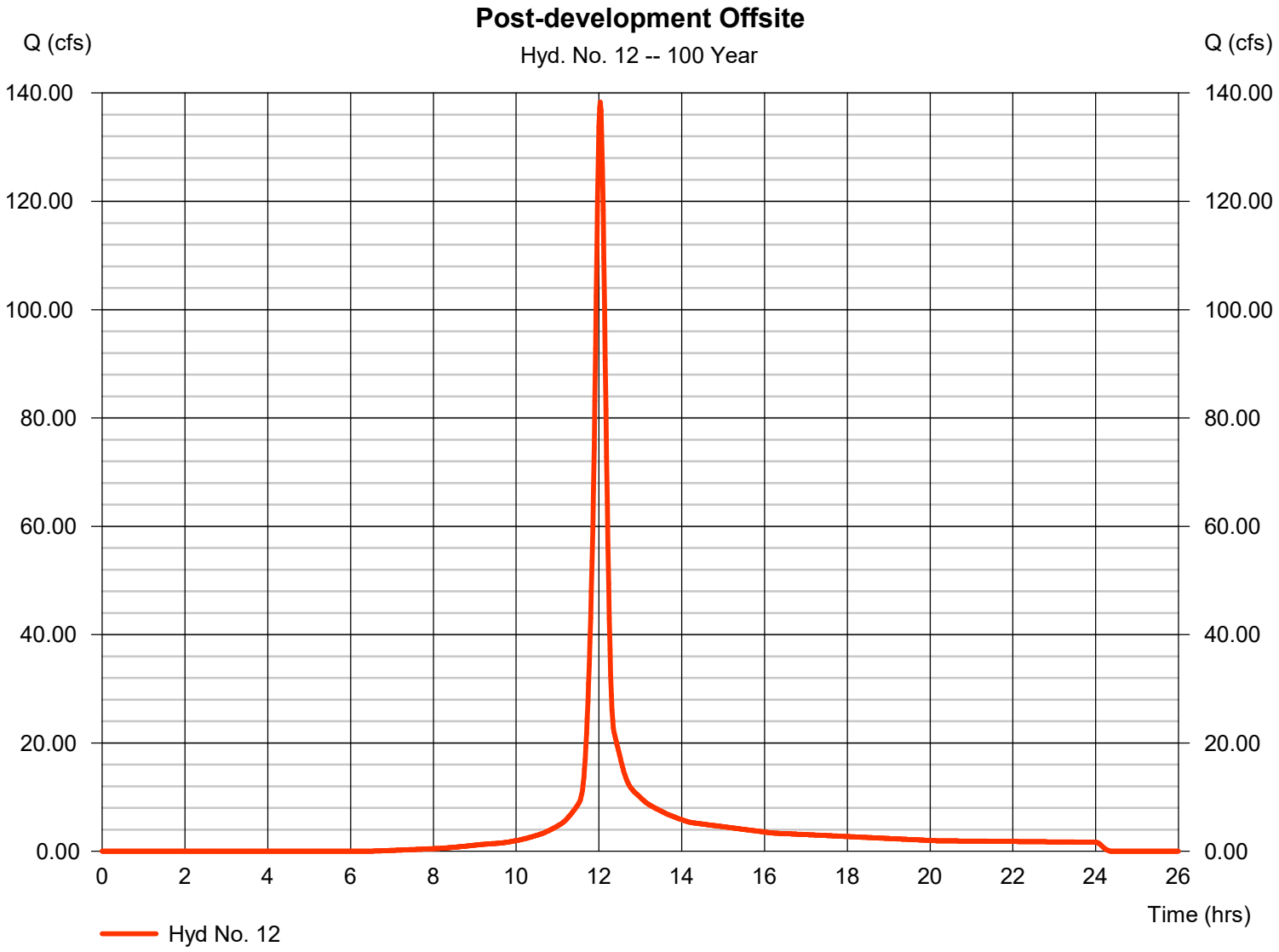


Hydrograph Report

Hyd. No. 12

Post-development Offsite

Hydrograph type	=	SCS Runoff	Peak discharge	=	138.33 cfs
Storm frequency	=	100 yrs	Time to peak	=	12.03 hrs
Time interval	=	1 min	Hyd. volume	=	374,710 cuft
Drainage area	=	18.000 ac	Curve number	=	72
Basin Slope	=	0.0 %	Hydraulic length	=	0 ft
Tc method	=	TR55	Time of conc. (Tc)	=	13.80 min
Total precip.	=	9.17 in	Distribution	=	Type II
Storm duration	=	24 hrs	Shape factor	=	484

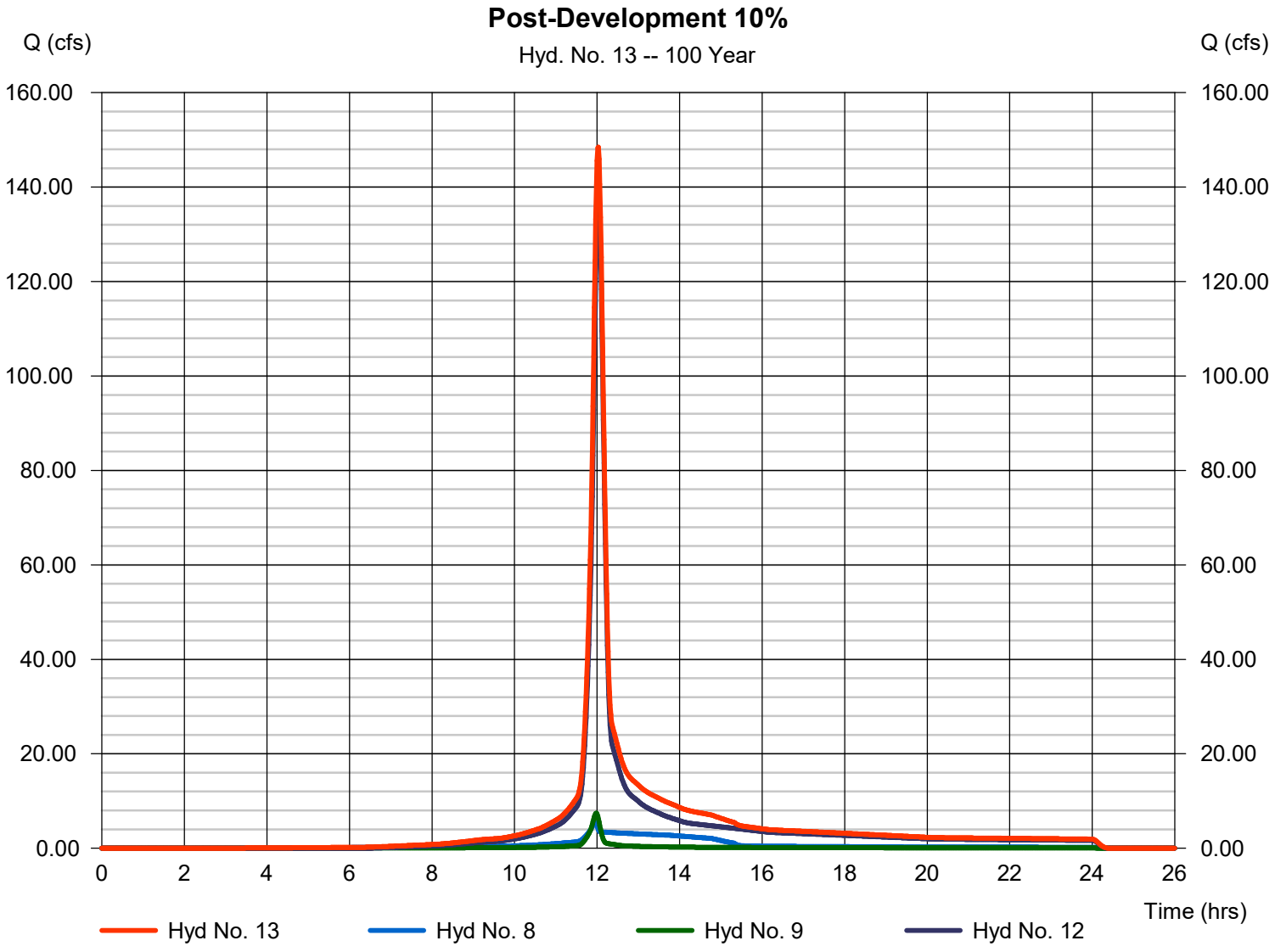


Hydrograph Report

Hyd. No. 13

Post-Development 10%

Hydrograph type	= Combine	Peak discharge	= 148.45 cfs
Storm frequency	= 100 yrs	Time to peak	= 12.03 hrs
Time interval	= 1 min	Hyd. volume	= 448,835 cuft
Inflow hyds.	= 8, 9, 12	Contrib. drain. area	= 18.000 ac



Hydraflow Rainfall Report

Return Period (Yrs)	Intensity-Duration-Frequency Equation Coefficients (FHA)			
	B	D	E	(N/A)
1	0.0000	0.0000	0.0000	--------
2	69.8703	13.1000	0.8658	--------
3	0.0000	0.0000	0.0000	--------
5	79.2597	14.6000	0.8369	--------
10	88.2351	15.5000	0.8279	--------
25	102.6072	16.5000	0.8217	--------
50	114.8193	17.2000	0.8199	--------
100	127.1596	17.8000	0.8186	--------

File name: SampleFHA.idf

Intensity = B / (Tc + D)^E

Return Period (Yrs)	Intensity Values (in/hr)											
	5 min	10	15	20	25	30	35	40	45	50	55	60
1	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
2	5.69	4.61	3.89	3.38	2.99	2.69	2.44	2.24	2.07	1.93	1.81	1.70
3	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
5	6.57	5.43	4.65	4.08	3.65	3.30	3.02	2.79	2.59	2.42	2.27	2.15
10	7.24	6.04	5.21	4.59	4.12	3.74	3.43	3.17	2.95	2.77	2.60	2.46
25	8.25	6.95	6.03	5.34	4.80	4.38	4.02	3.73	3.48	3.26	3.07	2.91
50	9.04	7.65	6.66	5.92	5.34	4.87	4.49	4.16	3.88	3.65	3.44	3.25
100	9.83	8.36	7.30	6.50	5.87	5.36	4.94	4.59	4.29	4.03	3.80	3.60

Tc = time in minutes. Values may exceed 60.

Precip. file name: Sample.pcp

Storm Distribution	Rainfall Precipitation Table (in)							
	1-yr	2-yr	3-yr	5-yr	10-yr	25-yr	50-yr	100-yr
SCS 24-hour	0.00	4.19	0.00	5.12	5.94	7.14	8.13	9.17
SCS 6-Hr	0.00	1.80	0.00	0.00	2.60	0.00	0.00	4.00
Huff-1st	0.00	1.55	0.00	2.75	4.00	5.38	6.50	8.00
Huff-2nd	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
Huff-3rd	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
Huff-4th	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
Huff-Indy	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
Custom	0.00	1.75	0.00	2.80	3.90	5.25	6.00	7.10

APPENDIX
Detention System Details

PROJECT INFORMATION	
ENGINEERED PRODUCT MANAGER:	
ADS SALES REP:	
PROJECT NO:	

ADS Advanced Drainage Systems, Inc.

ADS SiteAssist™ FOR STORMTECH INSTALLATION INSTRUCTIONS VISIT OUR APP

SOURCES OF COMMUNITY INDEPENDENT LIVING

SC-740 STORMTECH CHAMBER SPECIFICATIONS

1. CHAMBERS SHALL BE STORMTECH SC-740.
2. CHAMBERS SHALL BE ARCH-SHAPED AND SHALL BE MANUFACTURED FROM VIRGIN, IMPACT-MODIFIED POLYPROPYLENE COPOLYMERS.
3. CHAMBERS SHALL MEET THE REQUIREMENTS OF ASTM F2418, "STANDARD SPECIFICATION FOR POLYPROPYLENE (PP) CORRUGATED WALL STORMWATER COLLECTION CHAMBERS".
4. CHAMBER ROWS SHALL PROVIDE CONTINUOUS, UNOBSTRUCTED INTERNAL SPACE WITH NO INTERNAL SUPPORTS THAT WOULD IMPEDE FLOW OR LIMIT ACCESS FOR INSPECTION.
5. THE STRUCTURAL DESIGN OF THE CHAMBERS, THE STRUCTURAL BACKFILL, AND THE INSTALLATION REQUIREMENTS SHALL ENSURE THAT THE LOAD FACTORS SPECIFIED IN THE AASHTO LRFD BRIDGE DESIGN SPECIFICATIONS, SECTION 12.12, ARE MET FOR: 1) LONG-DURATION DEAD LOADS AND 2) SHORT-DURATION LIVE LOADS, BASED ON THE AASHTO DESIGN TRUCK WITH CONSIDERATION FOR IMPACT AND MULTIPLE VEHICLE PRESENCES.
6. CHAMBERS SHALL BE DESIGNED, TESTED AND ALLOWABLE LOAD CONFIGURATIONS DETERMINED IN ACCORDANCE WITH ASTM F2787, "STANDARD PRACTICE FOR STRUCTURAL DESIGN OF THERMOPLASTIC CORRUGATED WALL STORMWATER COLLECTION CHAMBERS". LOAD CONFIGURATIONS SHALL INCLUDE: 1) INSTANTANEOUS (<1 MIN) AASHTO DESIGN TRUCK LIVE LOAD ON MINIMUM COVER 2) MAXIMUM PERMANENT (75-YR) COVER LOAD AND 3) ALLOWABLE COVER WITH PARKED (1-WEEK) AASHTO DESIGN TRUCK.
7. REQUIREMENTS FOR HANDLING AND INSTALLATION:
   - TO MAINTAIN THE WIDTH OF CHAMBERS DURING SHIPPING AND HANDLING, CHAMBERS SHALL HAVE INTEGRAL, INTERLOCKING STACKING LUGS.
   - TO ENSURE A SECURE JOINT DURING INSTALLATION AND BACKFILL, THE HEIGHT OF THE CHAMBER JOINT SHALL NOT BE LESS THAN 2".
   - TO ENSURE THE INTEGRITY OF THE ARCH SHAPE DURING INSTALLATION, a) THE ARCH STIFFNESS CONSTANT SHALL BE GREATER THAN OR EQUAL TO 550 LBS/IN/IN. THE ASC IS DEFINED IN SECTION 6.2.8 OF ASTM F2418. AND b) TO RESIST CHAMBER DEFORMATION DURING INSTALLATION AT ELEVATED TEMPERATURES (ABOVE 73° F / 23° C), CHAMBERS SHALL BE PRODUCED FROM REFLECTIVE GOLD OR YELLOW COLORS.
8. ONLY CHAMBERS THAT ARE APPROVED BY THE SITE DESIGN ENGINEER WILL BE ALLOWED. UPON REQUEST BY THE SITE DESIGN ENGINEER OR OWNER, THE CHAMBER MANUFACTURER SHALL SUBMIT A STRUCTURAL EVALUATION FOR APPROVAL BEFORE DELIVERING CHAMBERS TO THE PROJECT SITE AS FOLLOWS:
   - THE STRUCTURAL EVALUATION SHALL BE SEALED BY A REGISTERED PROFESSIONAL ENGINEER.
   - THE STRUCTURAL EVALUATION SHALL DEMONSTRATE THAT THE SAFETY FACTORS ARE GREATER THAN OR EQUAL TO 1.95 FOR DEAD LOAD AND 1.75 FOR LIVE LOAD, THE MINIMUM REQUIRED BY ASTM F2787 AND BY SECTIONS 3 AND 12.12 OF THE AASHTO LRFD BRIDGE DESIGN SPECIFICATIONS FOR THERMOPLASTIC PIPE.
   - THE TEST DERIVED CREEP MODULUS AS SPECIFIED IN ASTM F2418 SHALL BE USED FOR PERMANENT DEAD LOAD DESIGN EXCEPT THAT IT SHALL BE THE 75-YEAR MODULUS USED FOR DESIGN.
9. CHAMBERS AND END CAPS SHALL BE PRODUCED AT AN ISO 9001 CERTIFIED MANUFACTURING FACILITY.

IMPORTANT - NOTES FOR THE BIDDING AND INSTALLATION OF THE SC-740 SYSTEM

1. STORMTECH SC-740 CHAMBERS SHALL NOT BE INSTALLED UNTIL THE MANUFACTURER'S REPRESENTATIVE HAS COMPLETED A PRE-CONSTRUCTION MEETING WITH THE INSTALLERS.
2. STORMTECH SC-740 CHAMBERS SHALL BE INSTALLED IN ACCORDANCE WITH THE "STORMTECH SC-310/SC-740/DC-780 CONSTRUCTION GUIDE".
3. CHAMBERS ARE NOT TO BE BACKFILLED WITH A DOZER OR AN EXCAVATOR SITUATED OVER THE CHAMBERS. STORMTECH RECOMMENDS 3 BACKFILL METHODS:
   - STONESHOOTER LOCATED OFF THE CHAMBER BED.
   - BACKFILL AS ROWS ARE BUILT USING AN EXCAVATOR ON THE FOUNDATION STONE OR SUBGRADE.
   - BACKFILL FROM OUTSIDE THE EXCAVATION USING A LONG BOOM HOE OR EXCAVATOR.
4. THE FOUNDATION STONE SHALL BE LEVELED AND COMPACTED PRIOR TO PLACING CHAMBERS.
5. JOINTS BETWEEN CHAMBERS SHALL BE PROPERLY SEATED PRIOR TO PLACING STONE.
6. MAINTAIN MINIMUM - 6" (150 mm) SPACING BETWEEN THE CHAMBER ROWS.
7. EMBEDMENT STONE SURROUNDING CHAMBERS MUST BE A CLEAN, CRUSHED, ANGULAR STONE 3/4-2" (20-50 mm).
8. THE CONTRACTOR MUST REPORT ANY DISCREPANCIES WITH CHAMBER FOUNDATION MATERIALS BEARING CAPACITIES TO THE SITE DESIGN ENGINEER.
9. ADS RECOMMENDS THE USE OF "FLEXSTORM CATCH IT" INSERTS DURING CONSTRUCTION FOR ALL INLETS TO PROTECT THE SUBSURFACE STORMWATER MANAGEMENT SYSTEM FROM CONSTRUCTION SITE RUNOFF.

NOTES FOR CONSTRUCTION EQUIPMENT

1. STORMTECH SC-740 CHAMBERS SHALL BE INSTALLED IN ACCORDANCE WITH THE "STORMTECH SC-310/SC-740/DC-780 CONSTRUCTION GUIDE".
2. THE USE OF CONSTRUCTION EQUIPMENT OVER SC-740 CHAMBERS IS LIMITED:
   - NO EQUIPMENT IS ALLOWED ON BARE CHAMBERS.
   - NO RUBBER TIRED LOADERS, DUMP TRUCKS, OR EXCAVATORS ARE ALLOWED UNTIL PROPER FILL DEPTHS ARE REACHED IN ACCORDANCE WITH THE "STORMTECH SC-310/SC-740/DC-780 CONSTRUCTION GUIDE".
   - WEIGHT LIMITS FOR CONSTRUCTION EQUIPMENT CAN BE FOUND IN THE "STORMTECH SC-310/SC-740/DC-780 CONSTRUCTION GUIDE".
3. FULL 36" (900 mm) OF STABILIZED COVER MATERIALS OVER THE CHAMBERS IS REQUIRED FOR DUMP TRUCK TRAVEL OR DUMPING.

USE OF A DOZER TO PUSH EMBEDMENT STONE BETWEEN THE ROWS OF CHAMBERS MAY CAUSE DAMAGE TO THE CHAMBERS AND IS NOT AN ACCEPTABLE BACKFILL METHOD. ANY CHAMBERS DAMAGED BY THE "DUMP AND PUSH" METHOD ARE NOT COVERED UNDER THE STORMTECH STANDARD WARRANTY.

CONTACT STORMTECH AT 1-888-892-2694 WITH ANY QUESTIONS ON INSTALLATION REQUIREMENTS OR WEIGHT LIMITS FOR CONSTRUCTION EQUIPMENT.

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PROPOSED LAYOUT

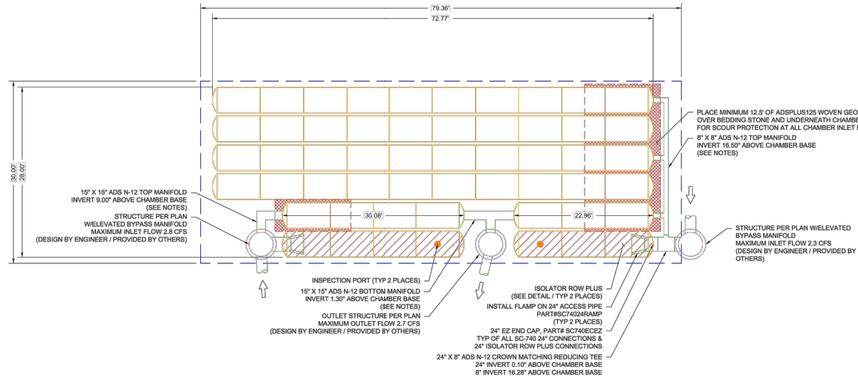
54	STORMTECH SC-740 CHAMBERS
16	STORMTECH SC-740 END CAPS
6	STONE ABOVE (in)
6	STONE BELOW (in)
40	% STONE VOID
4,922	**INSTALLED SYSTEM VOLUME (CF) (PERIMETER STONE INCLUDED)**
2381	SYSTEM AREA (ft²)
219	SYSTEM PERIMETER (ft)

PROPOSED ELEVATIONS

1239.00	MAXIMUM ALLOWABLE GRADE (TOP OF PAVEMENT/UNPAVED)
1232.50	MINIMUM ALLOWABLE GRADE (UNPAVED WITH TRAFFIC)
1232.50	MINIMUM ALLOWABLE GRADE (UNPAVED NO TRAFFIC)
1232.50	MINIMUM ALLOWABLE GRADE (BASE OF FLEXIBLE PAVEMENT)
1232.50	MINIMUM ALLOWABLE GRADE (TOP OF RIGID PAVEMENT)
1231.50	TOP OF STONE
1231.00	TOP OF SC-740 CHAMBER
1229.88	8" TOP MANIFOLD INVERT
1229.25	15" TOP MANIFOLD INVERT
1228.61	15" BOTTOM MANIFOLD INVERT
1228.51	24" ISOLATOR ROW PLUS CONNECTION INVERT
1228.50	BOTTOM OF SC-740 CHAMBER
1228.00	BOTTOM OF STONE

NOTES

- MANIFOLD SIZE TO BE DETERMINED BY SITE DESIGN ENGINEER. SEE TECHNICAL NOTE 6.32 FOR MANIFOLD SIZING GUIDANCE.
- DUE TO THE ADAPTATION OF THIS CHAMBER SYSTEM TO SPECIFIC SITE AND DESIGN CONSTRAINTS, IT MAY BE NECESSARY TO CUT AND COUPLE ADDITIONAL PIPE TO STANDARD MANIFOLD COMPONENTS IN THE FIELD.
- THIS CHAMBER SYSTEM WAS DESIGNED WITHOUT SITE-SPECIFIC INFORMATION ON SOIL CONDITIONS OR BEARING CAPACITY. THE SITE DESIGN ENGINEER IS RESPONSIBLE FOR DETERMINING THE SUITABILITY OF THE SOIL AND PROVIDING THE BEARING CAPACITY OF THE INSITU SOILS. THE BASE STONE DEPTH MAY BE INCREASED OR DECREASED ONCE THIS INFORMATION IS PROVIDED.



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DATE: 3-18-22 | DRAWN: BSW | PROJECT #: 8387266 | CHECKED: JEK
JEK | 3/30/22 | REVISED PER ISSUED FOR BID
DESCRIPTION | DATE | DRAWN (CHKD)

StormTech® Chamber System
888-892-2694 | WWW.STORMTECH.COM

ADS 4640 TRUEMAN BLVD HILLIARD, OH 43026

0 | 1/2" | 2"

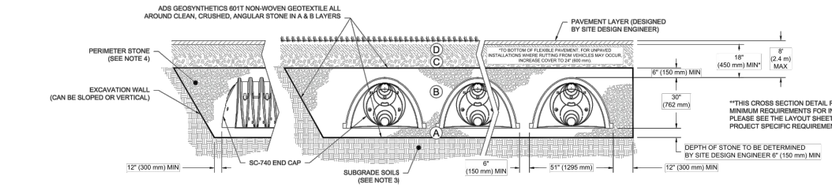
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SHEET 2 OF 5

ACCEPTABLE FILL MATERIALS: STORMTECH SC-740 CHAMBER SYSTEMS

	MATERIAL LOCATION	DESCRIPTION	AASHTO MATERIAL CLASSIFICATIONS	COMPACTION / DENSITY REQUIREMENT
D	**FINAL FILL:** FILL MATERIAL FOR LAYER 'D' STARTS FROM THE TOP OF THE 'C' LAYER TO THE BOTTOM OF FLEXIBLE PAVEMENT OR UNPAVED FINISHED GRADE ABOVE. NOTE THAT PAVEMENT SUBBASE MAY BE PART OF THE 'D' LAYER.	ANY SOIL/ROCK MATERIALS, NATIVE SOILS, OR PER ENGINEER'S PLANS. CHECK PLANS FOR PAVEMENT SUBGRADE REQUIREMENTS.	N/A	PREPARE PER SITE DESIGN ENGINEER'S PLANS. PAVED INSTALLATIONS MAY HAVE STRINGENT MATERIAL AND PREPARATION REQUIREMENTS.
C	**INITIAL FILL:** FILL MATERIAL FOR LAYER 'C' STARTS FROM THE TOP OF THE EMBEDMENT STONE ('B' LAYER) TO 18" (450 mm) ABOVE THE TOP OF THE CHAMBER. NOTE THAT PAVEMENT SUBBASE MAY BE A PART OF THE 'C' LAYER.	GRANULAR WELL-GRADED SOIL/AGGREGATE MIXTURES, <35% FINES OR PROCESSED AGGREGATE. MOST PAVEMENT SUBBASE MATERIALS CAN BE USED IN LIEU OF THIS LAYER.	AASHTO M145[1] A-1, A-2-4, A-3 OR AASHTO M43[1] 3, 357, 4, 467, 5, 56, 57, 6, 67, 68, 7, 78, 8, 89, 9, 10	BEGIN COMPACTIONS AFTER 12" (300 mm) OF MATERIAL OVER THE CHAMBERS IS REACHED. COMPACT ADDITIONAL LAYERS IN 6" (150 mm) MAX LIFTS TO A MIN. 95% PROCTOR DENSITY FOR WELL GRADED MATERIAL AND 95% RELATIVE DENSITY FOR PROCESSED AGGREGATE MATERIALS. ROLLER GROSS VEHICLE WEIGHT NOT TO EXCEED 12,000 lbs (53 kN). DYNAMIC FORCE NOT TO EXCEED 20,000 lbs (89 kN).
B	**EMBEDMENT STONE:** FILL SURROUNDING THE CHAMBERS FROM THE FOUNDATION STONE ('A' LAYER) TO THE 'C' LAYER ABOVE.	CLEAN, CRUSHED, ANGULAR STONE	AASHTO M43[1] 3, 357, 4, 467, 5, 56, 57	NO COMPACTION REQUIRED.
A	**FOUNDATION STONE:** FILL BELOW CHAMBERS FROM THE SUBGRADE UP TO THE FOOT (BOTTOM) OF THE CHAMBER.	CLEAN, CRUSHED, ANGULAR STONE	AASHTO M43[1] 3, 357, 4, 467, 5, 56, 57	PLATE COMPACT OR ROLL TO ACHIEVE A FLAT SURFACE[2,3]

PLEASE NOTE:
1. THE LISTED AASHTO DESIGNATIONS ARE FOR GRADATIONS ONLY. THE STONE MUST ALSO BE CLEAN, CRUSHED, ANGULAR. FOR EXAMPLE, A SPECIFICATION FOR #4 STONE WOULD STATE: "CLEAN, CRUSHED, ANGULAR NO. 4 (AASHTO M43) STONE".
2. STORMTECH COMPACTION REQUIREMENTS ARE MET FOR 'A' LOCATION MATERIALS WHEN PLACED AND COMPACTED IN 6" (150 mm) (MAX) LIFTS USING TWO FULL COVERAGES WITH A VIBRATORY COMPACTOR.
3. WHERE INFILTRATION SURFACES MAY BE COMPROMISED BY COMPACTION, FOR STANDARD DESIGN LOAD CONDITIONS, A FLAT SURFACE MAY BE ACHIEVED BY RAKING OR DRAGGING WITHOUT COMPACTION EQUIPMENT. FOR SPECIAL LOAD DESIGNS, CONTACT STORMTECH FOR COMPACTION REQUIREMENTS.
4. ONCE LAYER 'C' IS PLACED, ANY SOIL/MATERIAL CAN BE PLACED IN LAYER 'D' UP TO THE FINISHED GRADE. MOST PAVEMENT SUBBASE SOILS CAN BE USED TO REPLACE THE MATERIAL REQUIREMENTS OF LAYER 'C' OR 'D' AT THE SITE DESIGN ENGINEER'S DISCRETION.



NOTES:

1. CHAMBERS SHALL MEET THE REQUIREMENTS OF ASTM F2418, "STANDARD SPECIFICATION FOR POLYPROPYLENE (PP) CORRUGATED WALL STORMWATER COLLECTION CHAMBERS".
2. SC-740 CHAMBERS SHALL BE DESIGNED IN ACCORDANCE WITH ASTM F2787 "STANDARD PRACTICE FOR STRUCTURAL DESIGN OF THERMOPLASTIC CORRUGATED WALL STORMWATER COLLECTION CHAMBERS".
3. THE SITE DESIGN ENGINEER IS RESPONSIBLE FOR ASSESSING THE BEARING RESISTANCE (ALLOWABLE BEARING CAPACITY) OF THE SUBGRADE SOILS AND THE DEPTH OF FOUNDATION STONE WITH CONSIDERATION FOR THE RANGE OF EXPECTED SOIL MOISTURE CONDITIONS.
4. PERIMETER STONE MUST BE EXTENDED HORIZONTALLY TO THE EXCAVATION WALL FOR BOTH VERTICAL AND SLOPED EXCAVATION WALLS.
5. REQUIREMENTS FOR HANDLING AND INSTALLATION:
   - TO MAINTAIN THE WIDTH OF CHAMBERS DURING SHIPPING AND HANDLING, CHAMBERS SHALL HAVE INTEGRAL, INTERLOCKING STACKING LUGS.
   - TO ENSURE A SECURE JOINT DURING INSTALLATION AND BACKFILL, THE HEIGHT OF THE CHAMBER JOINT SHALL NOT BE LESS THAN 2".
   - TO ENSURE THE INTEGRITY OF THE ARCH SHAPE DURING INSTALLATION, a) THE ARCH STIFFNESS CONSTANT AS DEFINED IN SECTION 6.2.8 OF ASTM F2418 SHALL BE GREATER THAN OR EQUAL TO 550 LBS/IN/IN. AND b) TO RESIST CHAMBER DEFORMATION DURING INSTALLATION AT ELEVATED TEMPERATURES (ABOVE 73° F / 23° C), CHAMBERS SHALL BE PRODUCED FROM REFLECTIVE GOLD OR YELLOW COLORS.

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DATE: 3-18-22 | DRAWN: BSW | PROJECT #: 8387266 | CHECKED: JEK
JEK | 3/30/22 | REVISED PER ISSUED FOR BID
DESCRIPTION | DATE | DRAWN (CHKD)

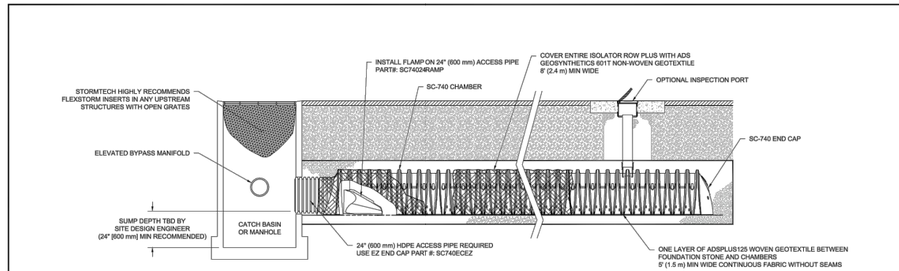
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ADS 4640 TRUEMAN BLVD HILLIARD, OH 43026

0 | 1/2" | 2"

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SHEET 3 OF 5



SC-740 ISOLATOR ROW PLUS DETAIL
NTS

INSPECTION & MAINTENANCE

STEP 1) INSPECT ISOLATOR ROW PLUS FOR SEDIMENT
- A. INSPECTION PORTS (IF PRESENT)
  - A.1. REMOVE/OPEN LID ON NYLOPLAST INLINE DRAIN
  - A.2. REMOVE AND CLEAN FLEXSTORM FILTER IF INSTALLED
  - A.3. USING A FLASHLIGHT AND STADIA ROD, MEASURE DEPTH OF SEDIMENT AND RECORD ON MAINTENANCE LOG
  - A.4. LOWER A CAMERA INTO ISOLATOR ROW PLUS FOR VISUAL INSPECTION OF SEDIMENT LEVELS (OPTIONAL)
  - A.5. IF SEDIMENT IS AT, OR ABOVE, 3" (80 mm) PROCEED TO STEP 2. IF NOT, PROCEED TO STEP 3.
- B. ALL ISOLATOR PLUS ROWS
  - B.1. REMOVE COVER FROM STRUCTURE AT UPSTREAM END OF ISOLATOR ROW PLUS
  - B.2. USING A FLASHLIGHT, INSPECT DOWN THE ISOLATOR ROW PLUS THROUGH OUTLET PIPE
    - i) MIRRORS ON POLES OR CAMERAS MAY BE USED TO AVOID A CONFINED SPACE ENTRY
    - ii) FOLLOW OSHA REGULATIONS FOR CONFINED SPACE ENTRY IF ENTERING MANHOLE
  - B.3. IF SEDIMENT IS AT, OR ABOVE, 3" (80 mm) PROCEED TO STEP 2. IF NOT, PROCEED TO STEP 3.

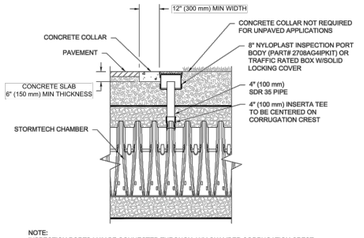
STEP 2) CLEAN OUT ISOLATOR ROW PLUS USING THE JETVAC PROCESS
- A. A FIXED CULVERT CLEANING NOZZLE WITH REAR FACING SPREAD OF 45° (1.1 m) OR MORE IS PREFERRED
- B. APPLY MULTIPLE PASSES OF JETVAC UNTIL BACKFLUSH WATER IS CLEAN
- C. VACUUM STRUCTURE SUMP AS REQUIRED

STEP 3) REPLACE ALL COVERS, GRATES, FILTERS, AND LIDS; RECORD OBSERVATIONS AND ACTIONS.

STEP 4) INSPECT AND CLEAN BASINS AND MANHOLES UPSTREAM OF THE STORMTECH SYSTEM.

NOTES

1. INSPECT EVERY 6 MONTHS DURING THE FIRST YEAR OF OPERATION. ADJUST THE INSPECTION INTERVAL BASED ON PREVIOUS OBSERVATIONS OF SEDIMENT ACCUMULATION AND HIGH WATER ELEVATIONS.
2. CONDUCT JETTING AND VACTORING ANNUALLY OR WHEN INSPECTION SHOWS THAT MAINTENANCE IS NECESSARY.



NOTE:
INSPECTION PORTS MAY BE CONNECTED THROUGH ANY CHAMBER CORRUGATION CREST.

4" PVC INSPECTION PORT DETAIL (SC SERIES CHAMBER)
NTS

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DATE: 3-18-22 | DRAWN: BSW | PROJECT #: 8387266 | CHECKED: JEK
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DESCRIPTION | DATE | DRAWN (CHKD)

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ADS 4640 TRUEMAN BLVD HILLIARD, OH 43026

0 | 1/2" | 2"

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SHEET 4 OF 5

Revision	By	Date

WHOLEE

PRELIMINARY NOT FOR CONSTRUCTION

Drawn By:	ICF
Approved By:	ATD
Date:	5.04.2022
Project No.:	BBD-01
Vertical Scale:	–
Horizontal Scale:	
Plotting Scale:	1
Drawing Name:	–

SOURCES FOR COMMUNITY INDEPENDENT LIVING

Little Rock, Pulaski County, Arkansas

UNDERGROUND DETENTION DETAILS 1

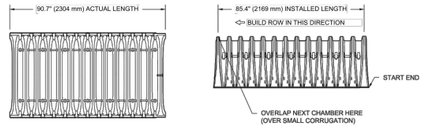
Issued for Review - 05.04.22

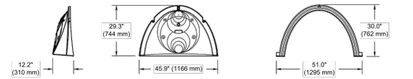
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Drawing Name: M:\SDX\PROJ\BBD-01\DWG\BBD-01 BASE.dwg Last Modified: May 04, 2022 - 8:59am Plotted on: May 04, 2022 - 9:33am by ahS198

SC-740 TECHNICAL SPECIFICATION

NTS

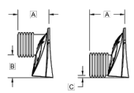




NOMINAL CHAMBER SPECIFICATIONS

SIZE (W X H X INSTALLED LENGTH)	51.0" X 30.0" X 85.4"	(1295 mm X 762 mm X 2169 mm)
CHAMBER STORAGE	45.9 CUBIC FEET	(1.30 m³)
MINIMUM INSTALLED STORAGE*	74.9 CUBIC FEET	(2.12 m³)
WEIGHT	75.0 lbs.	(33.6 kg)

*ASSUMES 6" (152 mm) STONE ABOVE, BELOW, AND BETWEEN CHAMBERS



PRE-FAB STUBS AT BOTTOM OF END CAP FOR PART NUMBERS ENDING WITH "B"
PRE-FAB STUBS AT TOP OF END CAP FOR PART NUMBERS ENDING WITH "T"
PRE-CORED END CAPS END WITH "PC"

PART #	STUB	A	B	C
SC740EPE06T / SC740EPE06TPC	6" (150 mm)	10.9" (277 mm)	18.5" (470 mm)	—
SC740EPE06B / SC740EPE06BPC			—	0.5" (13 mm)
SC740EPE08T /SC740EPE08TPC	8" (200 mm)	12.2" (310 mm)	16.5" (419 mm)	—
SC740EPE08B / SC740EPE08BPC			—	0.6" (15 mm)
SC740EPE10T / SC740EPE10TPC	10" (250 mm)	13.4" (340 mm)	14.5" (368 mm)	—
SC740EPE10B / SC740EPE10BPC			—	0.7" (18 mm)
SC740EPE12T / SC740EPE12TPC	12" (300 mm)	14.7" (373 mm)	12.5" (318 mm)	—
SC740EPE12B / SC740EPE12BPC			—	1.2" (30 mm)
SC740EPE15T / SC740EPE15TPC	15" (375 mm)	18.4" (467 mm)	9.0" (229 mm)	—
SC740EPE15B / SC740EPE15BPC			—	1.3" (33 mm)
SC740EPE18T / SC740EPE18TPC	18" (450 mm)	19.7" (500 mm)	5.0" (127 mm)	—
SC740EPE18B / SC740EPE18BPC			—	1.6" (41 mm)
SC740ECEZ*	24" (600 mm)	18.5" (470 mm)	—	0.1" (3 mm)

ALL STUBS, EXCEPT FOR THE SC740ECEZ ARE PLACED AT BOTTOM OF END CAP SUCH THAT THE OUTSIDE DIAMETER OF THE STUB IS FLUSH WITH THE BOTTOM OF THE END CAP. FOR ADDITIONAL INFORMATION CONTACT STORMTECH AT 1-888-892-2694.

* FOR THE SC740ECEZ THE 24" (600 mm) STUB LIES BELOW THE BOTTOM OF THE END CAP APPROXIMATELY 1.75" (44 mm). BACKFILL MATERIAL SHOULD BE REMOVED FROM BELOW THE N-12 STUB SO THAT THE FITTING SITS LEVEL.

NOTE: ALL DIMENSIONS ARE NOMINAL

SOURCES OF COMMUNITY INDEPENDENT LIVING

DATE: 3-18-22 | DRAWN: BSW | PROJECT # 8287268 | CHECKED: JEK

REV	BY	DATE	DESCRIPTION
A	BSW	4/4/22	REVISED PER ADS STANDARD DWG.

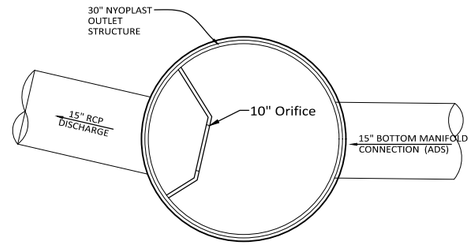
THIS DRAWING AND ADS PATENTED BASED ON STORMTECH PRODUCTS. PLAN SHEETS DESIGNED FOR USE WITH STORMTECH CHAMBERS ONLY. REVIEW THIS DRAWING PRIOR TO CONSTRUCTION. IT IS THE ULTIMATE RESPONSIBILITY OF THE SITE DESIGN ENGINEER TO ENSURE THAT THE SITE DESIGN COMPLIES WITH ALL LOCAL REGULATIONS AND REQUIREMENTS. THE SITE DESIGN ENGINEER SHALL REVIEW THIS DRAWING PRIOR TO CONSTRUCTION. IT IS THE RESPONSIBILITY OF THE SITE DESIGN ENGINEER TO ENSURE THAT THE SITE DESIGN COMPLIES WITH ALL LOCAL REGULATIONS AND REQUIREMENTS.

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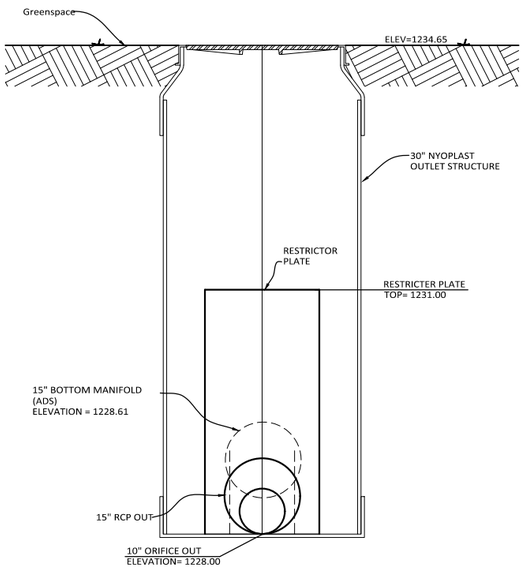
4640 TRUEMAN BLVD
HILLIARD, OH 43026

ADS

SHEET 5 OF 5



PLAN



1 DETENTION OUTLET STRUCTURE
(Not to Scale)

Revision	By	Date

WHOLEE

PRELIMINARY NOT FOR CONSTRUCTION

Drawn By:	ICF
Approved By:	ATD
Date:	5.04.2022
Project No.:	BBD–01
Vertical Scale:	–
Horizontal Scale:	
Plotting Scale:	1
Drawing Name:	–

SOURCES FOR COMMUNITY INDEPENDENT LIVING

Little Rock, Pulaski County, Arkansas

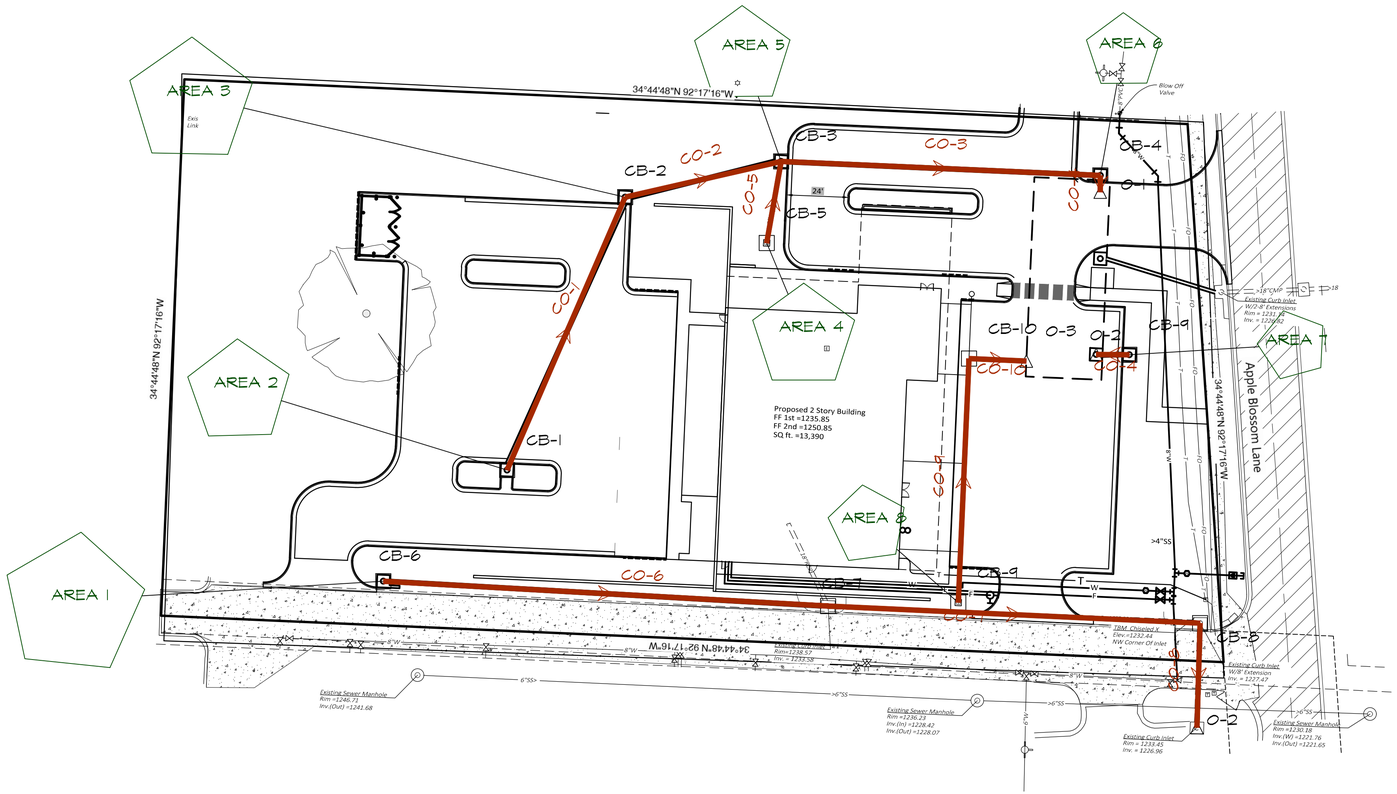
UNDERGROUND DETENTION DETAILS 2

Issued for Review – 05.04.22

Sheet No.: 13 of 18

Drawing Name: M:\SOS\PROJ\BBD-01\DRAFT\NOV\BBD-01 BASE.dwg Last Modified: May 04, 2022 - 8:59am Plotted on: May 04, 2022 - 9:33am by ah5198

APPENDIX
Hydraulic Calculations



Revision	By	Date

WHOLEE

Drawn By:	ICF
Approved By:	AD
Date:	05-04-2022
Project No.:	BBD-01

Vertical Scale:	–
Horizontal Scale:	1"=30'
Plotting Scale:	1-1
Drawing Name:	DAM

SOURCES FOR COMMUNITY INDEPENDENT LIVING

Little Rock, Pulaski County, Arkansas

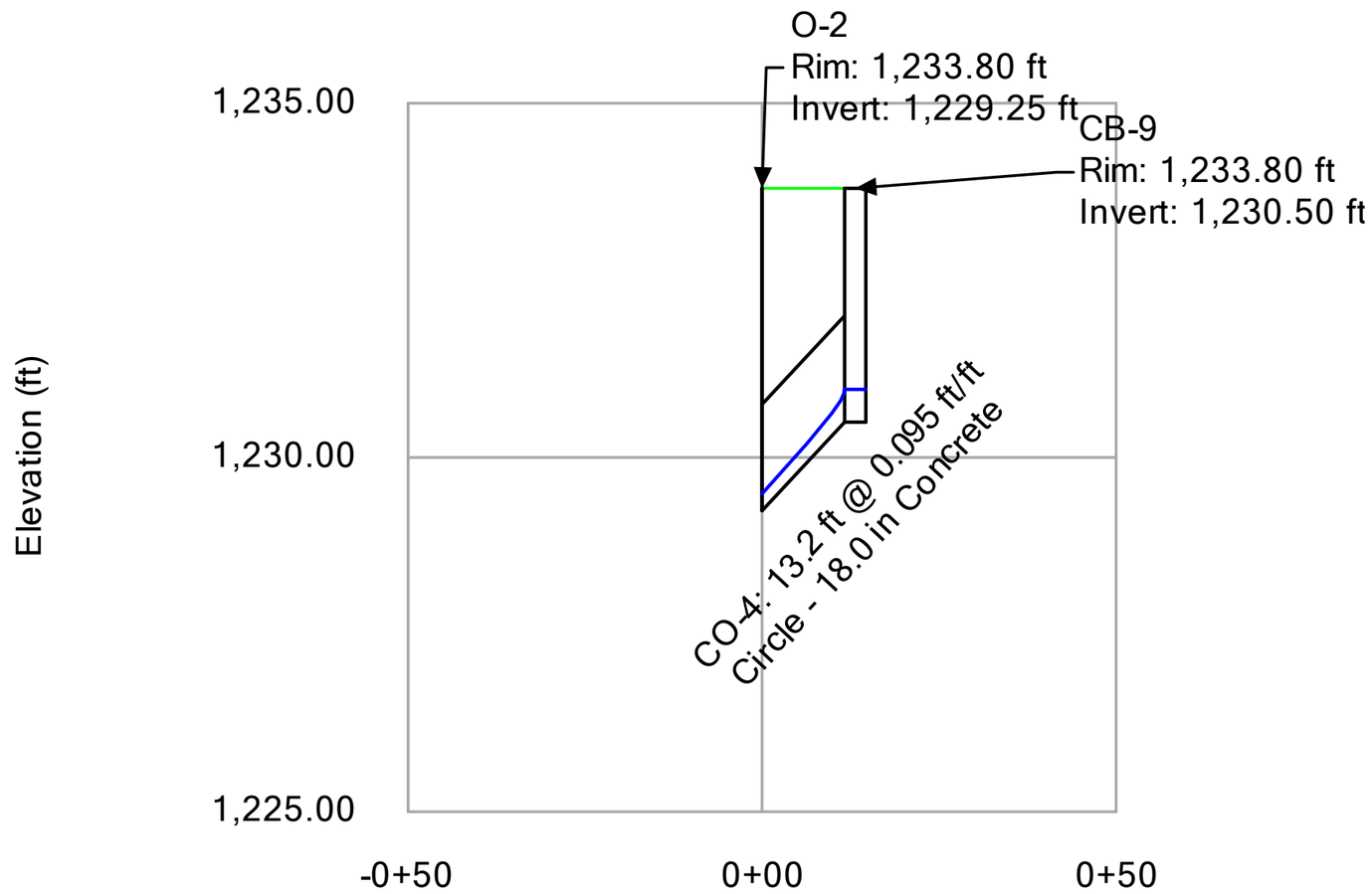
APPENDIX D: STORMCAD LAYOUT

Issued for Review - 05.04.22

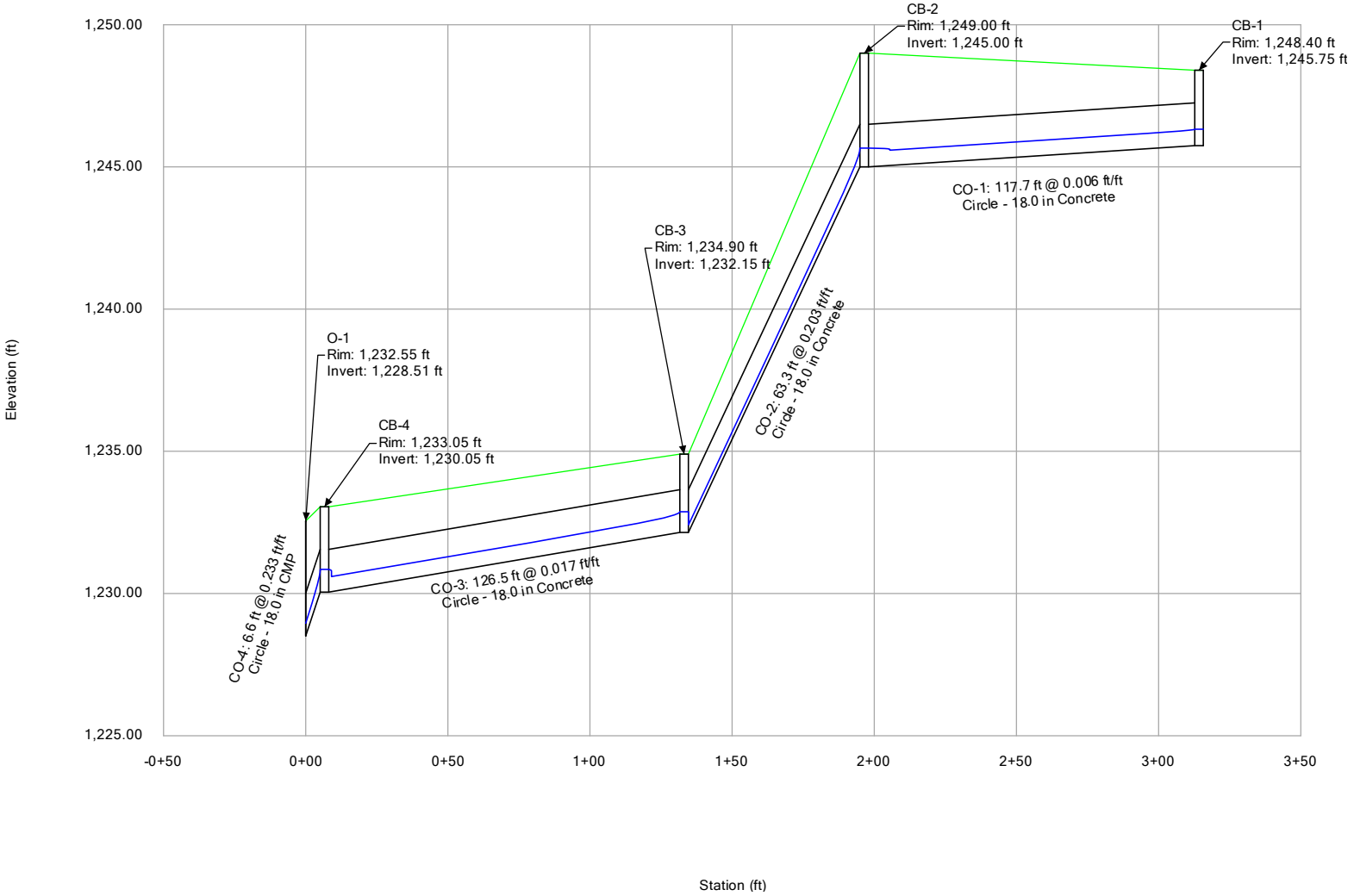
Sheet No.: 1 of 1

25-Year Storm Event

Profile Report
Engineering Profile - East Pipe To ADS System

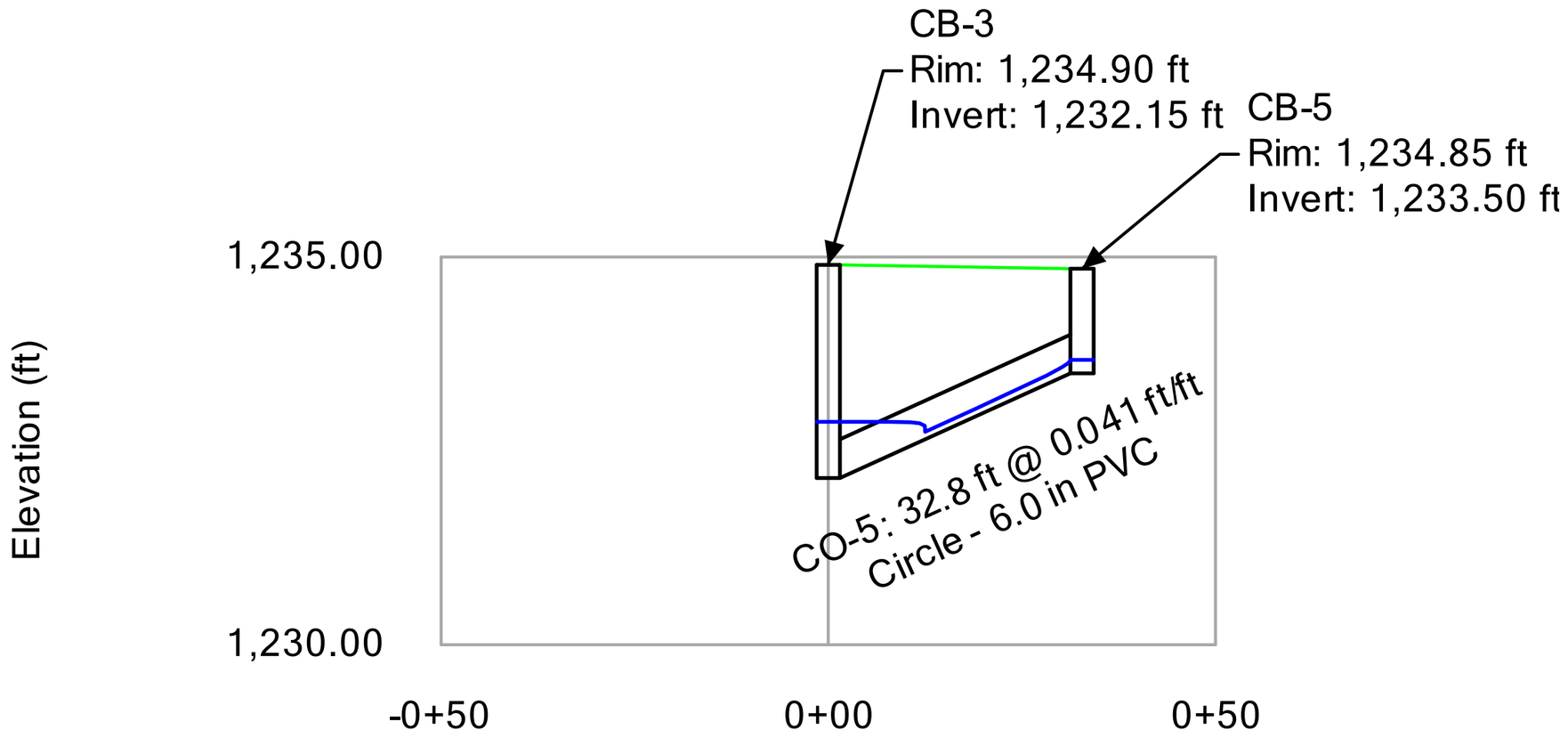


Profile Report
Engineering Profile - North System To ADS System

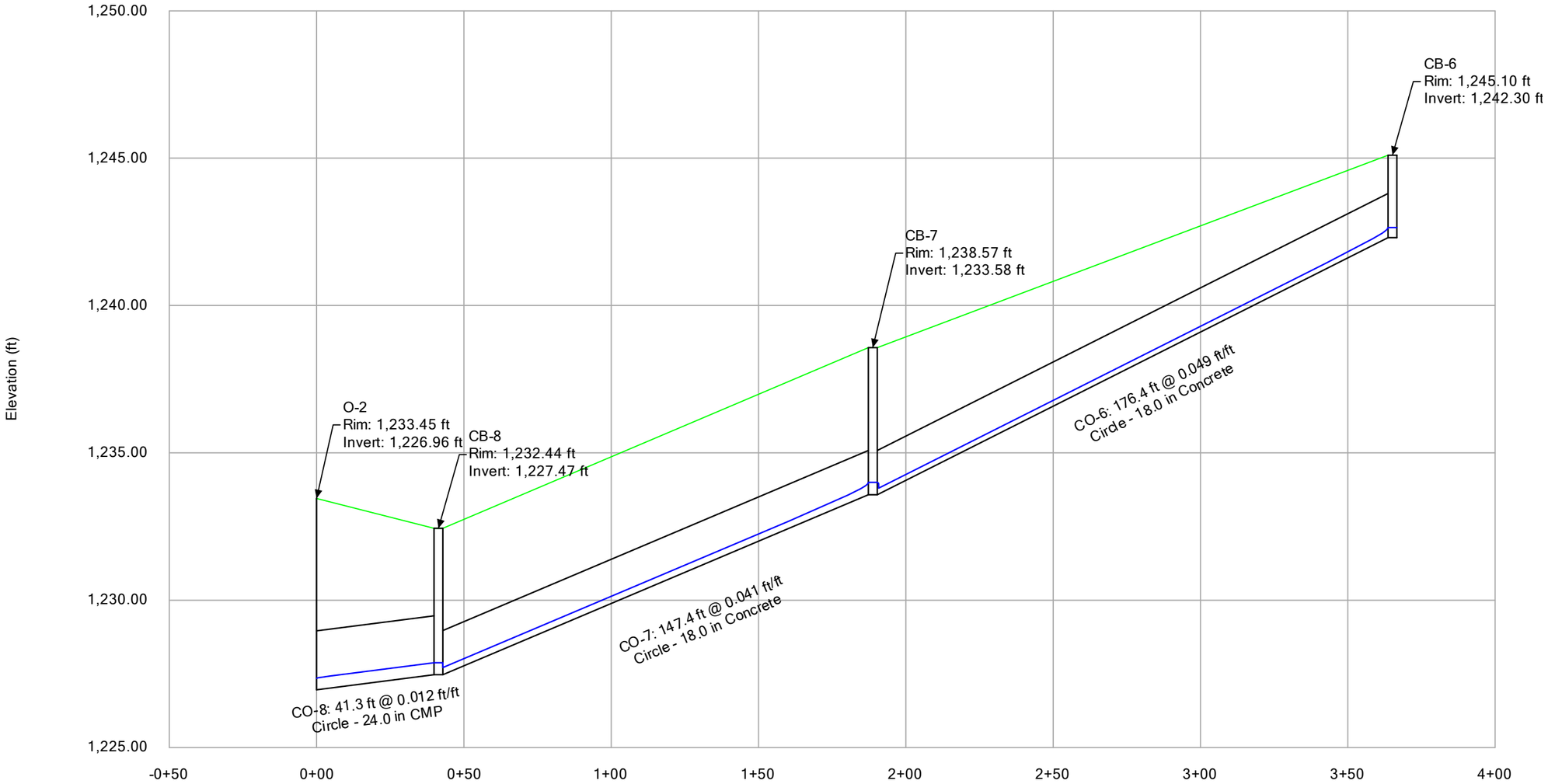


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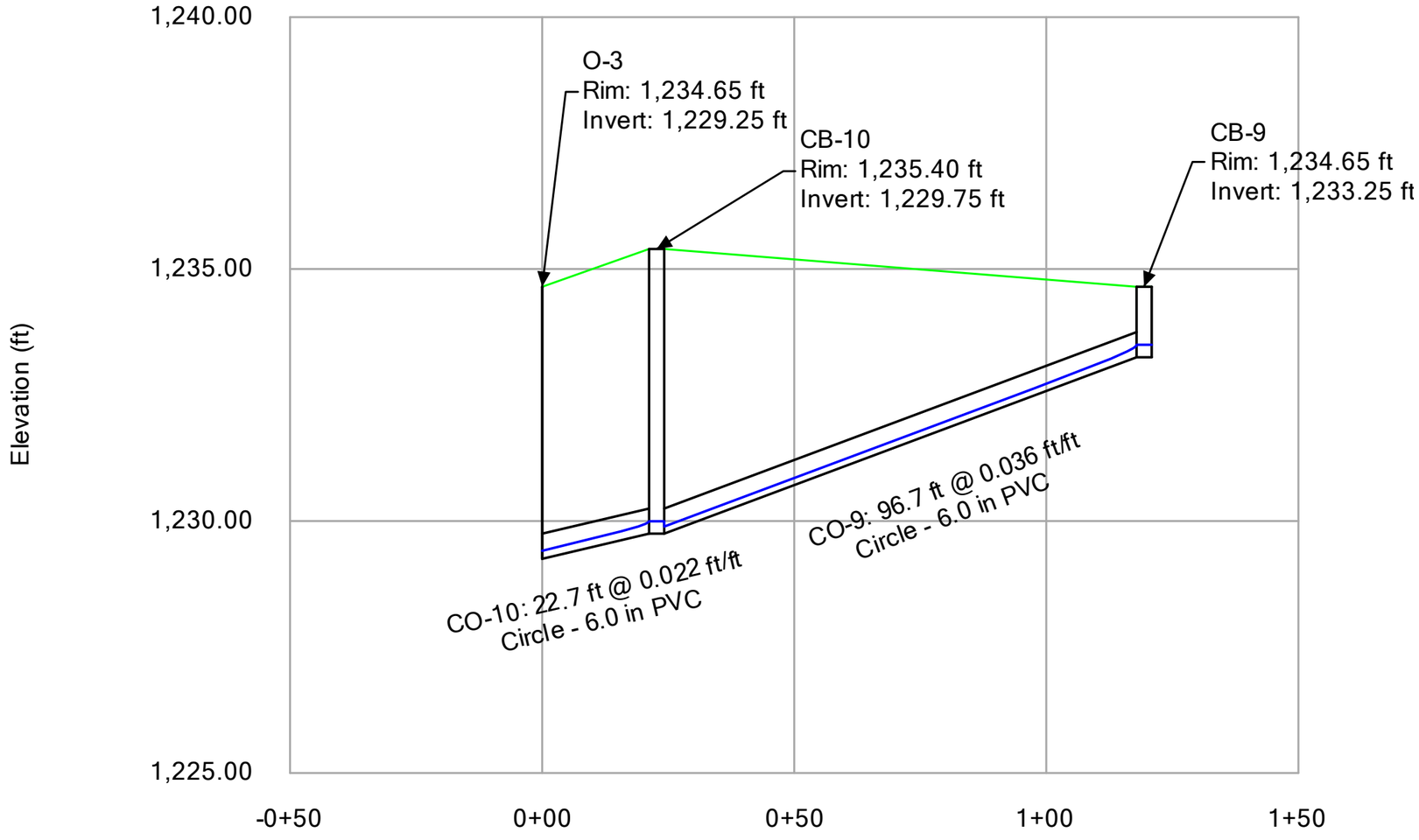
Profile Report
Engineering Profile - North Yard Drain



Profile Report
Engineering Profile - South System To Study Point B



Profile Report
Engineering Profile - South Yard Drain



Scenario: Base
Current Time Step: 0.000 h
FlexTable: Catch Basin Table

Label	Elevation (Rim) (ft)	Elevation (Invert) (ft)	Curb Opening Length (ft)	Capture Efficiency (Calculated) (%)	Flow (Captured) (cfs)	Hydraulic Grade Line (In) (ft)	Spread / Top Width (ft)	Depth (Gutter) (in)	Road Cross Slope (ft/ft)
CB-1	1,248.40	1,245.75	8.0	100.0	2.30	1,246.32	15.9	6.9	0.013
CB-2	1,249.00	1,245.00	4.0	100.0	0.83	1,245.66	3.6	6.1	0.040
CB-3	1,234.90	1,232.15	4.0	100.0	0.60	1,232.87	1.7	5.8	0.068
CB-4	1,233.05	1,230.05	8.0	100.0	0.87	1,230.85	2.2	5.7	0.050
CB-5	1,234.85	1,233.50	-	100.0	0.12	1,233.67	0.0	(N/A)	0.020
CB-6	1,245.10	1,242.30	4.0	65.6	0.84	1,242.64	6.4	1.5	0.020
CB-7	1,238.57	1,233.58	4.0	94.0	0.50	1,234.00	4.9	1.2	0.020
CB-8	1,232.44	1,227.47	-	100.0	0.03	1,227.88	1.4	0.3	0.020
CB-9	1,233.80	1,230.50	4.0	100.0	1.52	1,230.96	6.1	7.0	0.035
CB-9	1,234.65	1,233.25	-	100.0	0.24	1,233.50	0.0	(N/A)	0.020
CB-10	1,235.40	1,229.75	-	100.0	0.00	1,230.00	0.0	0.0	0.020

Scenario: Base
Current Time Step: 0.000 h
FlexTable: Catchment Table

Label	Outflow Element	Area (User Defined) (acres)	Runoff Coefficient (Rational)	Time of Concentration (min)	Catchment Intensity (in/h)	Flow (Total Out) (cfs)
AREA 1	CB-6	0.510	0.398	-	6.297	1.29
AREA 2	CB-1	0.620	0.593	-	6.224	2.30
AREA 3	CB-2	0.300	0.463	-	5.943	0.83
AREA 4	CB-5	0.030	0.560	6.000	7.294	0.12
AREA 5	CB-3	0.160	0.495	5.000	7.560	0.60
AREA 6	CB-4	0.150	0.885	-	6.483	0.87
AREA 7	CB-9	0.210	0.950	5.000	7.560	1.52
AREA 8	CB-9	0.060	0.528	5.000	7.560	0.24

Scenario: Base
Current Time Step: 0.000 h
FlexTable: Conduit Table

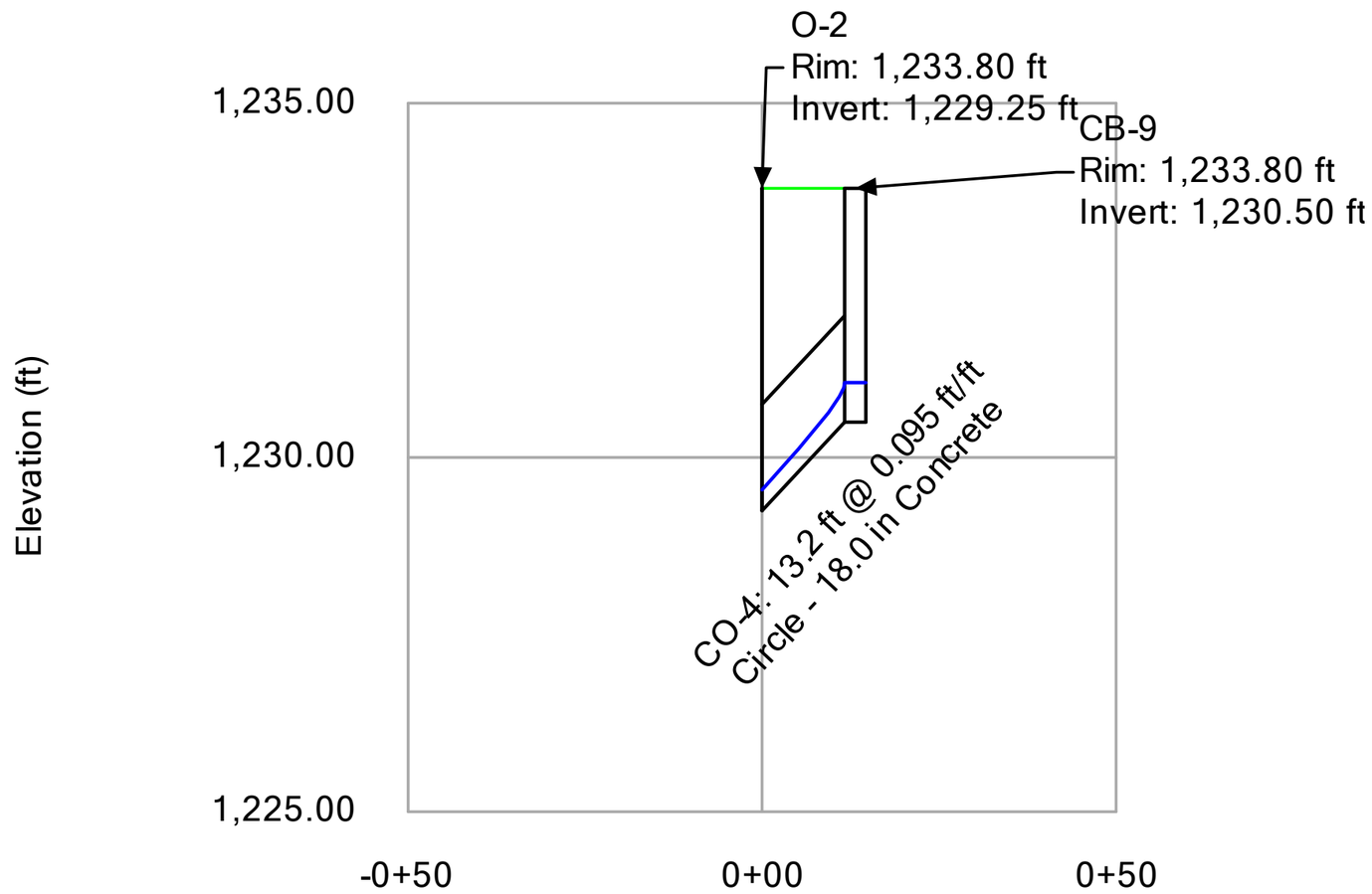
Label	Start Node	Stop Node	Invert (Start) (ft)	Invert (Stop) (ft)	Length (Scaled) (ft)	Slope (Calculated) (ft/ft)	Diameter (in)	Flow (cfs)	Size (Display)	Velocity (ft/s)	Depth (Out) (ft)
CO-1	CB-1	CB-2	1,245.75	1,245.00	117.7	0.006	18.0	2.30	18 inch	4.05	0.66
CO-2	CB-2	CB-3	1,245.00	1,232.15	63.3	0.203	18.0	3.03	18 inch	15.03	0.72
CO-3	CB-3	CB-4	1,232.15	1,230.05	126.5	0.017	18.0	3.60	18 inch	6.47	0.80
CO-4	CB-9	O-2	1,230.50	1,229.25	13.2	0.095	18.0	1.52	18 inch	9.37	0.24
CO-4	CB-4	O-1	1,230.05	1,228.51	6.6	0.233	18.0	4.32	18 inch	11.33	0.44
CO-5	CB-5	CB-3	1,233.50	1,232.15	32.8	0.041	6.0	0.12	6 inch	4.56	0.72
CO-6	CB-6	CB-7	1,242.30	1,233.58	176.4	0.049	18.0	0.84	18 inch	6.26	0.42
CO-7	CB-7	CB-8	1,233.58	1,227.47	147.4	0.041	18.0	1.24	18 inch	6.58	0.24
CO-8	CB-8	O-2	1,227.47	1,226.96	41.3	0.012	24.0	1.24	24 inch	2.70	0.38
CO-9	CB-9	CB-10	1,233.25	1,229.75	96.7	0.036	6.0	0.24	6 inch	5.30	0.14
CO-10	CB-10	O-3	1,229.75	1,229.25	22.7	0.022	6.0	0.24	6 inch	4.43	0.16

Scenario: Base
Current Time Step: 0.000 h
FlexTable: Outfall Table

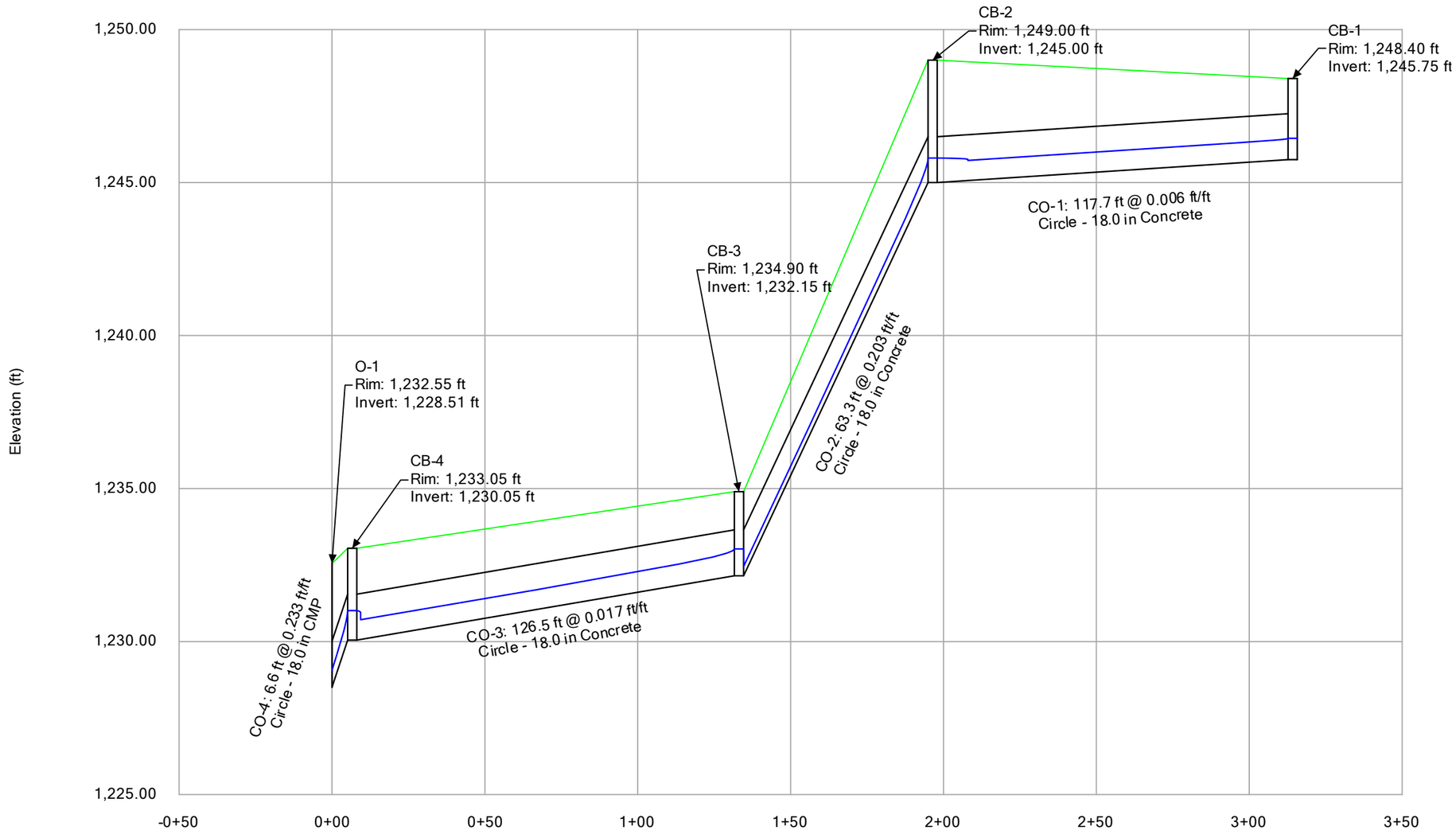
Label	Elevation (Ground) (ft)	Elevation (Invert) (ft)	Boundary Condition Type	Hydraulic Grade (ft)	Flow (Total Out) (cfs)	System Flow Time (min)	System Intensity (in/h)	System CA (acres)
O-1	1,232.55	1,228.51	Free Outfall	1,228.95	4.32	11.486	5.835	0.735
O-2	1,233.80	1,229.25	Free Outfall	1,229.49	1.52	5.023	7.554	0.199
O-2	1,233.45	1,226.96	Free Outfall	1,227.34	1.23	10.848	6.004	0.203
O-3	1,234.65	1,229.25	Free Outfall	1,229.41	0.24	5.390	7.456	0.032

100-Year Storm Event

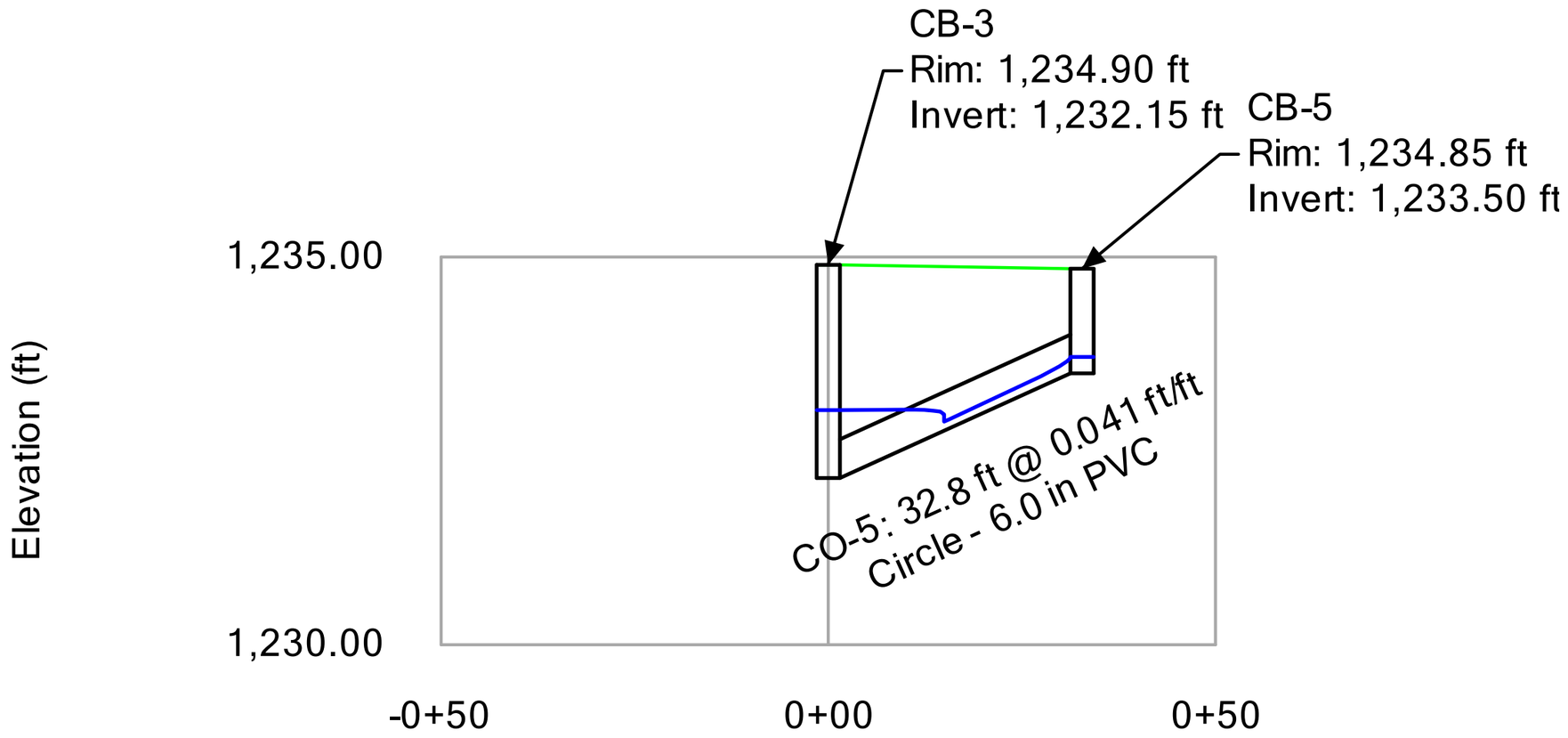
Profile Report
Engineering Profile - East Pipe To ADS System



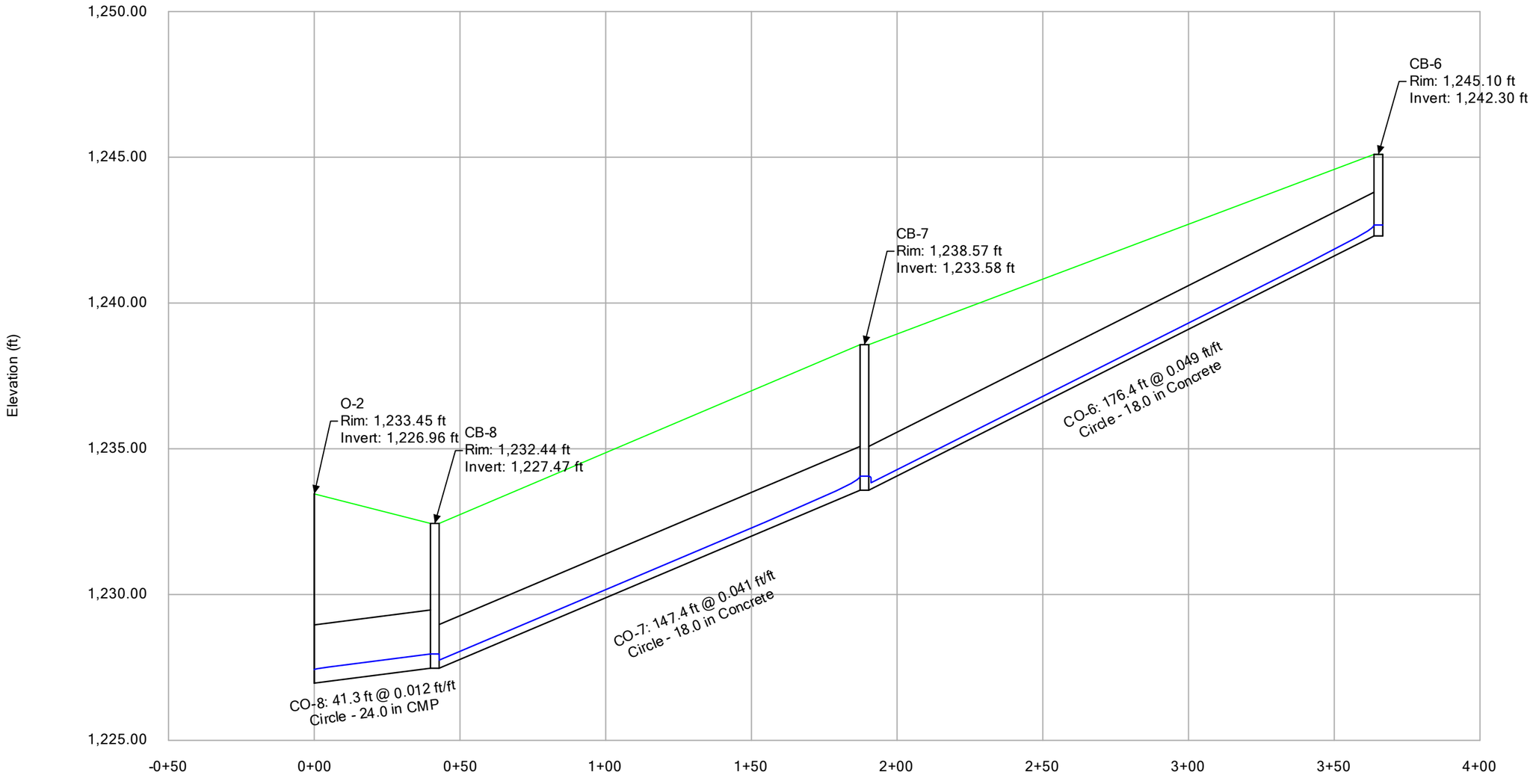
Profile Report
Engineering Profile - North System To ADS System



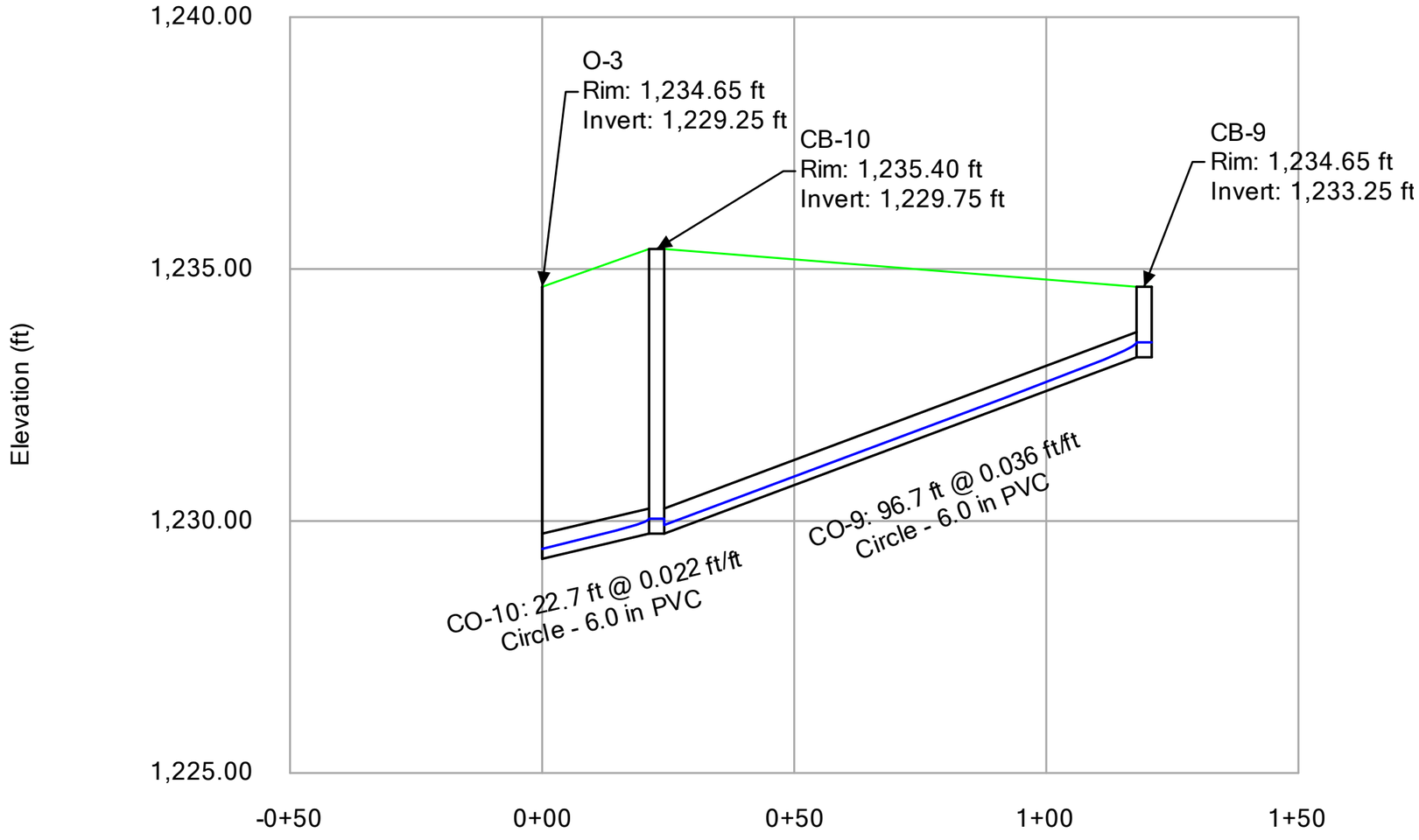
Profile Report
Engineering Profile - North Yard Drain



Profile Report
Engineering Profile - South System To Study Point B



Profile Report
Engineering Profile - South Yard Drain



Scenario: Base
Current Time Step: 0.000 h
FlexTable: Catch Basin Table

Label	Elevation (Rim) (ft)	Elevation (Invert) (ft)	Curb Opening Length (ft)	Capture Efficiency (Calculated) (%)	Flow (Captured) (cfs)	Hydraulic Grade Line (In) (ft)	Spread / Top Width (ft)	Depth (Gutter) (in)	Road Cross Slope (ft/ft)
CB-1	1,248.40	1,245.75	8.0	100.0	3.31	1,246.44	20.2	7.6	0.013
CB-2	1,249.00	1,245.00	4.0	100.0	1.19	1,245.80	4.5	6.6	0.040
CB-3	1,234.90	1,232.15	4.0	100.0	0.86	1,233.03	2.2	6.2	0.068
CB-4	1,233.05	1,230.05	8.0	100.0	1.25	1,231.01	2.7	6.0	0.050
CB-5	1,234.85	1,233.50	-	100.0	0.18	1,233.71	0.0	(N/A)	0.020
CB-6	1,245.10	1,242.30	4.0	55.7	1.03	1,242.68	7.4	1.8	0.020
CB-7	1,238.57	1,233.58	4.0	77.0	0.75	1,234.06	6.1	1.5	0.020
CB-8	1,232.44	1,227.47	-	100.0	0.23	1,227.96	2.9	0.7	0.020
CB-9	1,233.80	1,230.50	4.0	100.0	2.18	1,231.06	7.8	7.7	0.035
CB-9	1,234.65	1,233.25	-	100.0	0.35	1,233.55	0.0	(N/A)	0.020
CB-10	1,235.40	1,229.75	-	100.0	0.00	1,230.05	0.0	0.0	0.020

Scenario: Base
Current Time Step: 0.000 h
FlexTable: Catchment Table

Label	Outflow Element	Area (User Defined) (acres)	Runoff Coefficient (Rational)	Time of Concentration (min)	Catchment Intensity (in/h)	Flow (Total Out) (cfs)
AREA 1	CB-6	0.510	0.398	-	9.044	1.85
AREA 2	CB-1	0.620	0.593	-	8.941	3.31
AREA 3	CB-2	0.300	0.463	-	8.544	1.19
AREA 4	CB-5	0.030	0.560	6.000	10.454	0.18
AREA 5	CB-3	0.160	0.495	5.000	10.830	0.86
AREA 6	CB-4	0.150	0.885	-	9.308	1.25
AREA 7	CB-9	0.210	0.950	5.000	10.830	2.18
AREA 8	CB-9	0.060	0.528	5.000	10.830	0.35

Scenario: Base
Current Time Step: 0.000 h
FlexTable: Conduit Table

Label	Start Node	Stop Node	Invert (Start) (ft)	Invert (Stop) (ft)	Length (Scaled) (ft)	Slope (Calculated) (ft/ft)	Diameter (in)	Flow (cfs)	Size (Display)	Velocity (ft/s)	Depth (Out) (ft)
CO-1	CB-1	CB-2	1,245.75	1,245.00	117.7	0.006	18.0	3.31	18 inch	4.46	0.80
CO-2	CB-2	CB-3	1,245.00	1,232.15	63.3	0.203	18.0	4.36	18 inch	16.71	0.88
CO-3	CB-3	CB-4	1,232.15	1,230.05	126.5	0.017	18.0	5.17	18 inch	7.15	0.96
CO-4	CB-9	O-2	1,230.50	1,229.25	13.2	0.095	18.0	2.18	18 inch	10.43	0.29
CO-4	CB-4	O-1	1,230.05	1,228.51	6.6	0.233	18.0	6.23	18 inch	12.56	0.55
CO-5	CB-5	CB-3	1,233.50	1,232.15	32.8	0.041	6.0	0.18	6 inch	5.08	0.88
CO-6	CB-6	CB-7	1,242.30	1,233.58	176.4	0.049	18.0	1.03	18 inch	6.63	0.48
CO-7	CB-7	CB-8	1,233.58	1,227.47	147.4	0.041	18.0	1.63	18 inch	7.15	0.49
CO-8	CB-8	O-2	1,227.47	1,226.96	41.3	0.012	24.0	1.79	24 inch	3.00	0.46
CO-9	CB-9	CB-10	1,233.25	1,229.75	96.7	0.036	6.0	0.35	6 inch	5.88	0.17
CO-10	CB-10	O-3	1,229.75	1,229.25	22.7	0.022	6.0	0.34	6 inch	4.89	0.20

Scenario: Base
Current Time Step: 0.000 h
FlexTable: Outfall Table

Label	Elevation (Ground) (ft)	Elevation (Invert) (ft)	Boundary Condition Type	Hydraulic Grade (ft)	Flow (Total Out) (cfs)	System Flow Time (min)	System Intensity (in/h)	System CA (acres)
O-1	1,232.55	1,228.51	Free Outfall	1,229.06	6.23	11.447	8.406	0.735
O-2	1,233.80	1,229.25	Free Outfall	1,229.54	2.18	5.021	10.822	0.199
O-2	1,233.45	1,226.96	Free Outfall	1,227.42	1.77	10.766	8.662	0.203
O-3	1,234.65	1,229.25	Free Outfall	1,229.45	0.34	5.352	10.698	0.032

APPENDIX
Maintenance Agreement

Inspection and Maintenance Agreement
for

Sources for Community Independent Living

Apple Blossom Lane

Little Rock, Pulaski County, Arkansas

A. GENERAL

In accordance with the requirements stated by the Little Rock Stormwater Drainage Manual, this document outlines the aspects of long-term maintenance for the developed site.

Components

The drainage system on-site will be as shown in the submittal Appendix. Maintenance of these on-site BMP's will be crucial to the longevity and safety of the developed site and the existing roadway.

B. INSPECTION

Structural Inspection
The drainage system shall be professionally inspected twice a year. The thorough inspections will take place in months of June and October to ensure the culverts have been checked before and after the rainy season (November-May).

Erosion
Along with structural inspection, the drainage system shall be inspected for signs of erosion before and after the rainy season.

Clearance of Debris
During the rainy season, inlets to the drainage system will be inspected once a month for debris such as branches, leaves, trash, and sediment to ensure the culverts function as designed.

Bioretention

The bioretention structure maintenance and inspection will adhere to the recommendations of WQ-04 (Bioretention) of the submittal Appendix of the Little Rock Stormwater Drainage Manual.

C. DOCUMENTATION

A record shall be kept for all structural inspections and major maintenance activities. The report of inspections will be turned in to the City of Little Rock annually.

D. RESPONSIBILITY

The property owner will be responsible for the maintenance services.

E. CONCLUSION

As demonstrated above by the topics covered, the Inspections and Maintenance Agreement meets the requirements of the City of Little Rock.

I, Saul Kip, Licensed Contractor No. 12345 in the State of Arkansas, hereby certify that the long-term maintenance and inspection items contained in this agreement have been prepared in accordance with sound engineering practice and principles, and the requirements of the City of Little Rock. Further, I hereby acknowledge that the review of the long-term maintenance and inspection plan by the City of Little Rock or its representatives cannot and does not relieve me from any professional responsibility or liability.

Black Bear Builders, Inc.

_____

Saul Kip, G.C.

Site Developer

Appendix C: Stormwater BMPs & SCMs

Stormwater Best Management Practices (BMPs) and Stormwater Control Measures (SCMs)

Removal Efficiencies

Table C-1 Design Pollutant Removal Efficiencies for Structural Stormwater Controls

WQ	Structural Control	Total Suspended Solids	Total Phosphorus	Total Nitrogen	Fecal Coliform	Metal
	General Application					
02	Stormwater Ponds	80	50	30	70*	50
03	Stormwater Wetlands	80	40	30	70*	50
04	Bioretention Areas	80	60	50	---	80
05	Urban Bioretention	80	60	50	---	80
06	Sand Filters	80	50	25	40	50
	Limited Application					
07	Organic Filter	80	60	40	50	75
08	Underground Sand Filter	80	50	25	40	50
09	Submerged Gravel Wetland	80	50	20	70	50
10	Gravity (Oil-Grit) Separator	40	5	5	---	---
11	Proprietary Systems	***	***	***	***	***
	Pre-Treatment					
12	Permeable Pavement	80	50	50	---	60
13	Infiltration Devices	80	60	60	90	90
14	Water Quality Swale	80	25	40	---	20
15	Downspout Disconnect	60	25	25	---	40
16	Grass Channel	50	25	20	---	30
17	Vegetated Filter Strips	60	20	20	---	40
18	Green Roof	80	50	50	---	---

* If no resident waterfowl population present.

*** The performance of specific proprietary commercial devices and systems must be provided by the manufacturer and should be verified by independent third-party sources and data.

--- Insufficient data to provide design removal efficiency.

WQ-01 Floatables BMPs

1. Description

Floatables control focuses on using preventative measures to help reduce the amounts of floatable materials which may enter natural streams. “Floatable materials” means sediment, debris, trash, and other floating, or suspended solids. The reduction of solids and floatables is an important component of water quality management since they are one of the most visible pollutants that impact our receiving waters.

Floatables control technologies are designed to reduce or eliminate the visible trash and debris from stormwater runoff. Examples of floatables control technologies include:

- **Baffles** – Baffles are simple floatables control devices that are typically installed in outlet structures or manholes. They consist of vertical steel plates or concrete beams that extend from the top of the structure to just below the top of the weir. During a storm event, floatables are retained by the baffles while water passes under the baffles, over the weir, and into the receiving water body.
- **Screens and Trash racks** – Screens and trash racks consist of a series of vertical and horizontal bars or wires that trap floatables while allowing water to pass through the openings between the bars or wires. Screens can be installed at select points within a stormwater system to capture floatables and prevent their discharge to receiving water bodies.
- **Catch basin modifications** – Inlets, or catch basins, are often modified to prevent floatables from entering the system. Inlet grates installed at the top of the catch basins reduce the amount of street litter and debris that enters the catch basin. If floatables enter the basin through these grates, they can be collected in colander-like structures called trash buckets installed in the basin beneath the grate. These structures retain floatables while letting water flow through to the stormwater system.
- **Proprietary systems** – Multiple proprietary systems, either standalone and in conjunction with other stormwater control measures, can effectively remove floatables. Examples include skimmers, hydrodynamic separators, catch basin inserts, and netting systems.

2. Stormwater Management Suitability

Floatables control is a part of all stormwater quality management either as a component of another stormwater control measure, within the stormwater drainage system, or as a standalone measure. Regular maintenance and removal of collected trash and debris is required for all floatable control technologies. Floatable control measures have limited effectiveness at removing fine, soluble pollutants such as nutrients, metals, and bacteria.

3. Design Criteria and Specifications

- Floatables control is typically used on high density residential and commercial sites.
- Most floatable control measures are designed based on flowrate, which contrasts with other stormwater control measures that are designed based on a Water Quality Volume.
- Floatables control measures are typically used as pre-treatment for other stormwater control measures.

4. Typical Details

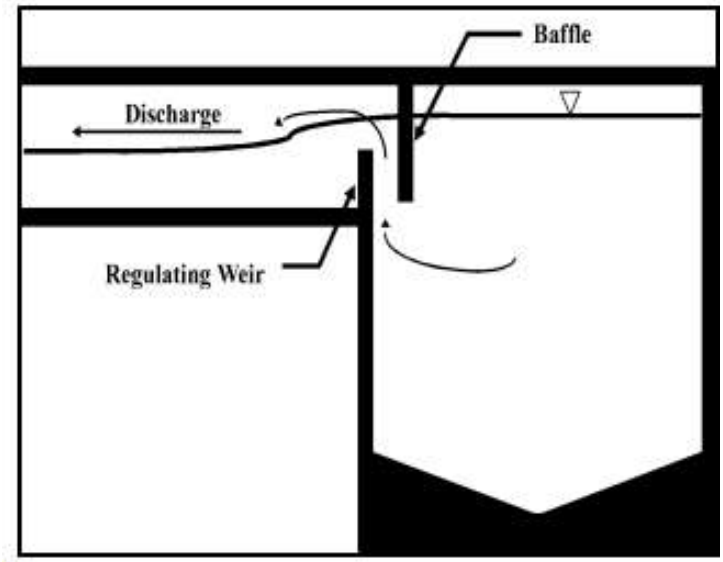


Figure C-1. Baffle in CSO Regulation

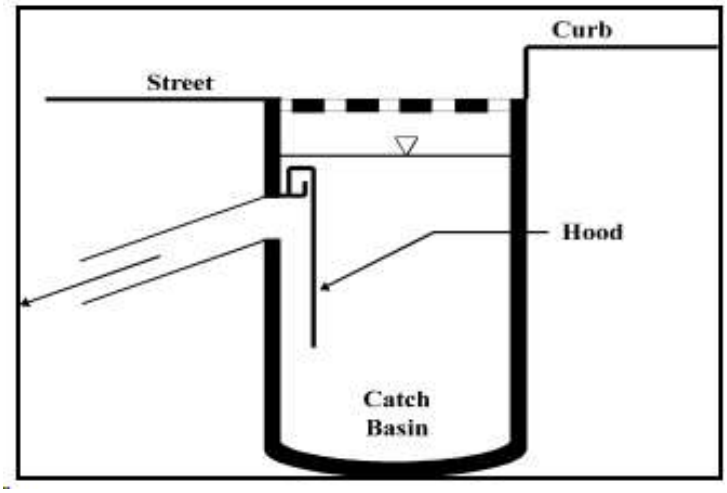


Figure C-2. Modified Catch Basin with Hood

5. Inspection and Maintenance Requirements

Table C-2. Typical Maintenance Activities for Underground Sand Filters

Activity	Schedule
• Emptying catch basin inserts, trash buckets, or nets	Quarterly and following large storm events
• Remove accumulated debris from trash racks, screens, and grate inlets	As needed
• Remove accumulated oil and floatables in catch basin or manholes	As needed, (typically every 3-6 months)

6. References

Combined Sewer Overflow Technology Fact Sheet: Floatables Control (epa.gov)

Stormwater Liter, Trash & Debris Removal | CSO Trash Control (stormtrap.com)

NPDES: Stormwater Best Management Practices, Stormwater Inlet Controls (epa.gov)

Chapter 7 Solids and Floatable Control (nj.gov)

WQ-02 Stormwater Ponds

1. Description

Stormwater ponds (also referred to as retention ponds, wet ponds, or wet extended detention ponds) are constructed stormwater retention basins that have a permanent (dead storage) pool of water throughout the year. They can be created by excavating an already existing natural depression or through the construction of embankments.

In a stormwater pond, runoff from each rain event is detained and treated in the pool through gravitational settling and biological uptake until it is displaced by runoff from the next storm. The permanent pool also serves to protect deposited sediments from resuspension. Above the permanent pool level, additional temporary storage (live storage) is provided for runoff quantity control. The upper stages of a stormwater pond are designed to provide extended detention of the 1-year storm for downstream channel protection, as well as normal detention of larger storm events.

Stormwater ponds are among the most cost-effective and widely used stormwater practices. A well-designed and landscaped pond can be an aesthetic feature on a development site when planned and located properly.

There are several different variants of stormwater pond design, the most common of which include the wet pond, the wet extended detention pond, and the micropool extended detention pond. In addition, multiple stormwater ponds can be placed in series or parallel to increase performance or meet site design constraints. Below are descriptions of each design variant:

- **Wet Pond** – Wet ponds are stormwater basins constructed with a permanent (dead storage) pool of water equal to the water quality volume. Stormwater runoff displaces the water already present in the pool. Temporary storage (live storage) can be provided above the permanent pool elevation for larger flows.
- **Wet Extended Detention (ED) Pond** – A wet extended detention pond is a wet pond where the water quality volume is split evenly between the permanent pool and extended detention (ED) storage provided above the permanent pool. During storm events, water is detained above the permanent pool and released over 24 hours. This design has similar pollutant removal to a traditional wet pond but consumes less space.
- **Micropool Extended Detention (ED) Pond** – The micropool extended detention pond is a variation of the wet ED pond where only a small “micropool” is maintained at the outlet to the pond. The outlet structure is sized to detain the water quality volume for 24 hours. The micropool prevents resuspension of previously settled sediments and prevents clogging of the low flow orifice.
- **Multiple Pond Systems** – Multiple Pond systems consist of constructed facilities that provide water quality and quantity volume storage in two or more cells. The additional cells can create longer pollutant removal pathways and improved downstream protection.

Figure C-3 shows several examples of stormwater pond variants. The Pollutant Removal Capabilities section provides plan view and profile schematics for the design of a wet pond, wet extended detention pond, micropool extended detention pond, and multiple pond system.

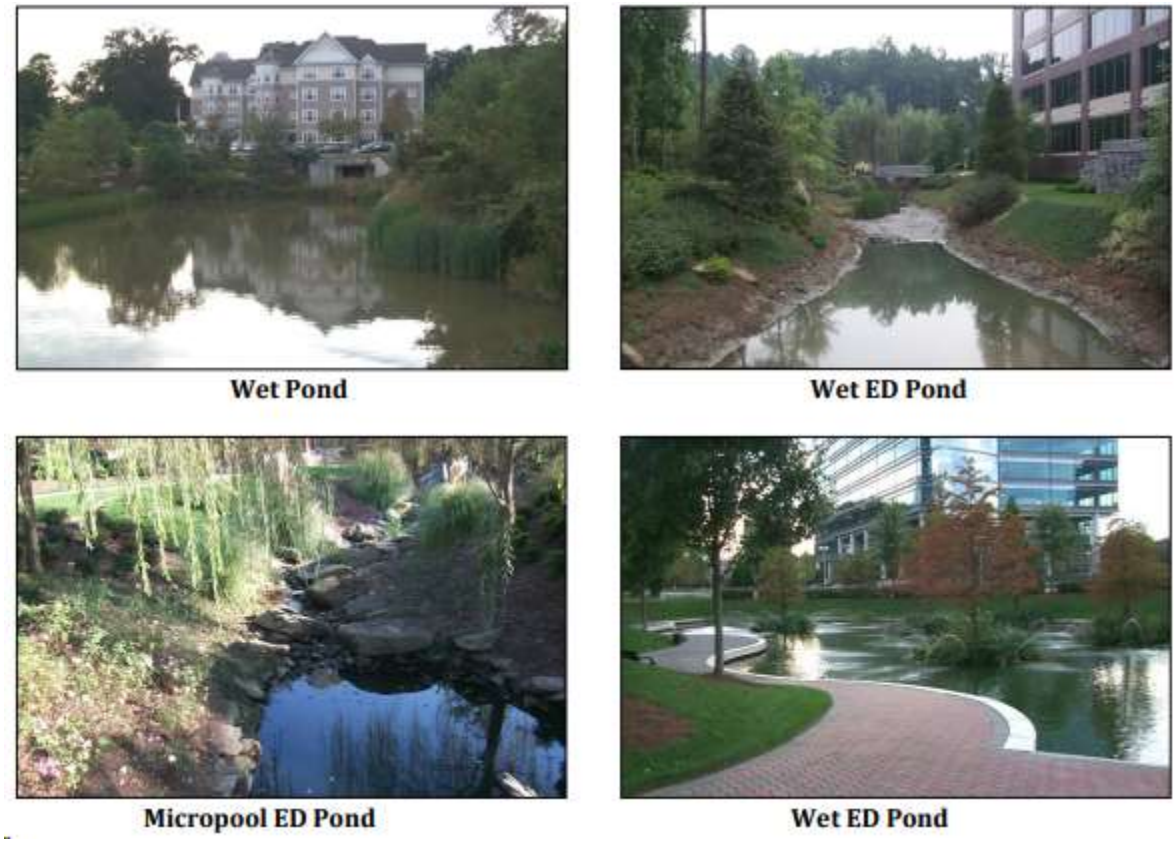


Figure C-3. Stormwater Pond Examples.

2. Stormwater Management Suitability

Stormwater ponds are designed to control both stormwater quantity and quality. Thus, a stormwater pond can be used to address water quality, channel protection, and overbank flood protection.

Water Quality

Pollutants are removed from stormwater runoff in a wetland through uptake by wetland vegetation and algae, vegetative filtering, and through gravitational settling in the slow-moving marsh flow. Other pollutant removal mechanisms are also at work in a stormwater wetland, including chemical and biological decomposition, and volatilization. The Pollutant Removal Capabilities section of this specification provides median pollutant removal efficiencies that can be used for planning and design purposes.

Downstream Flood Protection

A stormwater pond can also provide storage above the permanent pool to reduce the post-development peak flow of the design storm events to pre-development levels (detention).

3. Pollutant Removal Capabilities

All the stormwater pond design variants are presumed to be able to remove 80% of the total suspended solids load in typical urban post-development runoff when sized, designed, constructed, and maintained in accordance with the recommended specifications. Undersized or poorly designed ponds can reduce TSS removal performance. The following design pollutant removal rates are conservative average pollutant reduction percentages for design purposes derived

from sampling data, modeling, and professional judgment. In a situation where a removal rate is not deemed sufficient, additional controls may be put in place at the given site in a series or “treatment-train” approach.

- Total Suspended Solids – 80%
- Total Phosphorus – 50%
- Total Nitrogen – 30%
- Fecal Coliform – 70% (if no resident waterfowl population present)
- Heavy Metals – 50%

For additional information and data on pollutant removal capabilities for stormwater wetlands, see the National Pollutant Removal Performance Database (2nd Edition) available at www.cwp.org and the National Stormwater Best Management Practices (BMP) Database at www.bmpdatabase.org

4. Typical Details

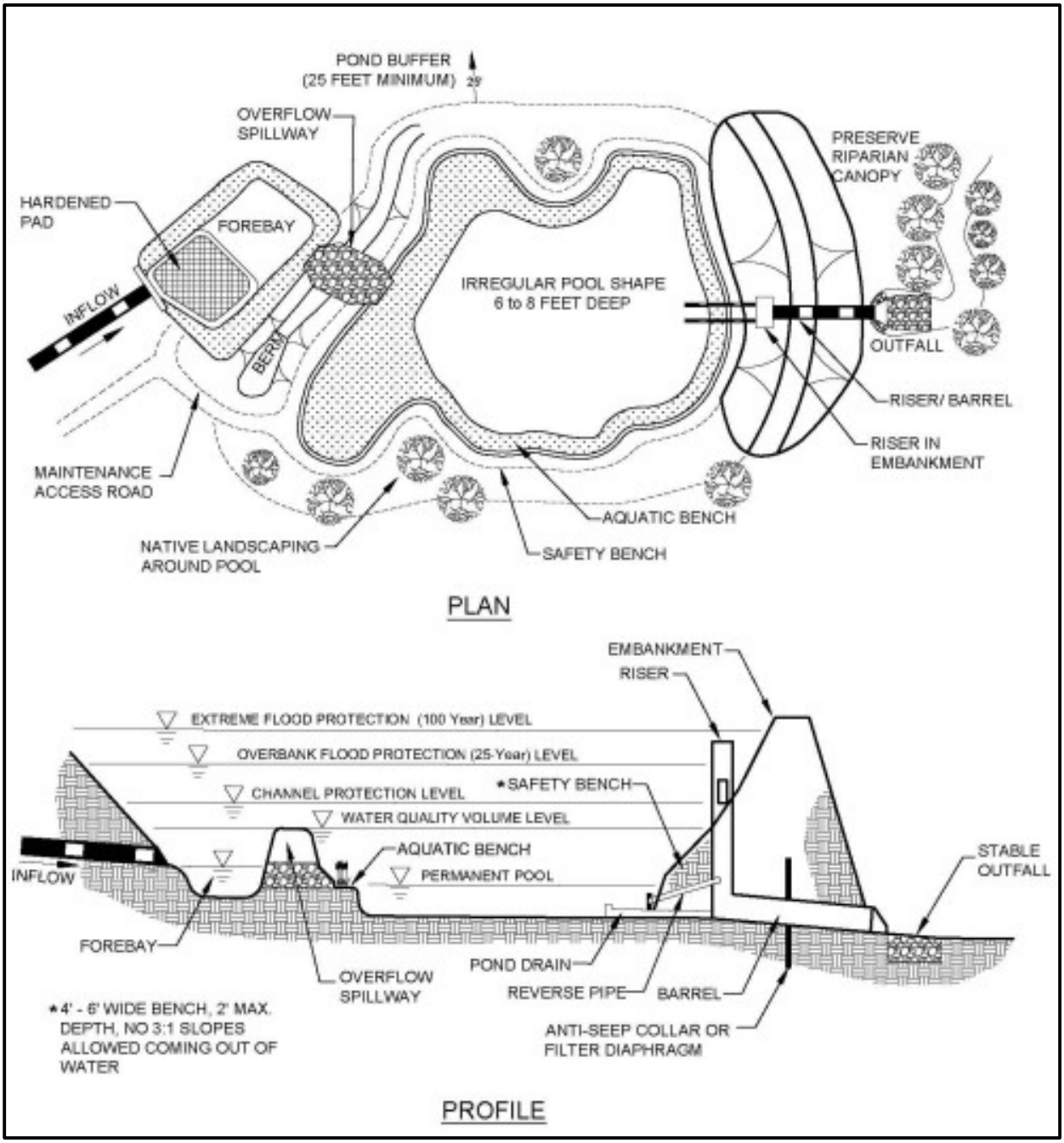


Figure C-4. Schematic of Wet Pond

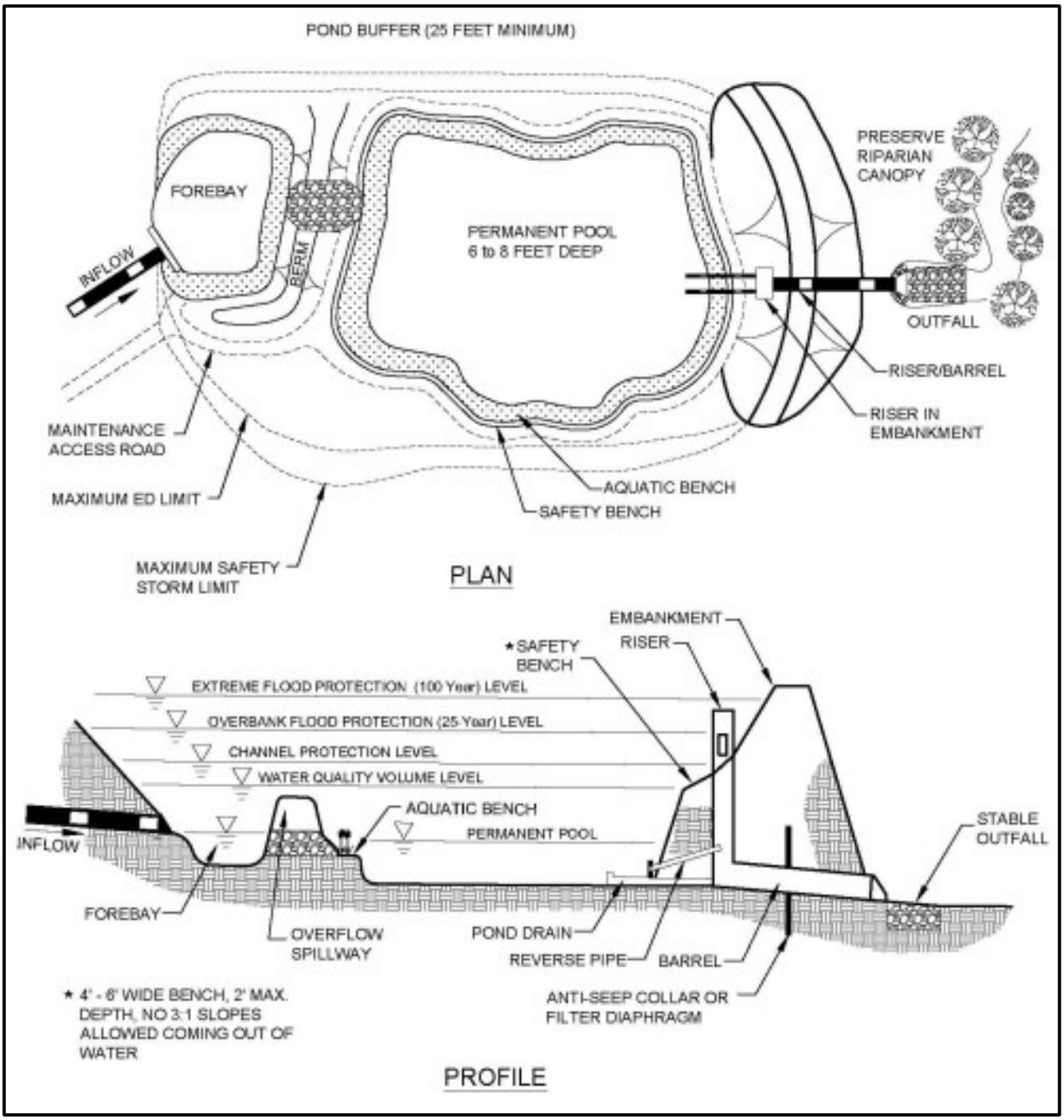


Figure C-5. Schematic of Wet Pond Extended Detention Pond.

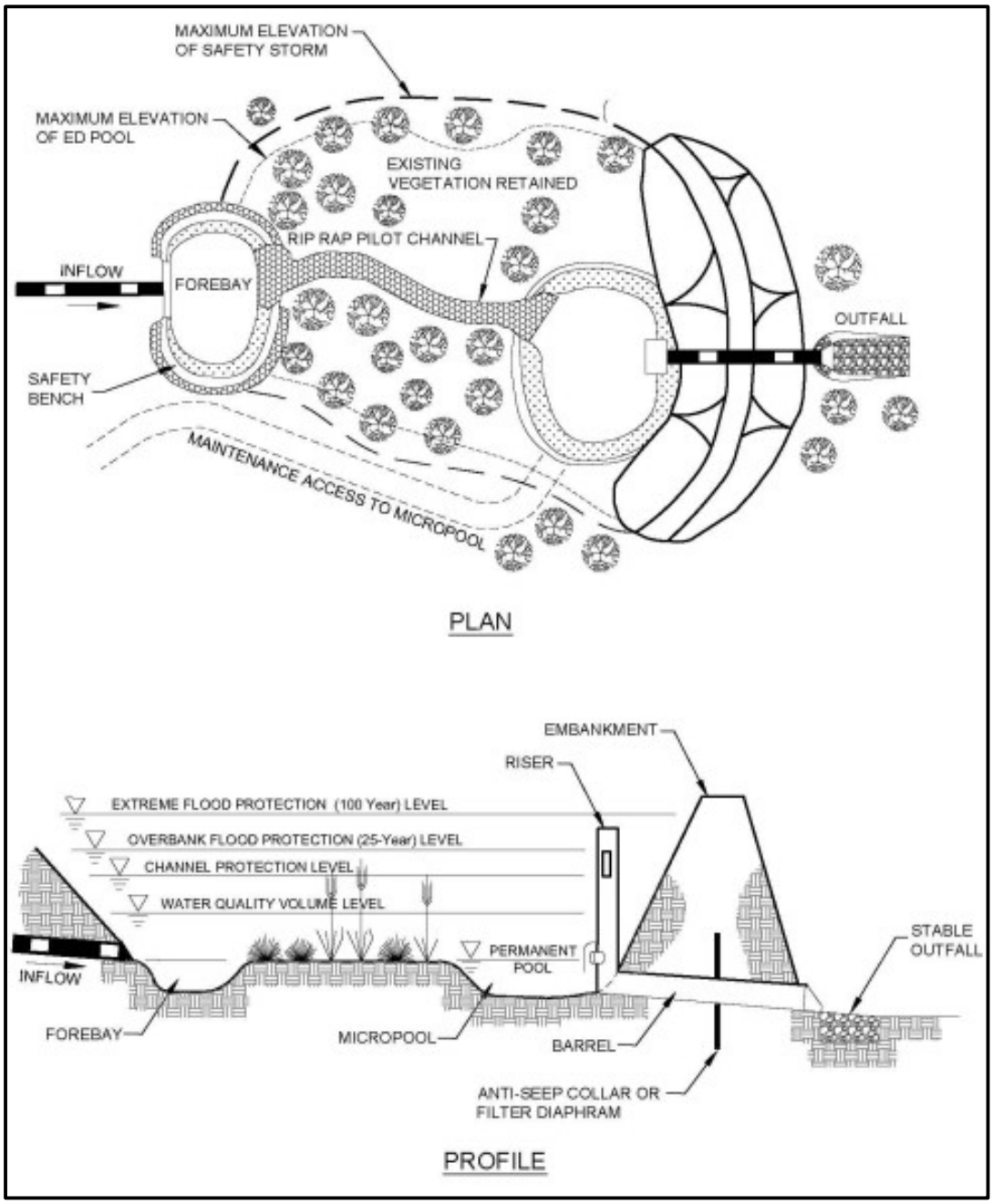


Figure C-6. Schematic of Micropool Extended Detention Pond.

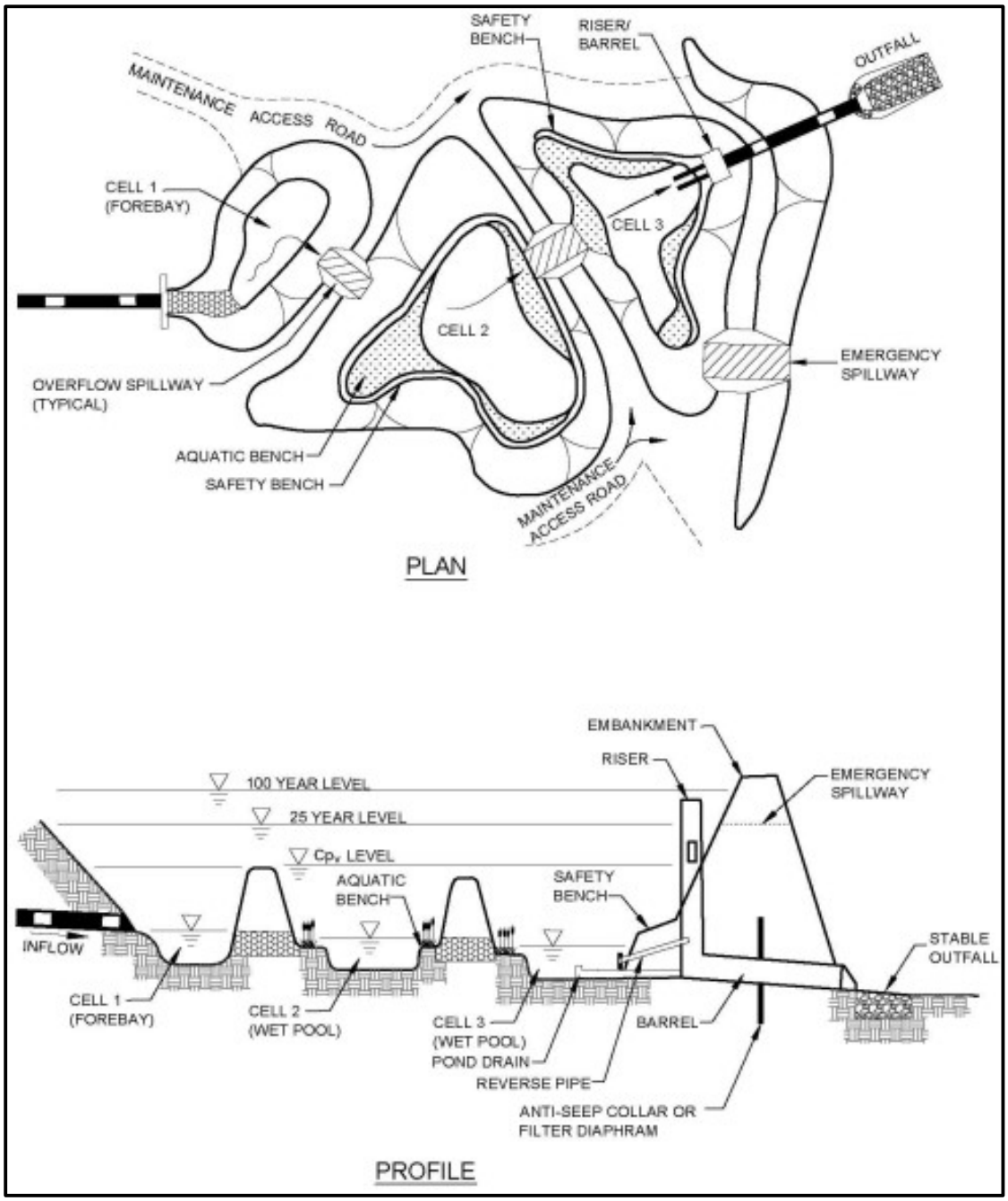


Figure C-7. Schematic of Multiple Pond System.

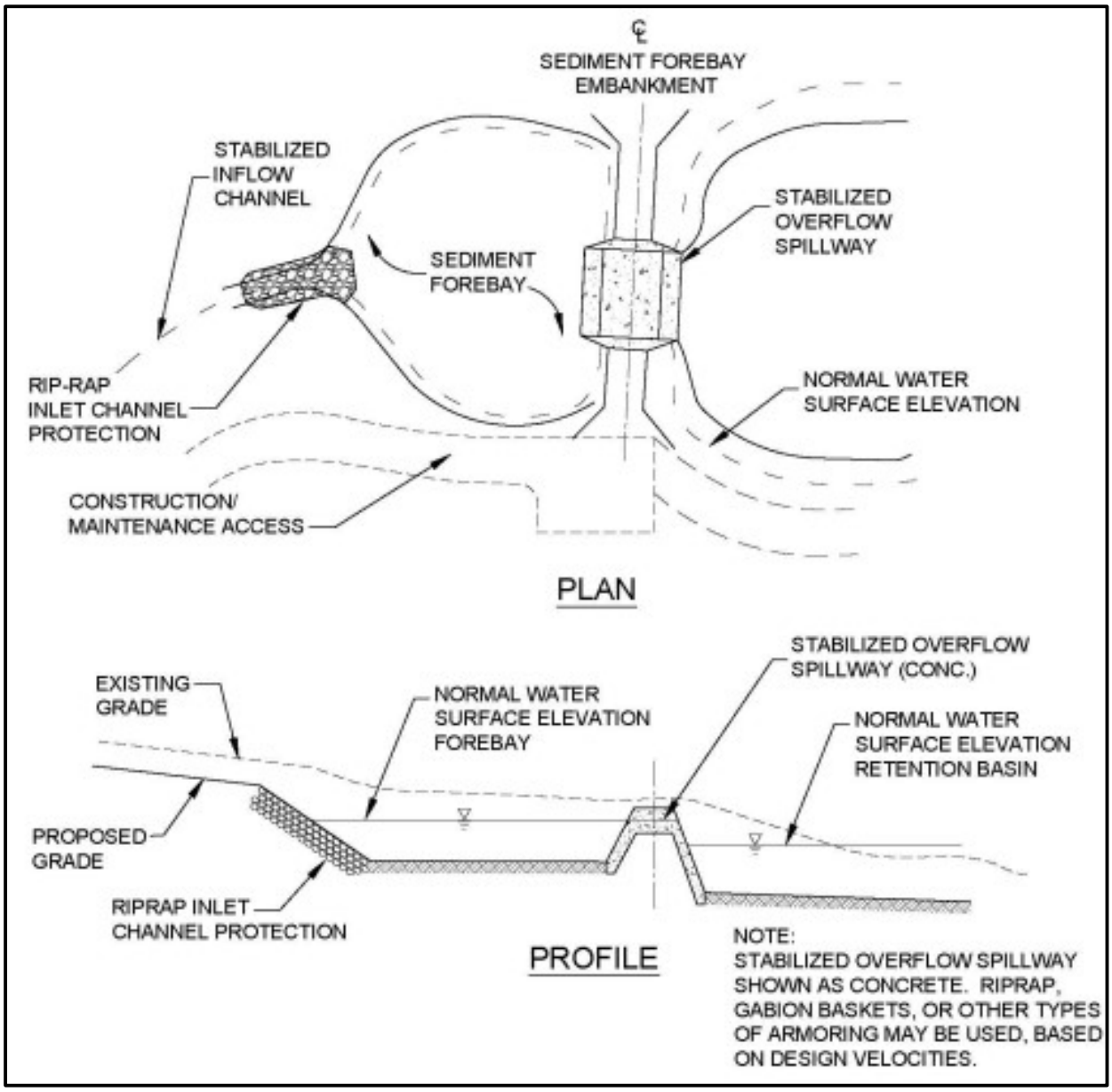


Figure C-8. Typical Sediment Forebay Plan and Section.

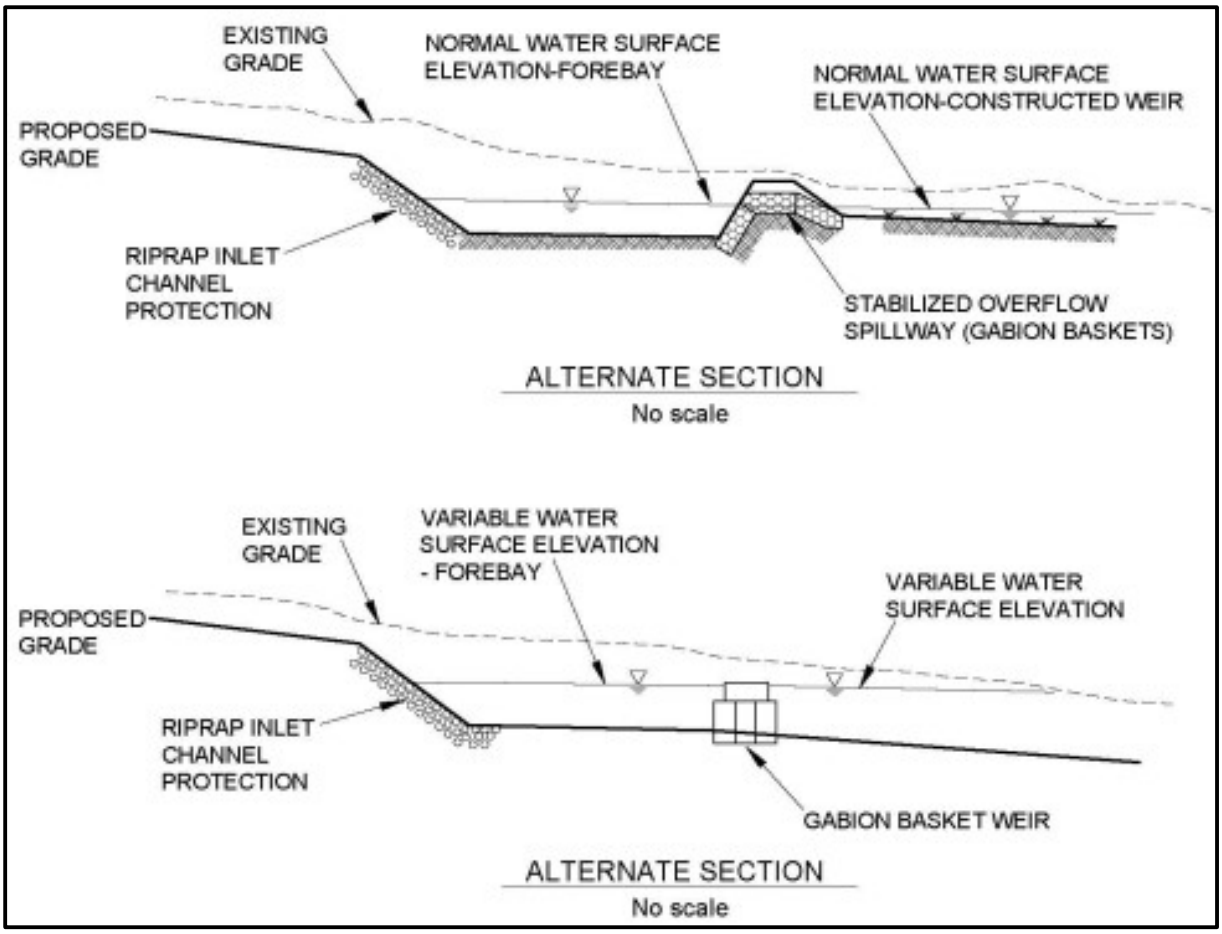


Figure C-9. Typical Sediment Forebay Alternate Sections.

5. Site Feasibility and Design Applications

Stormwater ponds are generally applicable to most types of new development and redevelopment and can be used in both residential and nonresidential areas. Ponds can also be used in retrofit situations. The following criteria should be evaluated to ensure the suitability of a stormwater pond for meeting stormwater management objectives on a site or development.

General Feasibility

- Suitable for Residential Subdivision Usage – YES
- Suitable for High Density/Ultra-Urban Areas – Land requirements may preclude use.
- Regional Stormwater Control – YES

Physical Feasibility - Physical Constraints at Project Site.

- **Drainage Area** – A minimum of 25 acres is needed for wet pond and wet ED pond to maintain a permanent pool, 10 acres minimum for micropool ED pond. A smaller drainage area may be acceptable with an adequate water balance and anti-clogging device.
- **Space Required** – Approximately 2 to 3% of the tributary drainage area.
- **Site Slope** – There should not be more than 15% slope across the pond site.
- **Minimum Head** – Elevation difference needed at a site from the inflow to the outflow: 6 to 8 ft.

- **Minimum Depth to Water Table** – If used on a site with an underlying water supply aquifer or when treating an area with potential for high pollutant loading, a separation distance of 2 ft is required between the bottom of the pond and the elevation of the seasonally high-water table.
- **Soils** – Underlying soils of hydrologic group "C" or "D" should be adequate to maintain a permanent pool. Most group "A" soils and some group "B" soils will require a pond liner. Evaluation of soils should be based upon an actual subsurface analysis and permeability tests.

6. Planning and Design Criteria

The following criteria are to be considered minimum standards for the design of a stormwater pond facility.

Location and Siting

- Stormwater ponds should have a minimum contributing drainage area of 25 acres or more for wet pond or wet ED pond to maintain a permanent pool. For a micropool ED pond, the minimum drainage area is 10 acres. A smaller drainage area can be considered when water availability can be confirmed (such as from a groundwater source or areas with a high-water table). In these cases, a water balance may be performed. Ensure that an appropriate anti-clogging device is provided for the pond outlet.
- A stormwater pond should be sited such that the topography allows for maximum runoff storage at minimum excavation or construction costs. Pond siting should also consider the location and use of other site features such as buffers and undisturbed natural areas and should attempt to aesthetically "fit" the facility into the landscape. Bedrock close to the surface may prevent excavation.
- Stormwater ponds should not be located on steep (>15%) or unstable slopes.
- Stormwater ponds cannot be located within a stream or any other navigable waters of the U.S., including wetlands, without obtaining a Section 404 permit under the Clean Water Act, and any other applicable State permit.
- Minimum setback requirements for stormwater pond facilities:
    - From a property line – 10 ft
    - From a private well – 100 ft; if well is downgradient from a land use area with potential for high pollutant loading, then the minimum setback is 250 ft
    - From a septic system tank/leach field – 50 ft
- All utilities should be located outside of the pond/basin site.

General Design

- A well-designed stormwater pond consists of:
    - Permanent pool of water,
    - Overlying zone in which runoff control volumes are stored, and
    - Shallow littoral zone (aquatic bench) along the edge of the permanent pool that acts as a biological filter.
- In addition, all stormwater pond designs need to include a sediment forebay at the inflow to the basin to allow heavier sediments to drop out of suspension before the runoff enters the permanent pool. Sediment forebay schematics can be found above (Figures C-8 and C-9).
- Additional pond design features include an emergency spillway, maintenance access, safety bench, pond buffer, and appropriate native landscaping.

Figures C-4 thru C-7 in this specification provide plan view and profile schematics for the design of a wet pond, wet ED pond, micropool ED pond and multiple pond system.

Physical Specifications / Geometry

In general, pond designs are unique for each site and application. However, there are number of geometric ratios and limiting depths for pond design that must be observed for adequate pollutant removal, ease of maintenance, and improved safety.

- Permanent pool volume is typically sized as follows:
  - Standard wet ponds: 100% of the water quality volume (1.0 WQ_v)
  - Wet ED ponds: 50% of the water quality volume (0.5 WQ_v)
  - Micropool ED ponds: Approximately 0.1 inch per impervious acre
- Proper geometric design is essential to prevent hydraulic short-circuiting (unequal distribution of inflow), which results in the failure of the pond to achieve adequate levels of pollutant removal. The minimum length-to-width ratio for the permanent pool shape is 1.5:1 and should ideally be greater than 3:1 to avoid short-circuiting. In addition, ponds should be wedge-shaped when possible so that flow enters the pond and gradually spreads out, improving the sedimentation process. Baffles, pond shaping, or islands can be added within the permanent pool to increase the flow path.
- Maximum depth of the permanent pool should generally not exceed 8 ft to avoid stratification and anoxic conditions. Minimum depth for the pond bottom should be 3 to 4 ft. Deeper depths near the outlet will yield cooler bottom water discharges that may mitigate downstream thermal effects.
- Side slopes to the pond should not usually exceed 3:1 (h:v) without safety precautions or if mowing is anticipated and should terminate on a safety bench (see Figure C-10). The safety bench requirement may be waived if slopes are 4:1 or gentler.

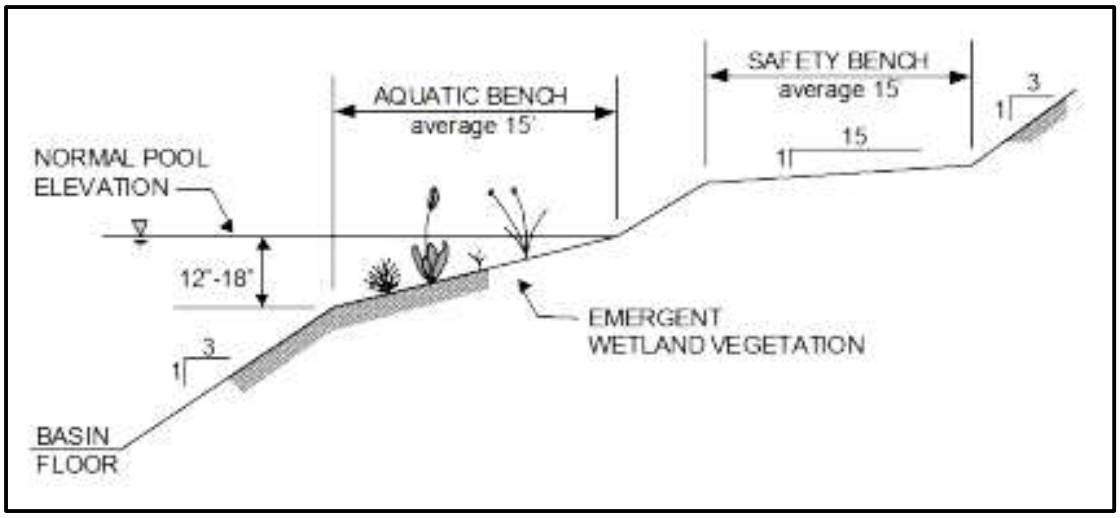


Figure C-10. Typical Stormwater Pond Geometry Criteria.

- The perimeter of all deep pool areas (4 ft or greater in depth) should be surrounded by two benches: safety and aquatic. For larger ponds, a safety bench extends approximately 15 ft outward from the normal water edge to the toe of the pond side slope. The maximum slope of the safety bench should be 6%. An aquatic bench extends inward from the normal pool edge (15 ft on average) and has a maximum depth of 18 inches below the normal pool water surface elevation (see Figure C-10).
- The contours and shape of the permanent pool should be irregular to provide a more natural landscaping effect.

Pretreatment / Inlets

- Each pond should have a sediment forebay or equivalent upstream pretreatment. A sediment forebay is designed to remove incoming sediment from the stormwater flow prior to dispersal in a larger permanent pool.

The forebay should consist of a separate cell, formed by an acceptable barrier. A forebay is to be provided at each inlet unless the inlet provides less than 10% of the total design storm inflow to the pond. In some design configurations, the pretreatment volume may be located within the permanent pool.

- The forebay is sized to contain 0.1 inches per impervious acre of contributing drainage and should be 4 to 6 ft deep. The pretreatment storage volume is part of the total WQ_v requirement and may be subtracted from WQ_v for permanent pool sizing.
- A fixed vertical sediment depth marker shall be installed in the forebay to measure sediment deposition over time. The bottom of the forebay may be hardened (e.g., using concrete, paver blocks, etc.) to make sediment removal easier.
- Inflow channels are to be stabilized with flared riprap aprons, or the equivalent. Inlet pipes to the pond can be partially submerged. Exit velocities from the forebay must be nonerosive.

Outlet Structures

- Flow control from a stormwater pond is typically accomplished with the use of a concrete or corrugated metal riser and barrel. The riser is a vertical pipe or inlet structure that is attached to the base of the pond with a watertight connection. The outlet barrel is a horizontal pipe attached to the riser that conveys flow under the embankment (see Figure C-11). The riser should be located within the embankment for maintenance access, safety, and aesthetics.

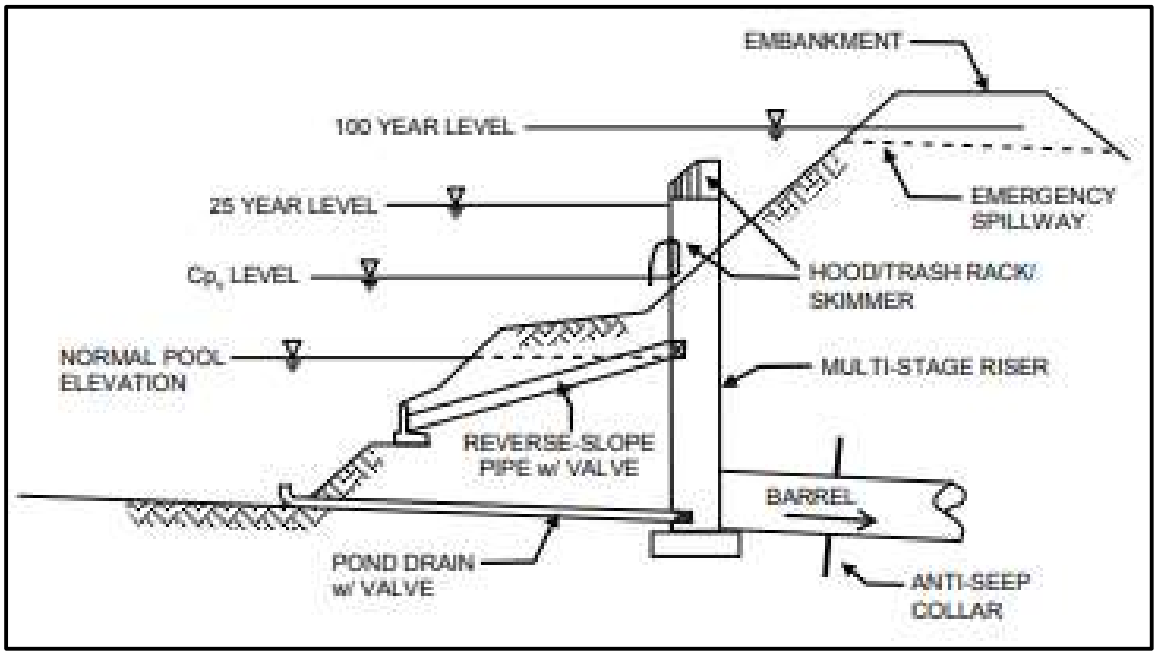


Figure C-11. Typical Pond Outlet Structure.

- Several outlets at varying depths in the riser provide internal flow control for routing of the water quality, channel protection, and overbank flood protection runoff volumes. The number of orifices can vary and is usually a function of the pond design.
- For example, a wet pond riser configuration is typically comprised of a channel protection outlet (usually an orifice) and overbank flood protection outlet (often a slot or weir). The channel protection orifice is sized to release the channel protection storage volume over a 24-hour period (12-hour extended detention may be warranted in some cold-water streams). Since the water quality volume is fully contained in the permanent pool, no orifice sizing is necessary for this volume. As runoff from a water quality event enters the wet pond, it simply displaces that same volume through the channel protection orifice. Thus, an off-line wet pond providing only

water quality treatment can use a simple overflow weir as the outlet structure. In the case of a wet ED pond or micropool ED pond, there is generally a need for an additional outlet (usually an orifice) that is sized to pass the extended detention water quality volume that is surcharged on top of the permanent pool. Flow will first pass through this orifice, which is sized to release the water quality ED volume in 24 hours. The preferred design is a reverse slope pipe attached to the riser, with its inlet submerged 1 ft below the elevation of the permanent pool to prevent floatables from clogging the pipe and to avoid discharging warmer water at the surface of the pond. The next outlet is sized for the release of the channel protection storage volume. The outlet (often an orifice) invert is located at the maximum elevation associated with the extended detention water quality volume and is sized to release the channel protection storage volume over a 24-hour period.

- Alternative hydraulic control methods to an orifice can be used and include the use of a broad-crested rectangular, V-notch, proportional weir, or an outlet pipe protected by a hood that extends at least 12 inches below the normal pool.
- The water quality outlet (if design is for a wet ED or micropool ED pond) and channel protection outlet should be fitted with adjustable gate valves or other mechanism that can be used to adjust detention time.
- Higher flows (overbank and extreme flood protection) flows pass through openings or slots protected by trash racks further up on the riser.
- After entering the riser, flow is conveyed through the barrel and is discharged downstream. Antiseep collars should be installed on the outlet barrel to reduce the potential for pipe failure.
- Riprap, plunge pools or pads, or other energy dissipaters are to be placed at the outlet of the barrel to prevent scouring and erosion. If a pond daylight to a channel with dry weather flow, care should be taken to minimize tree clearing along the downstream channel, and to reestablish a forested riparian zone in the shortest possible distance.
- Each pond must have a bottom drainpipe with an adjustable valve that can completely or partially drain the pond within 24 hours.
- The pond drain should be sized one pipe size greater than the calculated design diameter. The drain valve is typically a handwheel activated knife or gate valve. Valve controls shall be located inside of the riser at a point where they (a) will not normally be inundated and (b) can be operated in a safe manner. See the design procedures in Chapter 7 for additional information and specifications on pond routing and outlet works.

Emergency Spillway

- An emergency spillway is to be included in the stormwater pond design to safely pass the extreme flood flow. The spillway prevents pond water levels from overtopping the embankment and causing structural damage. The emergency spillway must be located so that downstream structures will not be impacted by spillway discharges.
- A minimum of 1 ft of freeboard must be provided, measured from the top of the water surface elevation for the extreme flood to the lowest point of the dam embankment, not counting the emergency spillway.

Maintenance Access

- A maintenance right of way or easement must be provided to a pond from a public or private road. Maintenance access should be at least 12 ft wide, have a maximum slope of no more than 15%, and be appropriately stabilized to withstand maintenance equipment and vehicles.
- The maintenance access must extend to the forebay, safety bench, riser, and outlet and, to the extent feasible, be designed to allow vehicles to turn around.
- Access to the riser is to be provided by lockable manhole covers, and manhole steps within easy reach of valves and other controls.

Safety Features

- All embankments and spillways for ponds with a volume over 50 acre-ft must be obtain an Arkansas Natural Resources Commission permit, in accordance with ANRC Title VII, Rules Governing Design and Operation of Dams.
- The safety bench should be landscaped to deter access to the pool.
- The principal spillway opening should not permit access by small children, and end walls above pipe outfalls greater than 48 inches in diameter should be fenced to prevent access. Warning signs should be posted near the pond to prohibit swimming and fishing in the facility.

Landscaping

- Aquatic vegetation can play an important role in pollutant removal in a stormwater pond. In addition, vegetation can enhance the appearance of the pond, stabilize side slopes, serve as wildlife habitat, and can temporarily conceal unsightly trash and debris. Therefore, wetland plants should be encouraged in a pond design, along the aquatic bench (fringe wetlands), the safety bench and side slopes (ED ponds), and within shallow areas of the pool itself. The best elevations for establishing wetland plants, either through transplantation or volunteer colonization, are within 6 inches (plus or minus) of the normal pool elevation.
- Woody vegetation may not be planted on the embankment or allowed to grow within 15 ft of the toe of the embankment and 25 ft from the principal spillway structure.
- A pond buffer should be provided that extends 25 ft outward from the maximum water surface elevation of the pond. The pond buffer should be contiguous with other buffer areas that are required by existing regulations (e.g., stream buffers) or that are part of the overall stormwater management concept plan. No structures should be located within the buffer, and an additional setback to permanent structures may be provided.
- Existing trees should be preserved in the buffer area during construction. It is desirable to locate forest conservation areas adjacent to ponds. To discourage resident geese populations, the buffer can be planted with trees, shrubs, and native ground covers.
- The soils of a pond buffer are often severely compacted during the construction process to ensure stability. The density of these compacted soils is so great that it effectively prevents root penetration and therefore may lead to premature mortality or loss of vigor. Consequently, it is advisable to excavate large and deep holes around the proposed planting sites and backfill these with uncompacted topsoil.
- A fountain or solar-powered aerator may be used for oxygenation of water in the permanent pool.
- Compatible multi-objective use of stormwater pond locations is strongly encouraged.

Additional Site-Specific Design Criteria and Issues

- Physiographic Factors - Local terrain design constraints
    - Low Relief – Maximum normal pool depth is limited; providing pond drain can be problematic.
    - High Relief – Embankment heights restricted.
- Soils
    - Hydrologic group “A” soils generally require pond liner; group “B” soils may require infiltration testing.

7. Design Procedures

Step 1. Compute runoff control volumes from the Stormwater Sizing Criteria.

- Calculate the Water Quality Volume (WQ_v), and design storm events.
- Details on the Stormwater Sizing Criteria are found in Chapter 2 of the Stormwater Drainage Manual.

Step 2. Determine if the development site and conditions are appropriate for the use of a stormwater pond.

- Consider the Application and Site Feasibility Criteria in the Planning and Design Criteria section of this specification.

Step 3. Confirm local design criteria and applicability.

- Consider any special site-specific design conditions/criteria from the Planning and Design Criteria section of this specification.
- Check with City Engineer to determine if there are any additional restrictions and/or surface water or watershed requirements that may apply.

Step 4. Determine pretreatment volume.

A sediment forebay is provided at each inlet unless the inlet provides less than 10% of the total design storm inflow to the pond. The forebay should be sized to contain 0.1 inches per impervious acre of contributing drainage and should be 4 to 6 ft deep. The forebay storage volume counts toward the total WQ_v requirement and may be subtracted from the WQ_v for subsequent calculations.

Step 5. Determine permanent pool volume (and water quality ED volume)

- **Wet Pond:** Size permanent pool volume to 1.0 WQ_v.
- **Wet ED Pond:** Size permanent pool volume to 0.5 WQ_v. Size extended detention volume to 0.5 WQ_v.
- **Micropool ED Pond:** Size permanent pool volume to 25 to 30% of WQ_v. Size extended detention volume to remainder of WQ_v.

Step 6. Determine pond location and preliminary geometry. Conduct pond grading and determine storage available for permanent pool (and water quality extended detention if wet ED pond or micropool ED pond).

This step involves initially grading the pond (establishing contours) and determining the elevation-storage relationship for the pond.

- Include safety and aquatic benches.
- Set W_{Qv} permanent pool elevation (and W_{Qv}-ED elevation for wet ED and micropool ED pond) based on volumes calculated earlier. See Section 6 of this specification for more details.

Step 8. Calculate design storms release rate and water surface elevation.

Set up a stage-storage-discharge relationship for the control structure for the design storms.

Step 9. Design embankment(s) and spillway(s).

Size emergency spillway, calculate 100-year water surface elevation, set top of embankment elevation, and analyze safe passage of the 100-year event.

Step 10. Investigate potential pond hazard classification.

The design and construction of stormwater management ponds are required to follow the latest version of the State of Arkansas dam safety rules (see Appendix E).

Step 11. Design inlets, sediment forebay(s), outlet structures, maintenance access, and safety features.

See Section 6 of this specification for more details.

Step 12. Prepare Vegetation and Landscaping Plan.

A landscaping plan for a stormwater pond and its buffer should be prepared to indicate how aquatic and terrestrial areas will be stabilized and established with vegetation. See the Planning and Design Criteria section of this specification for more details.

8. Inspection and Maintenance Requirements

Table C-3. Typical Maintenance Activities for Ponds.

Activity	Schedule
• Clean and remove debris from inlet and outlet structures. • Mow side slopes.	Monthly
• If wetland components are included, inspect for invasive vegetation.	Semiannual Inspection
• Inspect for damage, paying particular attention to the control structure. • Check for signs of eutrophic conditions. • Note signs of hydrocarbon build-up and remove appropriately. • Monitor for sediment accumulation in the facility and forebay. • Examine to ensure that inlet and outlet devices are free of debris and operational. • Check all control gates, valves, or other mechanical devices.	Annual Inspection
• Repair undercut or eroded areas.	As Needed
• Perform wetland plant management and harvesting.	Annually (if needed)
• Remove sediment from the forebay.	5 to 7 years or after 50% of the total forebay capacity has been lost
• Monitor sediment accumulations and remove sediment when the pool volume has become reduced significantly, or the pond becomes eutrophic.	10 to 20 years or after 25% of the permanent pool volume has been lost

9. References

Atlanta Regional Commission, 2001. *Georgia Stormwater Management Manual, Volume 2: Technical Handbook.* Atlanta, GA. http://www.georgiastormwater.com/GSMMVol2.pdf

Center for Watershed Protection (CWP), 1996. Design of Stormwater Filtering Systems. Prepared for Chesapeake Research Consortium.

City of Fayetteville, 2014. *Drainage Criteria Manual.* Fayetteville, AR. Drainage-Criteria-Manual-Appendix-F-PDF (fayetteville-ar.gov)

Watershed Management Institute (WMI), 1997. Operation, Maintenance, and Management of Stormwater Management Systems. Prepared for US EPA, Office of Water.

WQ-03 Stormwater Wetlands

1. Description

Stormwater wetlands (also referred to as constructed wetlands) are constructed shallow marsh systems that are designed to both treat urban stormwater and control runoff volumes. As stormwater runoff flows through the wetland facility, pollutant removal is achieved through settling and uptake by marsh vegetation.

Wetlands are among the most effective stormwater practices in terms of pollutant removal and offer aesthetic value and wildlife habitat. Constructed stormwater wetlands differ from natural wetland systems in that they are engineered facilities designed specifically for the purpose of treating stormwater runoff and typically have less biodiversity than natural wetlands both in terms of plant and animal life. However, as with natural wetlands, stormwater wetlands require a continuous base flow or a high-water table to support aquatic vegetation.



Figure C-12. Stormwater Wetland Examples.

There are several design variations of the stormwater wetland, each design differing in the relative amounts of shallow and deep water, and dry storage above the wetland. These include the shallow wetland, the extended detention shallow wetland, pond/wetland system and pocket wetland. Below are descriptions of each design variant:

- **Shallow Wetland** – In the shallow wetland design, most of the water quality volume is in the relatively shallow high marsh or low marsh depths. The only deep portions of the shallow wetland design are the forebay at the inlet to the wetland, and the micropool at the outlet. One disadvantage of this design is that, since the pool is very shallow, a relatively large amount of land is typically needed to store the water quality volume.
- **Extended Detention (ED) Shallow Wetland** – The extended detention (ED) shallow wetland design is the same as the shallow wetland; however, part of the water quality volume is provided as extended detention above the

surface of the marsh and released over a period of 24 hours. This design can treat a greater volume of stormwater in a smaller space than the shallow wetland design. In the extended detention wetland option, plants that can tolerate both wet and dry periods need to be specified in the ED zone.

- **Pond/Wetland Systems** – The pond/wetland system has two separate cells: a wet pond and a shallow marsh. The wet pond traps sediments and reduces runoff velocities prior to entry into the wetland, where stormwater flows receive additional treatment. Less land is required for a pond/wetland system than for the shallow wetland or the ED shallow wetland systems.
- **Pocket Wetland** – A pocket wetland is intended for smaller drainage areas of 5 to 10 acres and typically requires excavation down to the water table for a reliable water source to support the wetland system.

2. Stormwater Management Suitability

Like stormwater ponds, stormwater wetlands are designed to control both stormwater quantity and quality. Thus, a stormwater wetland can be used to address water quality, channel protection, and overbank flood protection.

Water Quality

Pollutants are removed from stormwater runoff in a wetland through uptake by wetland vegetation and algae, vegetative filtering, and through gravitational settling in the slow-moving marsh flow. Other pollutant removal mechanisms are also at work in a stormwater wetland, including chemical and biological decomposition, and volatilization. The Pollutant Removal Capabilities section of this specification provides median pollutant removal efficiencies that can be used for planning and design purposes.

Downstream Flood Protection

A stormwater wetland can also provide storage above the permanent pool to reduce the post-development peak flow of the design storms to pre-development levels (detention).

3. Pollutant Removal Capabilities

All the stormwater wetland design variants are presumed to be able to remove 80% of the total suspended solids load in typical urban post-development runoff when sized, designed, constructed, and maintained in accordance with the recommended specifications. Undersized or poorly designed wetland facilities can reduce TSS removal performance. The following design pollutant removal rates are conservative average pollutant reduction percentages for design purposes derived from sampling data, modeling, and professional judgment. In a situation where a removal rate is not deemed sufficient, additional controls may be put in place at the given site in a series or “treatment-train” approach.

- Total Suspended Solids – 80%
- Total Phosphorus – 40%
- Total Nitrogen – 30%
- Fecal Coliform – 70% (if no resident waterfowl population present)
- Heavy Metals – 50%

For additional information and data on pollutant removal capabilities for stormwater wetlands, see the National Pollutant Removal Performance Database (2nd Edition) available at www.cwp.org and the National Stormwater Best Management Practices (BMP) Database at www.bmpdatabase.org

4. Typical Details

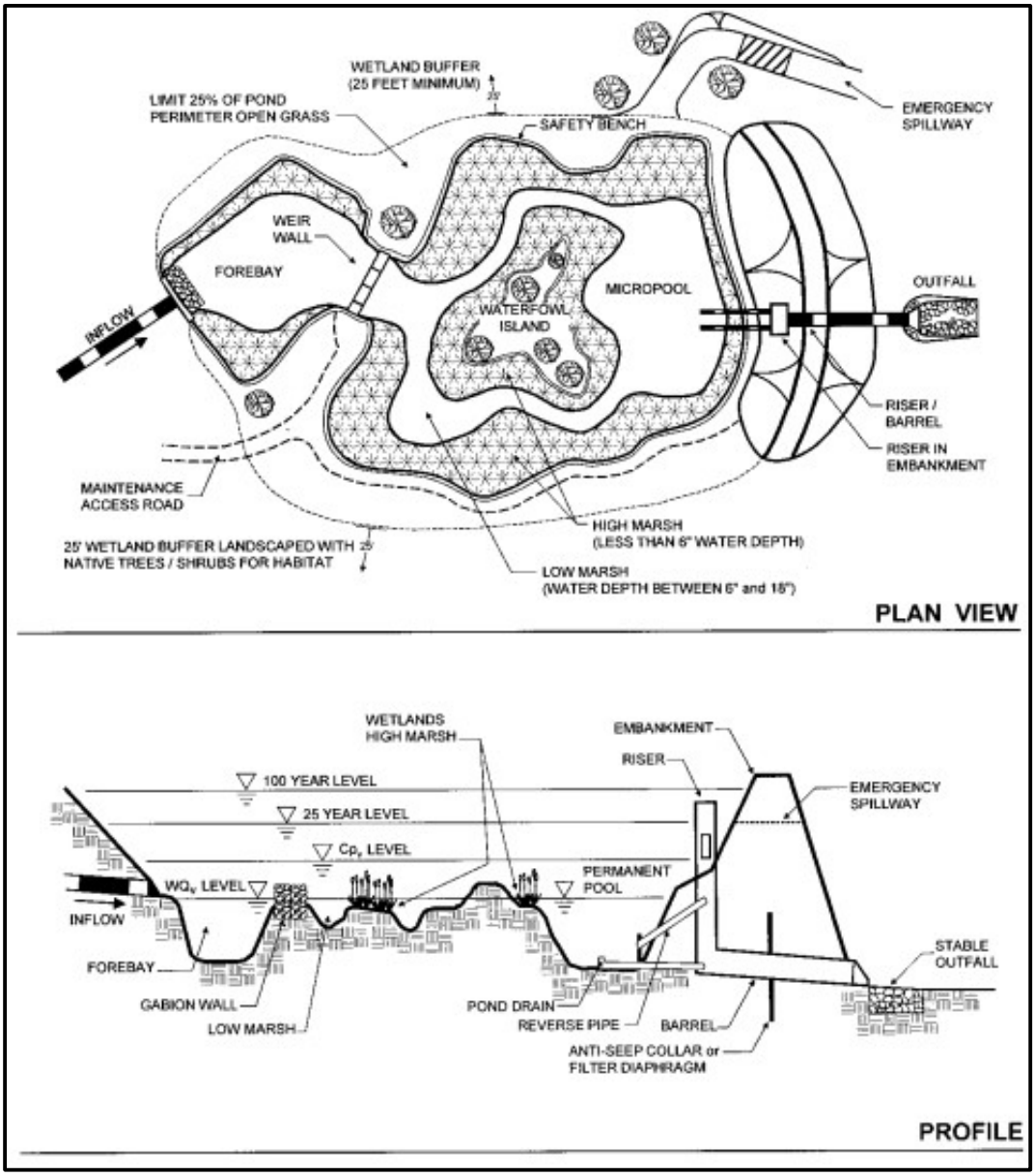


Figure C-13. Schematic of Shallow Wetland.

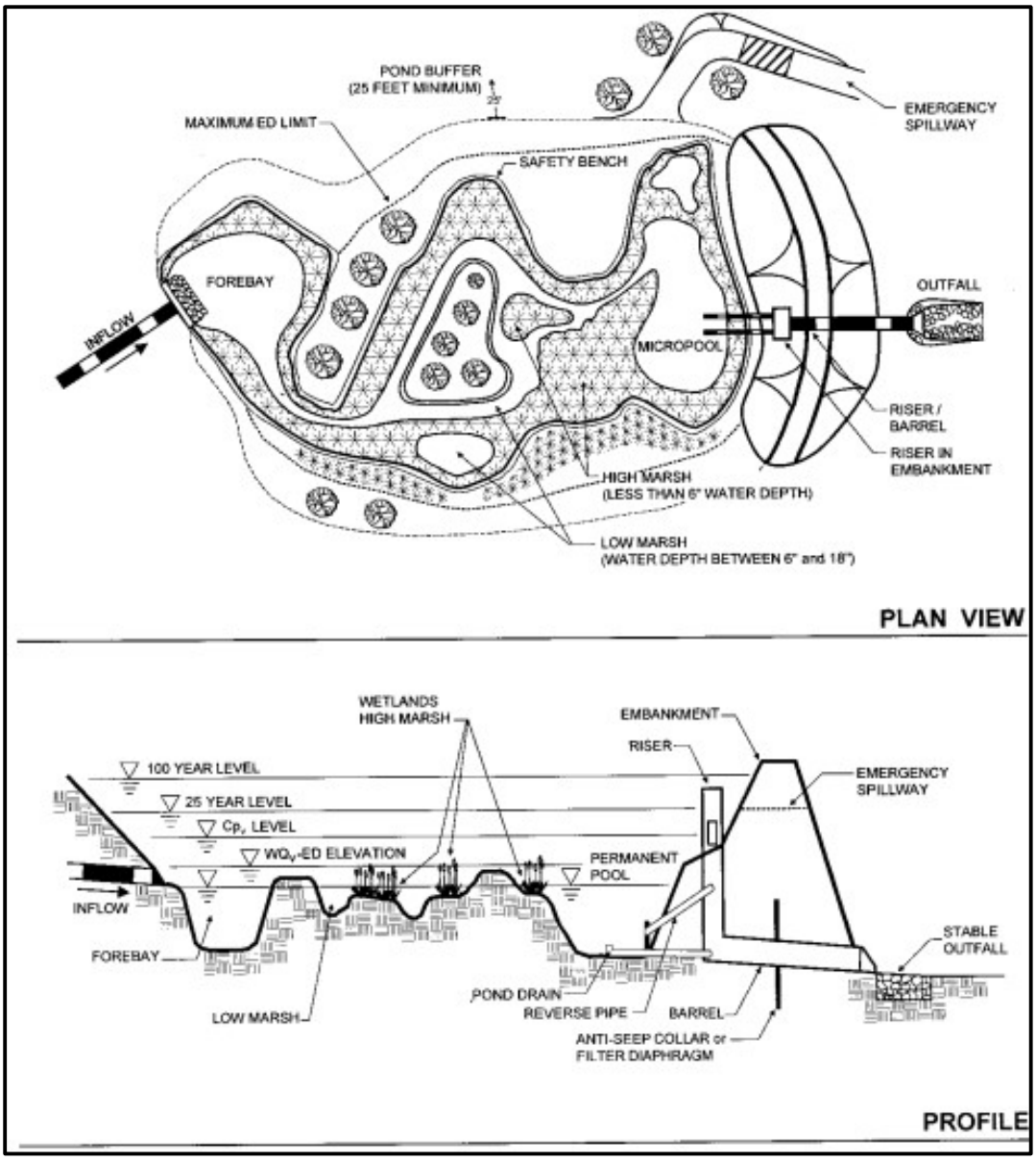


Figure C-14. Schematic of Extended Detention Shallow Wetland.

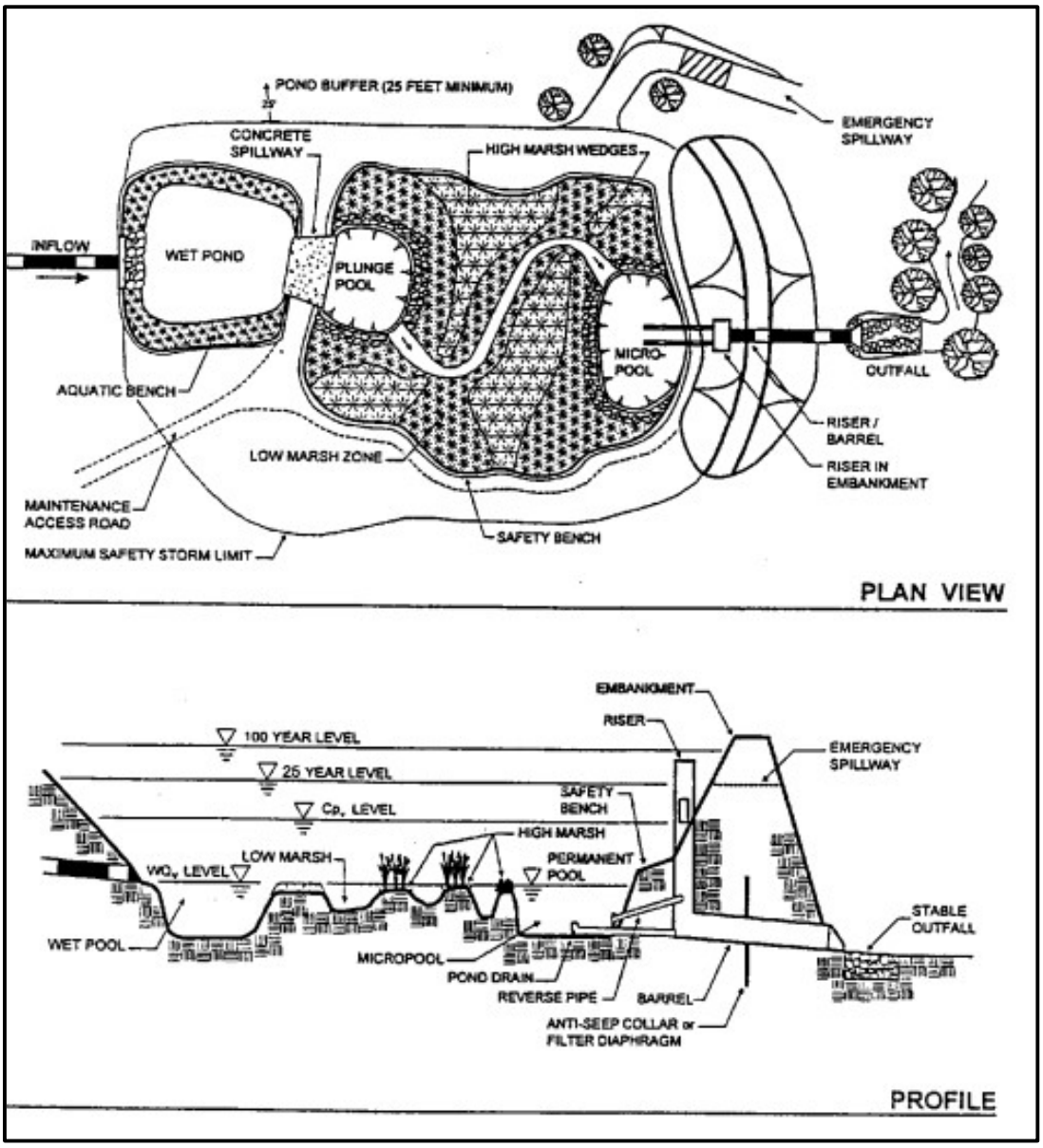


Figure C-15. Schematic of Pond/Wetland System.

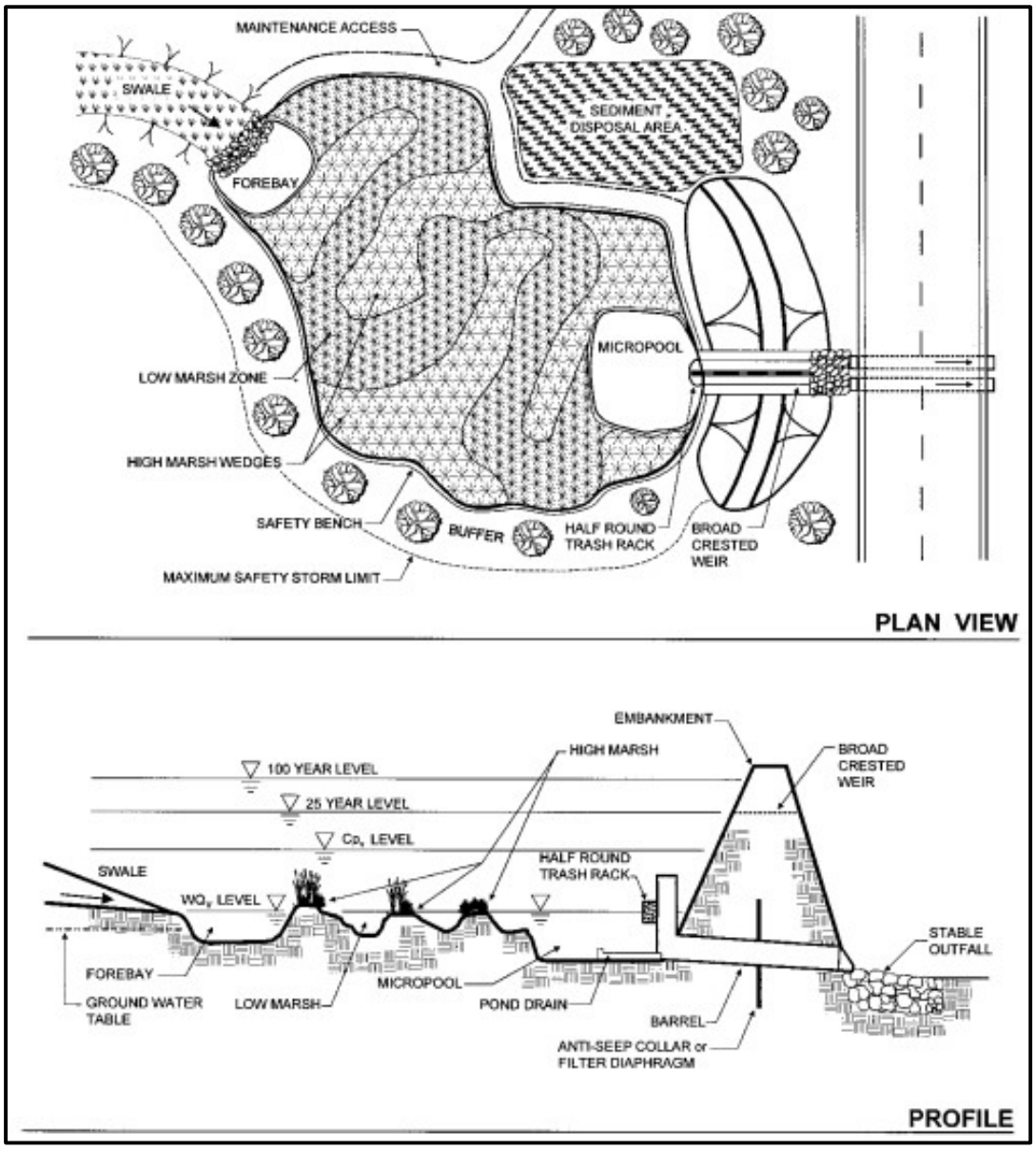


Figure C-16. Schematic of Pocket Wetland.

5. Site Feasibility and Design Applications

Stormwater wetlands are generally applicable to most types of new development and redevelopment and can be utilized in both residential and nonresidential areas. However, due to the large land requirements, wetlands may not be practical in higher density areas. The following criteria should be evaluated to ensure the suitability of a stormwater wetland for meeting stormwater management objectives on a site or development.

General Feasibility

- Suitable for Residential Subdivision Usage – YES
- Suitable for High Density/Ultra Urban Areas – Land requirements may preclude use
- Regional Stormwater Control – YES

Physical Feasibility - Physical Constraints at Project Site

- **Drainage Area** – A minimum of 25 acres and a positive water balance is needed to maintain wetland conditions; 5 acres for pocket wetland.
- **Space Required** – Approximately 3 to 5% of the tributary drainage area.
- **Site Slope** – There should be no more than 8% slope across the wetland site.
- **Minimum Head** – Elevation difference needed at a site from the inflow to the outflow: 3 to 5 ft; 2 to 3 ft for pocket wetland.
- **Minimum Depth to Water Table** – If used on a site with an underlying water supply aquifer or when treating an area with potential for high pollutant loading, a separation distance of 2 ft is recommended between the bottom of the wetland and the elevation of the seasonally high-water table; pocket wetland is typically below water table.
- **Soils** – Permeable soils are not well suited for a constructed stormwater wetland without a high-water table. Underlying soils of hydrologic group "C" or "D" should be adequate to maintain wetland conditions. Most group "A" soils and some group "B" soils will require a liner. Evaluation of soils should be based upon an actual subsurface analysis and permeability tests.

6. Planning and Design Criteria

The following criteria are to be considered minimum standards for the design of a stormwater wetland facility.

Location and Siting

- Stormwater wetlands should normally have a minimum contributing drainage area of 25 acres or more. For a pocket wetland, the minimum drainage area is 5 acres.
- A continuous base flow or high-water table is required to support wetland vegetation. A water balance must be performed to demonstrate that a stormwater wetland can withstand a 30-day drought at summer evaporation rates without completely drawing down.
- Wetland siting should also consider the location and use of other site features such as natural depressions, buffers, and undisturbed natural areas, and should attempt to aesthetically "fit" the facility into the landscape. Bedrock close to the surface may prevent excavation.
- Stormwater wetlands cannot be located within navigable waters of the U.S., including wetlands, without obtaining a Section 404 permit under the Clean Water Act, and any other applicable State permit. In some isolated cases, a wetlands permit may be granted to convert an existing degraded wetland in the context of local watershed restoration efforts.
- If a wetland facility is not used for overbank flood protection, it should be designed as an off-line system to bypass higher flows rather than passing them through the wetland system.
- Minimum setback requirements for stormwater wetland facilities (when not specified by local ordinance or criteria):
  1. From a property line – 10 ft
  2. From a private well – 100 ft; if well is downgradient from an area with potential for high pollutant loading land use, then the minimum setback is 250 ft
  3. From a septic system tank/leach field – 50 ft
- All utilities should be located outside of the wetland site.

General Design

- A well-designed stormwater wetland consists of:
  1. Shallow marsh areas of varying depths with wetland vegetation.
  2. Permanent micropool.
  3. Overlying zone in which runoff control volumes are stored. Pond/wetland systems also include a stormwater pond facility (see WQ-02, Stormwater Ponds, in Appendix C for pond design information).

- In addition, all wetland designs must include a sediment forebay at the inflow to the facility to allow heavier sediments to drop out of suspension before the runoff enters the wetland marsh.
- Additional pond design features include an emergency spillway, maintenance access, safety bench, wetland buffer, and appropriate wetland vegetation and native landscaping.

Figures C-13 through C-16 of this specification provide plan view and profile schematics for the design of a shallow wetland, ED shallow wetland, pond/wetland system, and pocket wetland.

Physical Specifications / Geometry

In general, wetland designs are unique for each site and application. However, there are number of geometric ratios and limiting depths for the design of a stormwater wetland that must be observed for adequate pollutant removal, ease of maintenance, and improved safety. Table C-4 provides the recommended physical specifications and geometry for the various stormwater wetland design variants.

Table C-4. Recommended Design Criteria for Stormwater Wetlands.

Design Criteria	Shallow Wetland	ED Shallow Wetland	Pond/ wetland	Pocket Wetland
Length to Width Ratio (minimum)	2:1	2:1	2:1	2:1
Extended Detention (ED)	No	Yes	Optional	Optional
Allocation of WQv Volume (pool/marsh/ED) in %	25/75/0	25/25/50	70/30/0 (Includes pond volume)	25/75/0
Allocation of Surface Area (deepwater/low marsh/high marsh/semi-wet) in %	20/35/40/5	10/35/45/10	45/25/25/5 (Includes pond surface area)	10/45/40/5
Forebay	Required	Required	Required	Optional
Micropool	Required	Required	Required	Required
Outlet Configuration	Reverse-slope pipe or hooded broad-crested weir	Reverse-slope pipe or hooded broad-crested weir	Reverse-slope pipe or hooded broad-crested weir	Hooded broad-crested weir

- The stormwater wetland should be designed with the recommended proportion of “depth zones.” Each of the four wetland design variants has depth zone allocations which are given as a percentage of the stormwater wetland surface area. Target allocations are found in Table 2.1. The four basic depth zones are:
    1. **Deepwater zone**
       From 1.5 to 6 ft deep. Includes the outlet micropool and deepwater channels through the wetland facility. This zone supports little emergent wetland vegetation but may support submerged or floating vegetation.
    2. **Low marsh zone**
       From 6 to 18 inches below the normal permanent pool or water surface elevation. This zone is suitable for the growth of several emergent wetland plant species.
    3. **High marsh zone**
       From 6 inches below the pool to the normal pool elevation. This zone will support a greater density and diversity of wetland species than the low marsh zone. The high marsh zone should have a higher surface area to volume ratio than the low marsh zone.
    4. **Semi-wet zone**
       Those areas above the permanent pool that are inundated during larger storm events. This zone supports several species that can survive flooding.

- A minimum dry weather flow path of 2:1 (length to width) is required from inflow to outlet across the stormwater wetland and should ideally be greater than 3:1. This path may be achieved by constructing internal dikes or berms, using marsh plantings, and by using multiple cells. Finger dikes are commonly used in surface flow systems to create serpentine configurations and prevent short-circuiting. Microtopography (contours along the bottom of a wetland or marsh that provide a variety of conditions for different species needs and increases the surface area to volume ratio) is encouraged to enhance wetland diversity.
- A 4- to 6-ft deep micropool must be included in the design at the outlet to prevent the outlet from clogging and resuspension of sediments, and to mitigate thermal effects.
- Maximum depth of any permanent pool areas should generally not exceed 6 ft.
- The volume of the extended detention must not comprise more than 50% of the total WQ_v, and its maximum water surface elevation must not extend more than 3 ft above the normal pool. Q_p and/or C_{pv} storage can be provided above the maximum WQ_v elevation within the wetland.
- The perimeter of all deep pool areas (4 ft or greater in depth) should be surrounded by safety and aquatic benches like those for stormwater ponds.
- The contours of the wetland should be irregular to provide a more natural landscaping effect.

Pretreatment / Inlets

- Sediment regulation is critical to sustain stormwater wetlands. A wetland facility should have a sediment forebay or equivalent upstream pretreatment. A sediment forebay is designed to remove incoming sediment from the stormwater flow prior to dispersal into the wetland. The forebay should consist of a separate cell, formed by an acceptable barrier. A forebay is to be provided at each inlet unless the inlet provides less than 10% of the total design storm inflow to the wetland facility.
- The forebay is sized to contain 0.1 inches per impervious acre of contributing drainage and should be 4 to 6 ft deep. The pretreatment storage volume is part of the total WQ_v requirement and may be subtracted from WQ_v for wetland storage sizing.
- A fixed vertical sediment depth marker shall be installed in the forebay to measure sediment deposition over time. The bottom of the forebay may be hardened (e.g., using concrete, paver blocks, etc.) to make sediment removal easier.
- Inflow channels are to be stabilized with flared riprap aprons, or the equivalent. Inlet pipes to the pond can be partially submerged. Exit velocities from the forebay must be nonerosive.

Outlet Structures

Flow control from a stormwater wetland is typically accomplished with the use of a concrete or corrugated metal riser and barrel. The riser is a vertical pipe or inlet structure that is attached to the base of the micropool with a watertight connection. The outlet barrel is a horizontal pipe attached to the riser that conveys flow under the embankment (see Figure C-17) The riser should be located within the embankment for maintenance access, safety, and aesthetics.

- Several outlets at varying depths in the riser provide internal flow control for routing of the water quality, channel protection, and overbank flood protection runoff volumes. The number of orifices can vary and is usually a function of the pond design.
    - For shallow and pocket wetlands, the riser configuration is typically comprised of a channel protection outlet (usually an orifice) and overbank flood protection outlet (often a slot or weir). The channel protection orifice is sized to release the channel protection storage volume over a 24-hour period (12-hour extended detention may be warranted in some cold-water streams). Since the water quality volume is fully contained in the permanent pool, no orifice sizing is necessary for this volume. As runoff from a water quality event enters the wet pond, it simply displaces that same volume through the channel protection orifice. Thus, an off-line shallow or pocket wetland providing only water quality treatment can use a simple overflow weir as the outlet structure.

- In the case of an extended detention (ED) shallow wetland, there is generally a need for an additional outlet (usually an orifice) that is sized to pass the extended detention water quality volume that is surcharged on top of the permanent pool. Flow will first pass through this orifice, which is sized to release the water quality ED volume in 24 hours. The preferred design is a reverse slope pipe attached to the riser, with its inlet submerged 1 ft below the elevation of the permanent pool to prevent floatables from clogging the pipe and to avoid discharging warmer water at the surface of the pond. The next outlet is sized for the release of the channel protection storage volume. The outlet (often an orifice) invert is located at the maximum elevation associated with the extended detention water quality volume and is sized to release the channel protection storage volume over a 24-hour period (12-hour extended detention may be warranted in some cold-water streams).
- Alternative hydraulic control methods to an orifice can be used and include the use of a broad-crested rectangular, V-notch, proportional weir, or an outlet pipe protected by a hood that extends at least 12 inches below the normal pool.

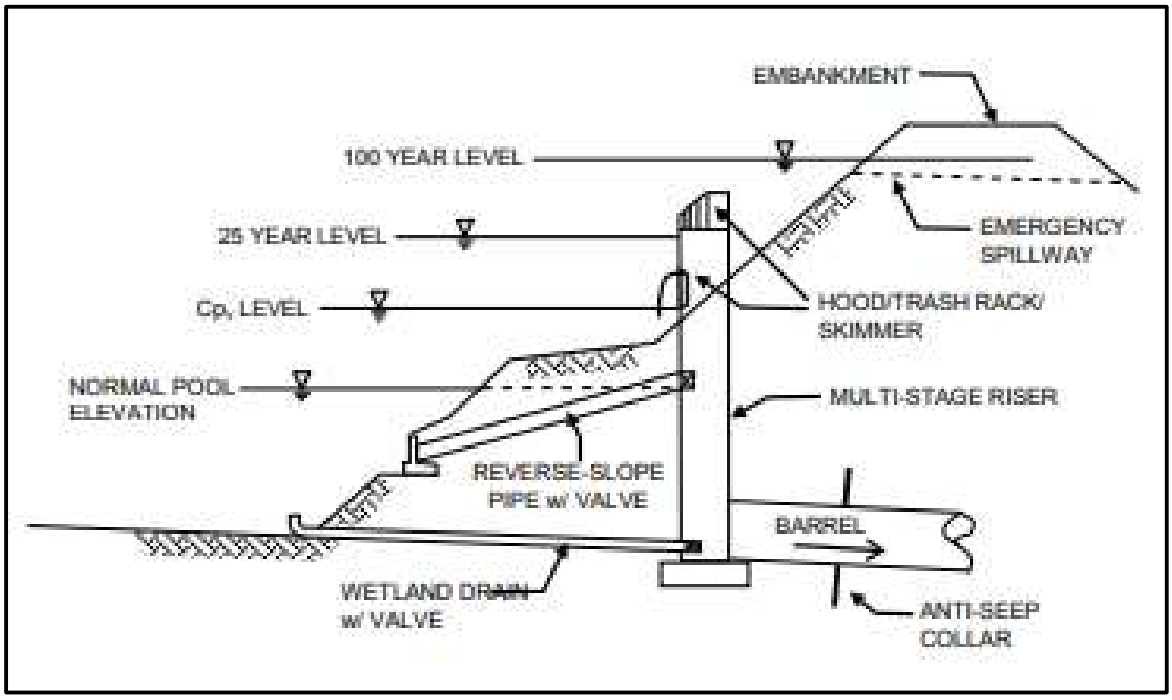


Figure C-17. Typical Wetland Facility Outlet Structure.

- The water quality outlet (if design is for an ED shallow wetland) and channel protection outlet should be fitted with adjustable gate valves or other mechanism that can be used to adjust detention time.
- Higher flows (overbank and extreme flood protection) flows pass through openings or slots protected by trash racks further up on the riser.
- After entering the riser, flow is conveyed through the barrel and is discharged downstream. Anti-seep collars should be installed on the outlet barrel to reduce the potential for pipe failure.
    - Riprap, plunge pools or pads, or other energy dissipators are to be placed at the outlet of the barrel to prevent scouring and erosion. If a wetland facility daylight to a channel with dry weather flow, care should be taken to minimize tree clearing along the downstream channel, and to reestablish a forested riparian zone in the shortest possible distance. See Section 4.5 (Energy Dissipation Design) for more guidance.

- The wetland facility must have a bottom drainpipe located in the micropool with an adjustable valve that can completely or partially dewater the wetland within 24 hours. (This requirement may be waived for coastal areas, where positive drainage is difficult to achieve due to very low relief)
- The wetland drain should be sized one pipe size greater than the calculated design diameter. The drain valve is typically a handwheel activated knife or gate valve. Valve controls shall be located inside of the riser at a point where they (a) will not normally be inundated and (b) can be operated in a safe manner.

See the design procedures in Chapter 7 for additional information and specifications on pond routing and outlet works.

Emergency Spillway

- An emergency spillway is to be included in the stormwater wetland design to safely pass flows that exceed the design storm flows. The spillway prevents the wetland's water levels from overtopping the embankment and causing structural damage. The emergency spillway must be located so that downstream structures will not be impacted by spillway discharges.
- A minimum of 1 ft of freeboard must be provided, measured from the top of the water surface elevation for the extreme flood to the lowest point of the dam embankment, not counting the emergency spillway.

Maintenance Access

- A maintenance right of way or easement must be provided to the wetland facility from a public or private road. Maintenance access should be at least 12 ft wide, have a maximum slope of no more than 15%, and be appropriately stabilized to withstand maintenance equipment and vehicles.
- The maintenance access must extend to the forebay, safety bench, riser, and outlet and, to the extent feasible, be designed to allow vehicles to turn around.
- Access to the riser is to be provided by lockable manhole covers, and manhole steps within easy reach of valves and other controls.
- Safety Features
- All embankments and spillways must be designed to State of Arkansas guidelines for dam safety (see Appendix E).
- Fencing of wetlands is not generally desirable but may be required by the local review authority. A preferred method is to manage the contours of deep pool areas through the inclusion of a safety bench (see above) to eliminate drop-offs and reduce the potential for accidental drowning.
- The principal spillway opening should not permit access by small children, and end walls above pipe outfalls greater than 48 inches in diameter should be fenced to prevent a hazard.

Landscaping

- A landscaping plan should be provided that indicates the methods used to establish and maintain wetland coverage. Minimum elements of a plan include delineation of landscaping zones, selection of corresponding plant species, planting plan, sequence for preparing wetland bed (including soil amendments, if needed) and sources of plant material.
- Landscaping zones include low marsh, high marsh, and semi-wet zones. The low marsh zone ranges from 6 to 18 inches below the normal pool. This zone is suitable for the growth of several emergent plant species. The high marsh zone ranges from 6 inches below the pool up to the normal pool. This zone will support greater density and diversity of emergent wetland plant species. The high marsh zone should have a higher surface area to volume ratio than the low marsh zone. The semi-wet zone refers to those areas above the permanent pool that are inundated on an irregular basis and can be expected to support wetland plants.
- The landscaping plan should provide elements that promote greater wildlife and waterfowl use within the wetland and buffers.

- Woody vegetation may not be planted on the embankment or allowed to grow within 15 ft of the toe of the embankment and 25 ft from the principal spillway structure.
- A wetland buffer shall extend 25 ft outward from the maximum water surface elevation, with an additional 15-ft setback to structures. The wetland buffer should be contiguous with other buffer areas that are required by existing regulations (e.g., stream buffers) or that are part of the overall stormwater management concept plan. No structures should be located within the buffer, and an additional setback to permanent structures may be provided.
- Existing trees should be preserved in the buffer area during construction. It is desirable to locate forest conservation areas adjacent to ponds. To discourage resident geese populations, the buffer can be planted with trees, shrubs, and native ground covers.
- The soils of a wetland buffer are often severely compacted during the construction process to ensure stability. The density of these compacted soils is so great that it effectively prevents root penetration and therefore may lead to premature mortality or loss of vigor. Consequently, it is advisable to excavate large and deep holes around the proposed planting sites and backfill these with uncompacted topsoil.
- Additional Site-Specific Design Criteria and Issues
    - Physiographic Factors - Local terrain design constraints
    - Low Relief – Providing wetland drain can be problematic.
    - High Relief – Embankment heights restricted.
- Soils
    - Hydrologic group “A” soils and some group “B” soils may require liner (not relevant for pocket wetland)

7. Design Procedures

Step 1. Compute runoff control volumes from the Stormwater Sizing Criteria

Calculate the Water Quality Volume (WQ_v) and design storm events. Details on the Stormwater Sizing Criteria are found in Chapter 2.

Step 2. Determine if the development site and conditions are appropriate for the use of a stormwater wetland.

Consider the Application and Site Feasibility Criteria in the Planning and Design Criteria section of this specification (Location and Siting)

Step 3. Confirm local design criteria and applicability.

Consider any special site-specific design conditions/criteria from the Planning and Design Criteria section of this specification (Additional Site-Specific Design Criteria and Issues). Check with the City Engineer to determine if there are any additional restrictions and/or surface water or watershed requirements that may apply.

Step 4. Determine pretreatment volume.

A sediment forebay is provided at each inlet unless the inlet provides less than 10% of the total design storm inflow to the pond. The forebay should be sized to contain 0.1 inches per impervious acre of contributing drainage and should be 4 to 6 ft deep. The forebay storage volume counts toward the total WQ_v requirement and may be subtracted from the WQ_v for subsequent calculations.

Step 5. Allocate the WQ_v volume among marsh, micropool, and ED volumes.

Use recommended criteria from Table C-4 of this specification.

Step 6. Determine wetland location and preliminary geometry, including distribution of wetland depth zones.

This step involves initially laying out the wetland design and determining the distribution of wetland surface area among the various depth zones (high marsh, low marsh, and deepwater). Set WQ_v permanent pool elevation

(and WQ_v-ED elevation for ED shallow wetland) based on volumes calculated earlier. See the Planning and Design Criteria section of this specification (Physical Specification / Geometry) for more details.

Step 7. Compute extended detention orifice release rate(s) and size(s).

ED Shallow Wetland: Based on the elevations established in Step 6 for the extended detention portion of the water quality volume, the water quality orifice is sized to release this extended detention volume in 24 hours. The water quality orifice should have a minimum diameter of 3 inches and should be adequately protected from clogging by an acceptable external trash rack. A reverse slope pipe attached to the riser, with its inlet submerged 1 ft below the elevation of the permanent pool, is a recommended design. Adjustable gate valves can also be used to achieve this equivalent diameter. The C_{pv} elevation is then determined from the stage-storage relationship. The invert of the channel protection orifice is located at the water quality extended detention elevation, and the orifice is sized to release the channel protection storage volume over a 24-hour period (12-hour extended detention may be warranted in some cold-water streams).

Step 8. Calculate design storm release rates and water surface elevations.

Set up a stage-storage-discharge relationship for the control structure for the design storms.

Step 9. Design embankment(s) and spillway(s).

Size emergency spillway, calculate 100-year water surface elevation, set top of embankment elevation, and analyze safe passage of the 100-year event. Attenuation may not be required.

Step 10. Design inlets, sediment forebay(s), outlet structures, maintenance access, and safety features.

See the Planning and Design Criteria section of this specification for more details.

Step 11. Prepare Vegetation and Landscaping Plan.

A landscaping plan for the wetland facility and its buffer should be prepared to indicate how aquatic and terrestrial areas will be stabilized and established with vegetation. See the Planning and Design Criteria section of this specification (Landscaping) for more details.

8. Inspection and Maintenance Requirements

Maintenance Considerations and Requirements in addition to Table C-5.

- Maintenance requirements for constructed wetlands are particularly high while vegetation is being established. Monitoring during these first years is crucial to the future success of the wetland as a stormwater structural control. Wetland facilities should be inspected after major storms (greater than 2 inches of rainfall) during the first year of establishment to assess bank stability, erosion damage, flow channelization, and sediment accumulation within the wetland. For the first 3 years, inspections should be conducted at least twice a year.
- A sediment marker should be in the forebay to determine when sediment removal is required.
- Accumulated sediments will gradually decrease wetland storage and performance. The effects of sediment deposition can be mitigated by the removal of the sediments.
- Sediments excavated from stormwater wetlands that do not receive runoff from designated hotspots are not considered toxic or hazardous material and can be safely disposed of by either land application or landfilling. Sediment testing may be required prior to sediment disposal when a hotspot land use is present. Sediment removed from stormwater wetlands should be disposed of according to an approved erosion and sediment control plan.
- Periodic mowing of the wetland buffer is only required along maintenance rights-of-way and the embankment. The remaining buffer can be managed as a meadow (mowing every other year) or forest.
- Regular inspection and maintenance are critical to the effective operation of stormwater wetlands as designed.

Table C-5. Typical Maintenance Activities for Wetlands.

Activity	Schedule
• Replace wetland vegetation to maintain at least 50% surface area coverage in wetland plants after the second growing season.	One-Time Activity
• Clean and remove debris from inlet and outlet structures. • Mow side slopes.	Frequently (3 to 4 times/year)
• Monitor wetland vegetation and perform replacement planting as necessary.	Semi-annual Inspection (first 3 years)
• Examine stability of the original depth zones and microtopographical features. • Inspect for invasive vegetation and remove where possible. • Inspect for damage to the embankment and inlet/outlet structures. Repair as necessary. • Note signs of hydrocarbon build-up and remove appropriately. • Monitor for sediment accumulation in the facility and forebay. • Examine to ensure that inlet and outlet devices are free of debris and operational.	Annual Inspection
• Repair undercut or eroded areas.	As Needed
• Harvest wetland plants that have been “choked out” by sediment build-up.	Annually
• Removal of sediment from the forebay.	5 to 7 years or after 50% of the total forebay capacity has been lost
• Monitor sediment accumulations and remove sediment when the pool volume has become reduced significantly, plants are “choked” with sediment, or the wetland becomes eutrophic.	10 to 20 years or after 25% of the wetland volume has been lost

9. References

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WQ-04 Bioretention

1. Description

Individual bioretention areas can serve impervious drainage areas of 2.5 acres or less; though several cells may be designed adjacent to each other to accommodate larger areas. Surface runoff is directed into a shallow landscaped depression that incorporates many of the pollutant removal mechanisms that operate in forested ecosystems. The primary component of a bioretention practice is the filter bed, which has a mixture of sand, soil, and organic material as the filtering media with a surface mulch layer. During storms, runoff temporarily ponds 6 inches above the cell surface and then rapidly filters through the bed. If the subsoil infiltration rate is less than 0.5 inches per hour, the filtered runoff is collected in an underdrain and returned to the storm drain system. The underdrain consists of a perforated pipe in a gravel layer installed along the bottom of the filter bed.

Bioretention can also be designed to infiltrate runoff into native soils. This can be done at sites with permeable soils, a low groundwater table, and a low risk of groundwater contamination. This design features the use of a “partial exfiltration” system that promotes greater groundwater recharge. Underdrains are only installed beneath a portion of the filter bed, above a stone “sump” layer, or eliminated altogether, thereby increasing stormwater infiltration.

2. Pollutant Removal Capabilities

Bioretention creates a good environment for runoff reduction, filtration, biological uptake, and microbial activity, and provides high pollutant removal. Bioretention can become an attractive landscaping feature with high amenity value and community acceptance. The following design pollutant removal rates are conservative average pollutant reduction percentages for design purposes derived from sampling data, modeling, and professional judgment.

- Total Suspended Solids – 80%
- Total Phosphorus – 60%
- Total Nitrogen – 50%
- Fecal Coliform – insufficient data
- Heavy Metals – 80%

3. Design Variations

The most important design consideration for the application of bioretention to development sites is the scale, as follows:

Rain Gardens. These are small, distributed practices designed to treat runoff from small areas, such as individual rooftops, driveways, and other on-lot features in single-family detached residential developments. Inflow is typically sheet flow, or can be concentrated flow with energy dissipation, when located at downspouts. Rain gardens do not count toward a Runoff Reduction credit unless operations and maintenance are addressed to ensure their longevity.

Bioretention Basins. These are structures treating parking lots and/or commercial rooftops, usually in commercial or institutional areas. Inflow can be either sheet flow or concentrated flow. Bioretention basins may also be distributed throughout a residential subdivision, but they should be located in common areas and within drainage easements, to treat a combination of roadway and lot runoff.

The major design goal for bioretention is to maximize runoff volume reduction and pollutant removal. To this end, designers may choose to go with the baseline design (Level 1) or choose an enhanced design (Level 2) that maximizes pollutant and runoff reduction. If soil conditions require an underdrain, bioretention areas can still qualify for the Level 2 design if they contain a stone storage layer beneath the invert of the underdrain. Table C-7 and C-8, in Section 5 outlines the Level 1 and 2 bioretention design guidelines.

4. Typical Details

Figures C-18 through C-22 provide some typical details for several bioretention configurations.

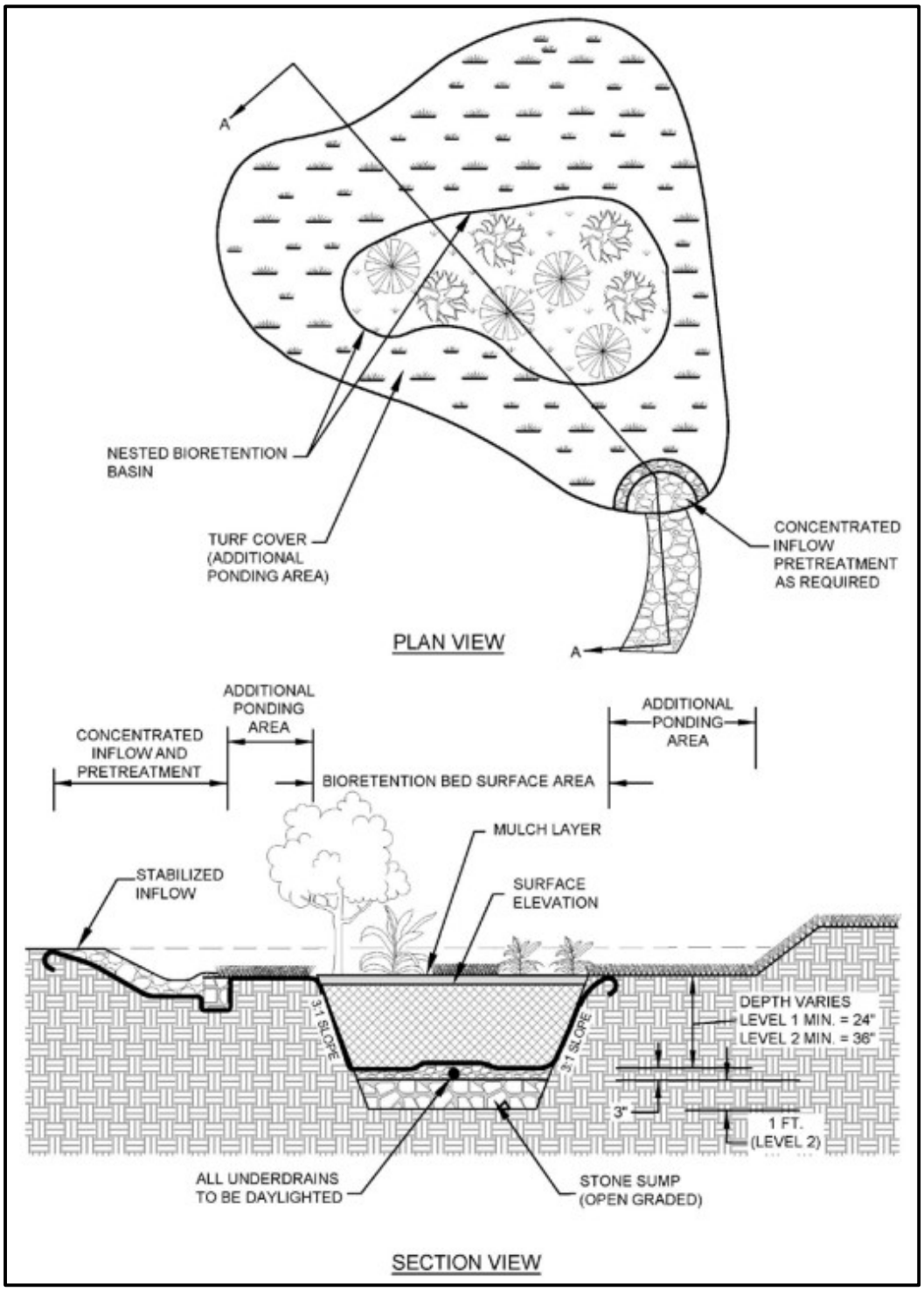


Figure C-18. Typical detail of Bioretention with additional surface ponding (Source: VADCR, 2010).

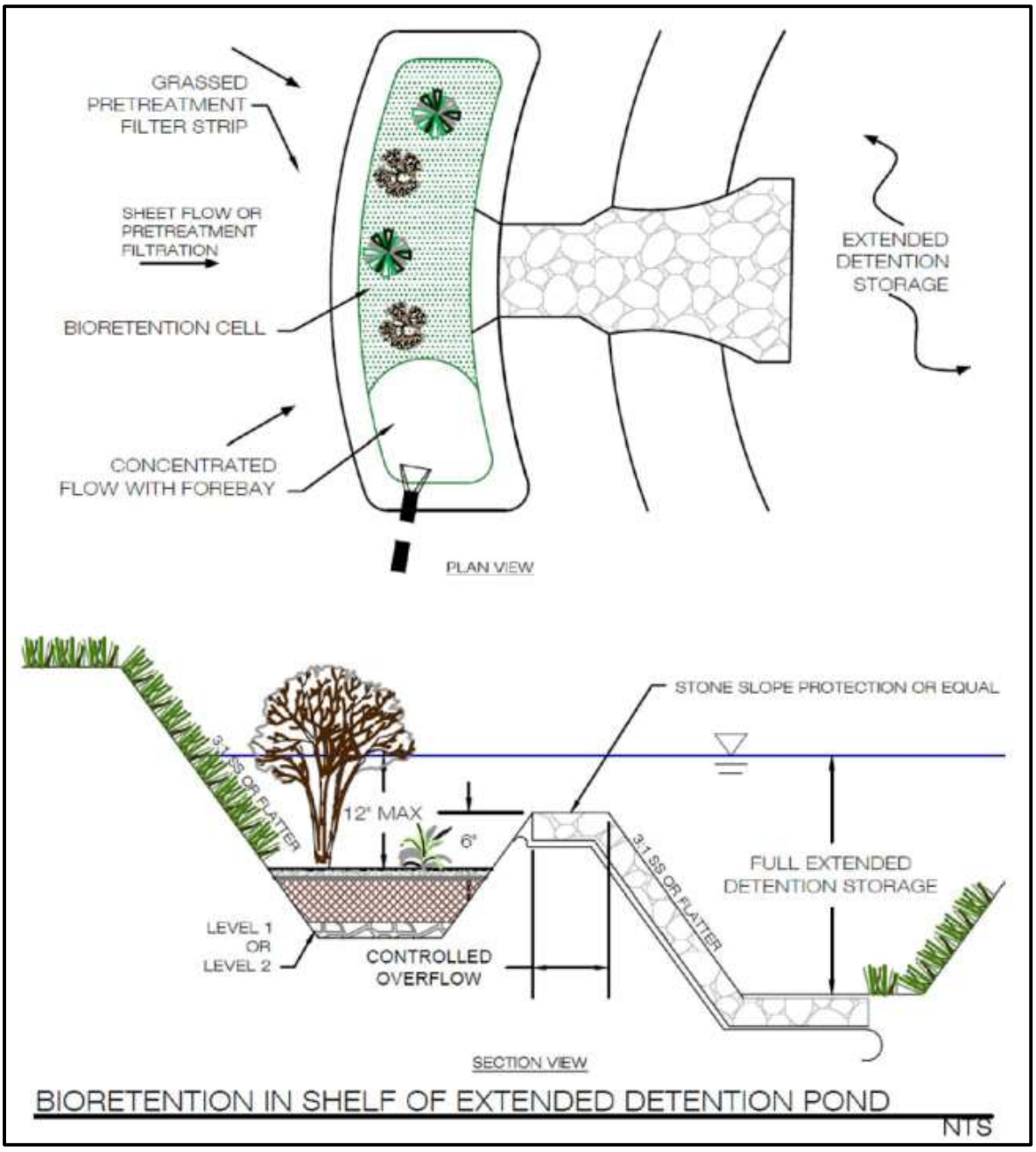


Figure C-19. Typical detail of a bioretention basin within the upper shelf of an ED pond (Source: VADCR, 2010).

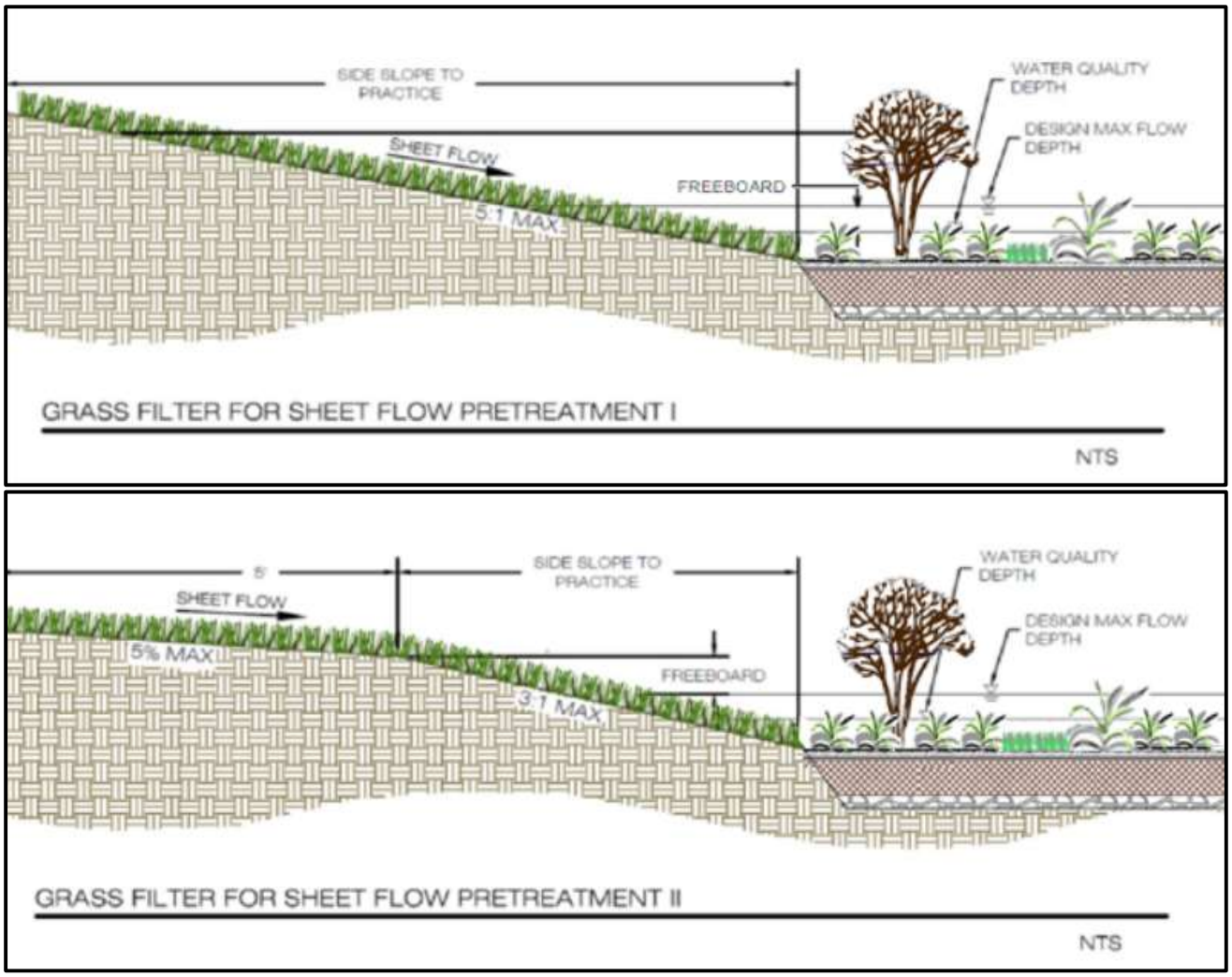


Figure C-20. Pretreatment option - grass filter for sheet flow (Source: VADCR, 2010).

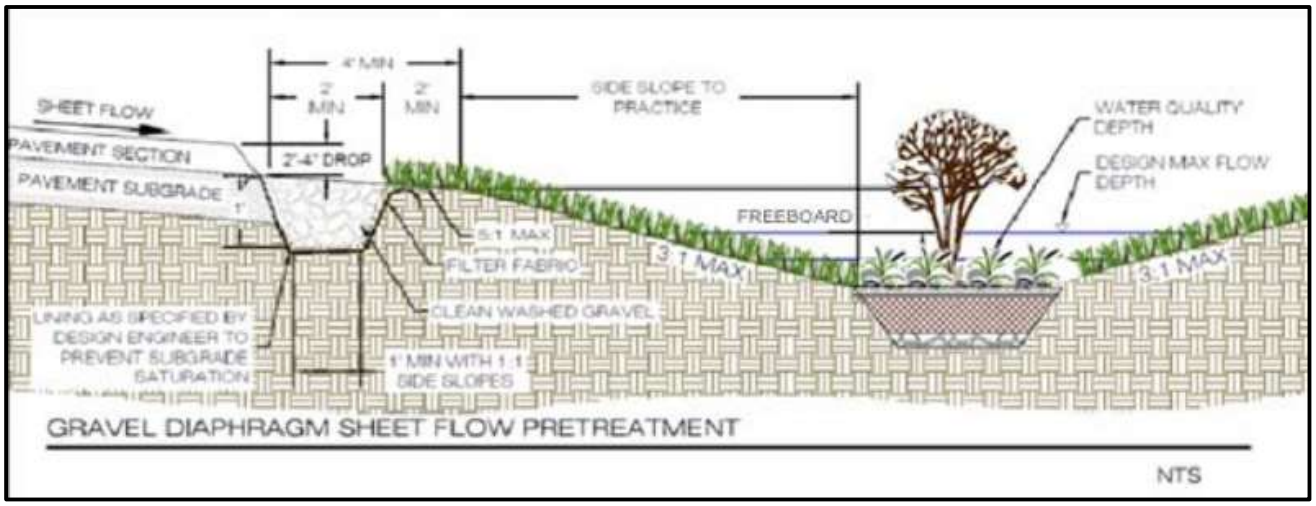


Figure C-21. Pretreatment option – gravel diaphragm for sheet flow from impervious or pervious (Source: VADCR, 2010).

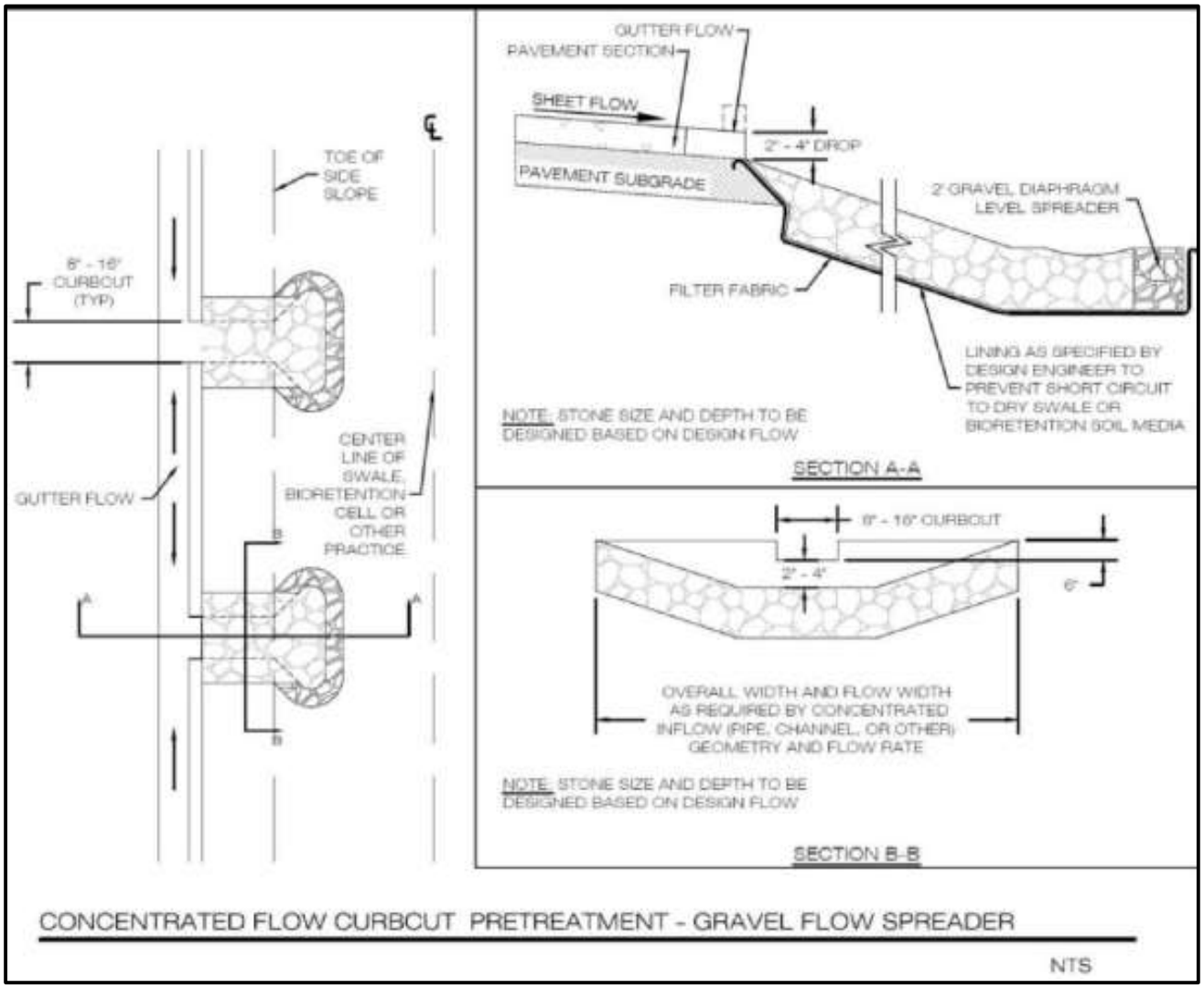


Figure C-22. Pre-Treatment Option – Gravel Flow Spreader for Concentrated Flow Outside of ROW (Source: VADCR, 2010).

5. Physical Feasibility & Design Applications

Table C-6. Bioretention design criteria.

Category	Details
Building and road setbacks[1] (Section 5):	0-to-0.5-acre CDA = 10 ft. if down-gradient from building/road; 50 ft. if up-gradient. 0.5-to-2.5-acre CDA = 25 ft. if down-gradient from building/road or level; 100 ft. if up-gradient. Refer to additional setback criteria in Section 5)
Sizing	(Section 6.1)
Surface Area (sq. ft.)	= (WQv – the volume reduced by an upstream BMP) / Storage Depth[2]
Recommended maximum contributing drainage area per cell	2.5-acres; 5-acre total
Maximum Ponding Depth: Maximum Ponding Elevation:	6 to 12 inches[3] > 0.5 ft. below lowest adjacent grade of structure
Filter Media Depth minimum: recommended maximum:	24 inches 6 ft.
Media & Surface Cover (Section 6.6, Appendix C) =	mixed onsite or supplied by vendor; the final composition shall be as specified in Appendix C.
Sub-soil Testing (Section 6.2)	not needed if an underdrain is used; Min infiltration rate > 0.5 inch/hour in order to remove the underdrain requirement.
Underdrain (Section 6.7)	PVC or Corrugated HDPE with clean-outs or, none, if soil infiltration requirements are met (Section 6.2)
Inflow:	sheet flow, curb cuts, trench drains, concentrated flow, or the equivalent
Geometry (Section 6.3): Length of shortest flow path/Overall length	= 0.3; or other design methods used to prevent short- circuiting; a one-cell design (not including the pre- treatment cell).
Pre-treatment (Section 6.4):	a pretreatment cell, grass filter strip, gravel diaphragm, gravel flow spreader, or another approved (manufactured) pre-treatment structure.
Conveyance & Overflow	(Section 6.5)
Planting Plan (Section 6.8):	a planting template to include perennials, grasses, sedges, or shrubs to achieve a surface area coverage of at least 75% within 2 years by using the recommended spacing in Table C-11 Material Specifications
Deeded Maintenance O&M Plan (Section 8)	

1. These are recommendations for simple, shallow building foundations or road subgrade. All designs shall be prepared / reviewed by appropriately licensed professionals. Also, special footing design or use of impermeable linings with bioretention may be used to justify a reduction of the setbacks noted above. The urban bioretention WQ-05 may also be used to reduce setbacks.
2. Storage depth is based on the Void Ratio (Vr) of the soil media and gravel layers and their respective thicknesses, plus the surface ponding depth. Compute as described in Section 6.1.
3. A ponding depth of 6 inches is preferred. Ponding depths greater than 6 inches will require a specific planting plan to ensure appropriate plant selection (Section 6.8).

5.1 Physical Feasibility

Bioretention can be applied in most soils or topography since runoff simply percolates through an engineered soil bed and can be returned to the stormwater system if the infiltration rate of the underlying soils is low. Key constraints with bioretention include the following:

Available Space. Planners and designers can assess the feasibility of using bioretention facilities based on a simple relationship between the contributing drainage area and the corresponding required surface area. The bioretention surface area will be approximately 3% to 10% of the contributing drainage area, depending on the imperviousness of the contributing drainage area (CDA), the subsoil infiltration rate, and the desired bioretention design level.

Site Topography. Bioretention is best applied when the grade of contributing slopes is greater than 1% and less than 5%. Terracing or other inlet controls may be used to slow runoff velocities entering the facility.

Available Hydraulic Head. Bioretention is fundamentally constrained by the invert elevation of the existing conveyance system to which the practice discharges (i.e., the bottom elevation needed to tie the underdrain from the bioretention area into the storm drain system.) In general, 3 ft of elevation above this invert is needed to create the hydraulic head needed to drive stormwater through a proposed bioretention filter bed. Less hydraulic head is needed if the underlying soils are permeable enough to dispense with the underdrain.

Water Table. Bioretention shall always be separated from the water table to ensure that groundwater does not intersect the filter bed. Mixing can lead to possible groundwater contamination or failure of the bioretention facility. A separation distance of 2 ft is required between the bottom of the excavated bioretention area and the seasonally high ground water table.

Utilities, Infrastructure, and Setbacks. Designers should ensure that future tree canopy growth in the bioretention area will not interfere with existing overhead utility lines. Interference with underground utilities should also be avoided, particularly water and sewer lines. Local utility design guidance should be consulted in order to determine the horizontal and vertical clearance required between stormwater infrastructure and other dry and wet utility lines. At a minimum, locate bioretention basins at least 10 ft from down-gradient wet utility lines. Dry utility lines such as electric, cable and telephone may cross under bioretention areas if they are double cased, if approved by the utility where applicable. To avoid the risk of seepage, do not allow bioretention areas to be hydraulically connected to structure foundations or pavement / pavement subgrade. Setbacks to structures and roads vary, based on the scale of the bioretention design (see Table C-6). At a minimum, bioretention basins shall be located a horizontal distance of 100 ft from any water supply well or septic system. Bioretention basins can be constructed closer to structures and roads if appropriately designed and constructed. Please see WQ-05 for additional information on ROW applications.

Soils. Soil conditions do not constrain the use of bioretention, although they determine whether an underdrain is needed. Low permeability soils in Hydrologic Soil Group (HSG) C or D usually require an underdrain, whereas HSG A soils and most HSG B soils generally do not. Initially, soil infiltration rates can be estimated from NRCS soil data, but they must be confirmed by an on-site infiltration evaluation.

Contributing Drainage Area. Bioretention cells work best with smaller contributing drainage areas, where it is easier to achieve flow distribution over the filter bed without experiencing erosive velocities and excessive ponding times. Typical drainage area size can range from 0.1 to 2.5 acres of impervious cover due to limitations on the ability of bioretention to effectively manage large volumes and peak rates of runoff. However, if hydraulic considerations are adequately addressed to manage the potentially large peak inflow of larger drainage areas (such as off-line or low-flow diversions, forebays, etc.), there may be case-by-case instances where the City Engineer may allow these recommended maximums to be adjusted. In such cases, the bioretention facility shall be located within the drainage area so as to capture the Water Quality Volume (WQv) equally from the entire contributing area, and not fill the entire volume from the immediately adjacent area, thereby bypassing the runoff from the more remote portions of the site.

Land Uses with Potential for High Pollutant Loading. Runoff within areas designated to have potential for high pollutant loading shall not be treated with infiltrating bioretention (i.e., constructed without an underdrain). An impermeable bottom liner and an underdrain system may be utilized, at the discretion of the City Engineer, when bioretention is used to receive and treat such runoff.

Floodplains. Bioretention areas shall be constructed outside the limits of the 1-percent annual-chance floodplain.

No Irrigation or Baseflow. The planned bioretention area shall not receive baseflow, irrigation water, chlorinated wash-water or other such non-stormwater flows that are not stormwater runoff, except for irrigation as necessary for the survival of plantings within the bioretention area.

5.2 Potential Bioretention Applications:

Bioretention can be used wherever water can be conveyed to a surface area. Bioretention has been used at commercial, institutional, and residential sites in spaces that are traditionally pervious and landscaped. It should be noted that special care must be taken to provide adequate pre-treatment for bioretention cells in space-constrained high traffic areas. Typical locations for bioretention include the following:

Parking lot islands. The parking lot grading is designed for sheet flow towards linear landscaping areas and parking islands between rows of spaces. Curb-less pavement edges can be used to convey water into a depressed island landscaping area. Curb cuts can also be used for this purpose, but they are more prone to blockage, clogging and erosion.

Parking lot edge. Small parking lots can be graded so that flows reach a curb-less pavement edge or curb cut before reaching catch basins or storm drain inlets. The turf at the edge of the parking lot functions as a filter strip to provide pre-treatment for the bioretention practice. The depression for bioretention is located in the pervious area adjacent to the parking lot.

Right of Way or commercial setback. A linear configuration can be used to convey runoff in sheet flow from the roadway, or a grass channel or pipe may convey flows to the bioretention practice. See Section 5.1 for specific setback recommendations.

Courtyards. Runoff collected in a storm drain system or roof leaders can be directed to courtyards or other pervious areas on site where bioretention can be installed.

Unused pervious areas on a site. Storm flows can be redirected from a storm drainpipe to discharge into a bioretention area.

Dry Extended Detention (ED) basin. A bioretention cell can be located on an upper shelf of an extended detention basin, after the sediment forebay, in order to boost treatment. Depending on the ED basin design, the designer may choose to locate the bioretention cell in the bottom of the basin. However, the design must account for the potentially deeper ponding depths (greater than 6 or 12 inches) associated with extended detention.

Retrofitting. Numerous options are available to retrofit bioretention in the urban landscape, as described in WQ-05, Urban Bioretention

6. Design Criteria

6.1 Sizing of Bioretention Practices

6.1.1 Stormwater Quality

Sizing of the surface area (SA) for bioretention practices is based on the computed Water Quality Volume (WQv) of the contributing drainage area and the storage provided in the facility. The required surface area (in sq feet) is computed as the Water Quality Volume (in cubic feet) divided by the equivalent storage depth (in feet). The equivalent storage depth is computed as the depth of soil media, gravel, or surface ponding (in feet) multiplied by the void ratio. The maximum allowable Void Ratios (Vr) are shown in Table C-7.

Table C-7. Maximum Allowable Void Ratios.

Type	V_r
Bioretention Soil Media (See Section 6.6)	0.40
Gravel	0.35
Surface Storage	1.0

The equivalent storage depth for bioretention with, for example, 2 ft of soil media, a 12-inch layer of gravel, and a 6-inch surface ponding depth is therefore computed as:

Weighted Storage Depth = $D_w = V_{r1}(D_1) + V_{r2}(D_2) + \cdots$ **Eq. C-1**

(0.4 x 2 ft) + (0.35 x1 ft) + (1.0 x 0.5 ft) = 1.65ft

Where V_{r1} and D_1 are for the first layer, V_{r2} and D_2 are for the second layer, etc.

While this method is simplistic, simulation modeling has indicated that it yields a total storage volume somewhat equivalent to 80% total average annual rainfall volume removal for infiltration rates from 0.5 inches/hour through 1.2 inches/hour. Depending on the in-situ subsurface infiltration rate, surface area size decreases may be granted at the discretion of City Engineer.

The Level 1 Bioretention Surface Area (SA), for the above example, Equation C-1, is computed as:

SA (sq. ft.) = [WQ_v – the volume reduced by any upstream BMP] /D_w **Eq. C-2**

Where: SA = Minimum surface area of bioretention filter (sq. ft.)

WQ_v = Water Quality Volume (cu. ft.) = [(1.05 in.) (Rv)(A)*43560/12]

D_W = Weighted Storage Depth as per Equation C-1

Where: A = Area in acres

Rv = the volumetric runoff coefficient

For example, a Bioretention Surface Area (using the media described above) for a treated area A = 1 acre, volumetric runoff coefficient Rv = 0.8, with no upstream BMPs would require a minimum design surface area of:

SA = [(1.05 * 0.8*1*43560) / 12] / 1.65 = 1848 sq ft

Equations C-1 and C-2 should be modified with the addition of any surface or subsurface storage components (e.g., additional area of surface ponding, subsurface storage chambers, etc.).

6.1.2 Stormwater Quantity

Designers may be able to create additional surface storage by expanding the surface ponding footprint in order to accommodate a greater quantity credit for channel and/or flood protection, without necessarily increasing the soil media footprint. In other words, the engineered soil media would only underlie part of the surface area of the bioretention (as shown in Figure C-23).

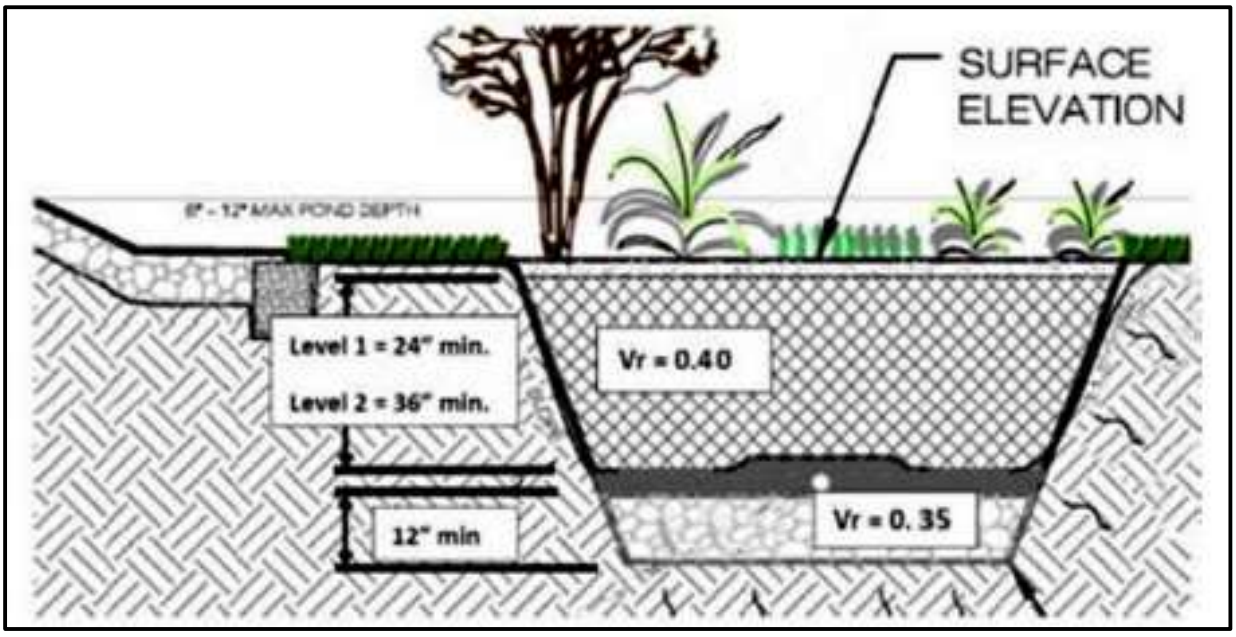


Figure C-23. Typical Level 2 Bioretention Section with void ratios for volume computations

In this regard, the ponding footprint can be increased as follows to allow for additional storage:

- 50% surface area increase if the ponding depth is 6 inches or less.
- 25% surface area increase if the ponding depth is between 6 and 12 inches.

These values may be modified as additional data on the long-term permeability of bioretention filters becomes available.

The removal of volume by bioretention changes the runoff depth entering downstream flood control facilities. An approximate approach to accounting for this in reducing the size of peak flow detention facilities is to calculate an "effective SCS curve number" (CN_{adj}) which is less than the actual curve number (CN) based on typical SCS parameters. CN_{adj} can then be used in hydrologic calculations and in routing; however, the CN must be recomputed for each storm or rainfall volume analyzed. The method can also be used for other hydrologic methods in which a reduction in runoff volume is possible.

6.2 Soil Infiltration Rate Testing

In order to determine if an underdrain will be needed, one must measure the infiltration rate of subsoils at the invert elevation of the bioretention area, as noted in the soil testing requirements for each scale of bioretention, in Design Table C-6. The infiltration rate of subsoils must exceed 0.5 inch per hour for bioretention basins. On-site soil infiltration rate testing procedures are outlined in Appendix C. Soil testing is not needed for bioretention areas where an underdrain is used.

6.3 BMP Geometry Bioretention

basins must be designed with internal flow path geometry such that the treatment mechanisms provided by the bioretention are not bypassed or short-circuited. Examples of short-circuiting include inlets or curb cuts that are very close to outlet structures (see Figure C-24), or incoming flow that is diverted immediately to the underdrain through stone layers. Short-circuiting can be particularly problematic when there are multiple curb cuts or inlets.



Figure C-24. Examples of short-circuiting at bioretention facilities (Source: VADCR, 2010).

In order for these bioretention areas to have an acceptable internal geometry, the "travel time" from each inlet to the outlet should be maximized, and incoming flow must be distributed as evenly as possible across the filter surface area.

One important characteristic is the length of the shortest flow path compared to the overall length, as shown in Figure C-25 below. In this figure, the ratio of the shortest flow path to the overall length is represented as:

SPF/L **Eq. C-3**

Where: SFP = length of the shortest flow path

L = length from the most distant inlet to the outlet

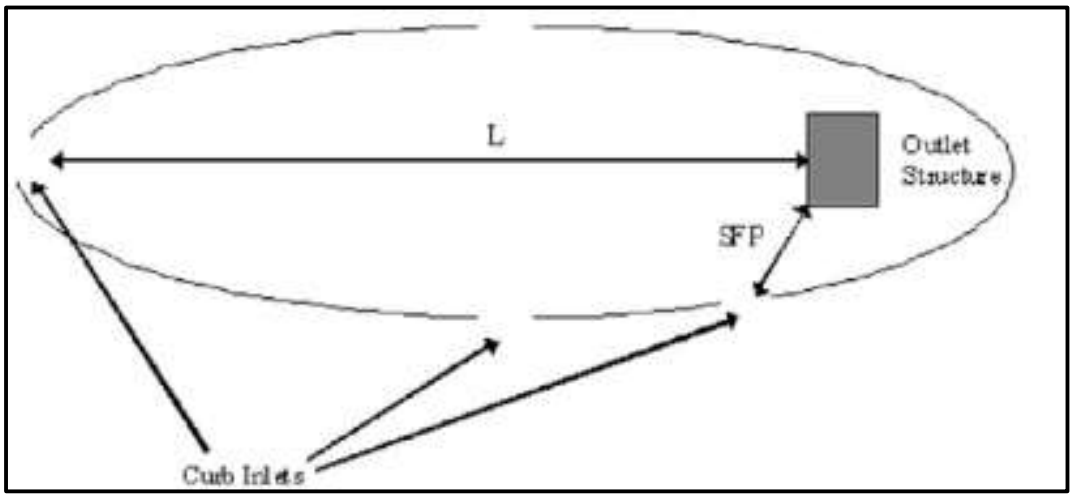


Figure C-25. Diagram showing shortest flow path as part of BMP geometry (Source: VADCR, 2010).

For Level 1 designs, the SFP/L ratio must be 0.3 or greater; the ratio must be 0.8 or greater for Level 2 designs. In some cases, due to site geometry, some inlets may not be able to meet these ratios. However, the drainage area served by such inlets should constitute no more than 20% of the contributing drainage area. Alternately, the designer may incorporate other design features that prevent short-circuiting, including features that help spread and distribute runoff as evenly as possible across the filter surface.

Field experience has shown that improperly prepared and poorly compacted soil media immediately around a raised inlet/outlet structures is prone to scouring, erosion and, thus, short-circuiting of the treatment mechanism. For example, water can flow straight down through scour holes or sinkholes to the underdrain system (Hirschman et al., 2009). Appropriate design options shall be used, and good construction practices implemented to prevent this type of scouring. The designer should ensure that incoming flow is spread as evenly as possible across the filter surface to maximize the treatment potential. See Figure C-26.

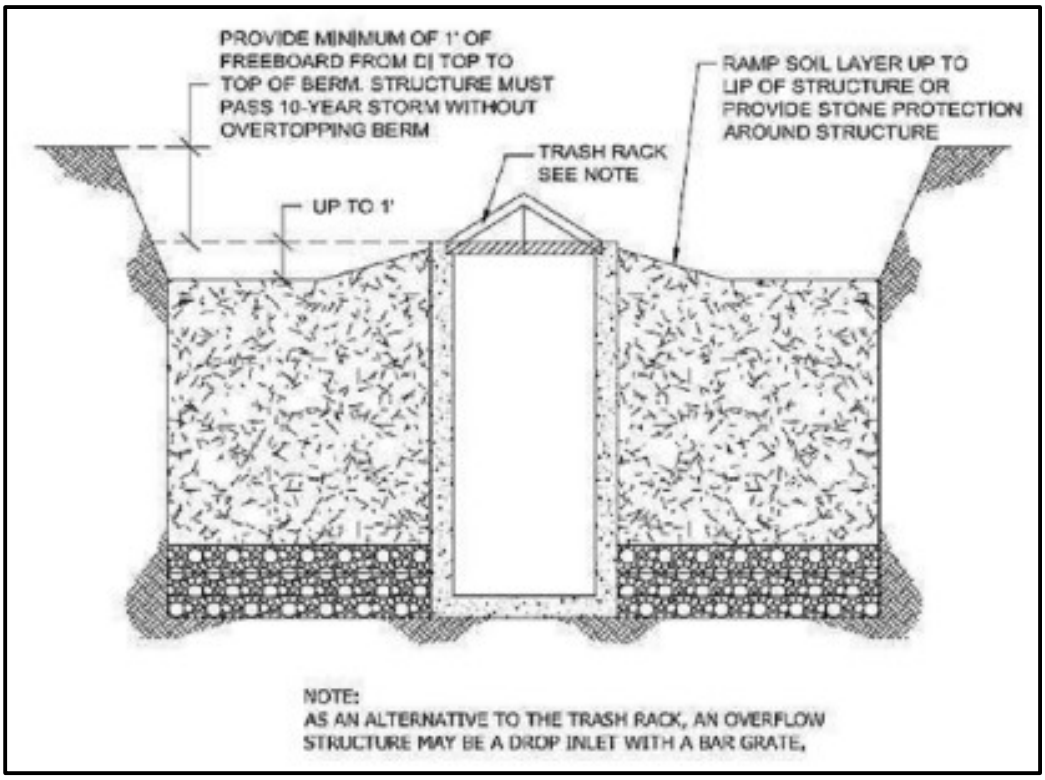


Figure C-26. Typical Detail of how to prevent bypass or short-circuiting around the overflow structure (Source: VADCR, 2010).

6.4 Conveyance and Overflow

For On-line Bioretention, an overflow structure shall be incorporated into on-line designs to safely convey larger storms through the bioretention area. The following criteria apply to overflow structures:

- The overflow associated with the design storms of the 2-, 5-, 10-, 25-, and 50-year frequencies referenced in the Drainage Criteria Manual shall be controlled so that velocities are non-erosive at the inlet and outlet locations (i.e., to prevent downstream erosion).
- Common overflow systems within bioretention practices consist of an inlet structure, where the top of the structure is placed at the maximum water surface elevation of the bioretention area, which is typically 6 to 12 inches above the surface of the filter bed (6 inches is the preferred ponding depth).
- The overflow capture device (typically a yard inlet) shall be scaled to the application – this may be a landscape grate inlet or a commercial-type structure.
- The filter bed surface should generally be flat so the bioretention area fills up like a bathtub.

Off-line Bioretention: Off-line designs are preferred (see Figure C-27 for an example). One common approach is to create an alternate flow path at the inflow point into the structure such that when the maximum ponding depth is reached, the incoming flow is diverted past the facility. In this case, the higher flows do not pass over the filter bed and through the facility, and additional flow is able to enter as the ponding water filtrates through the soil media.

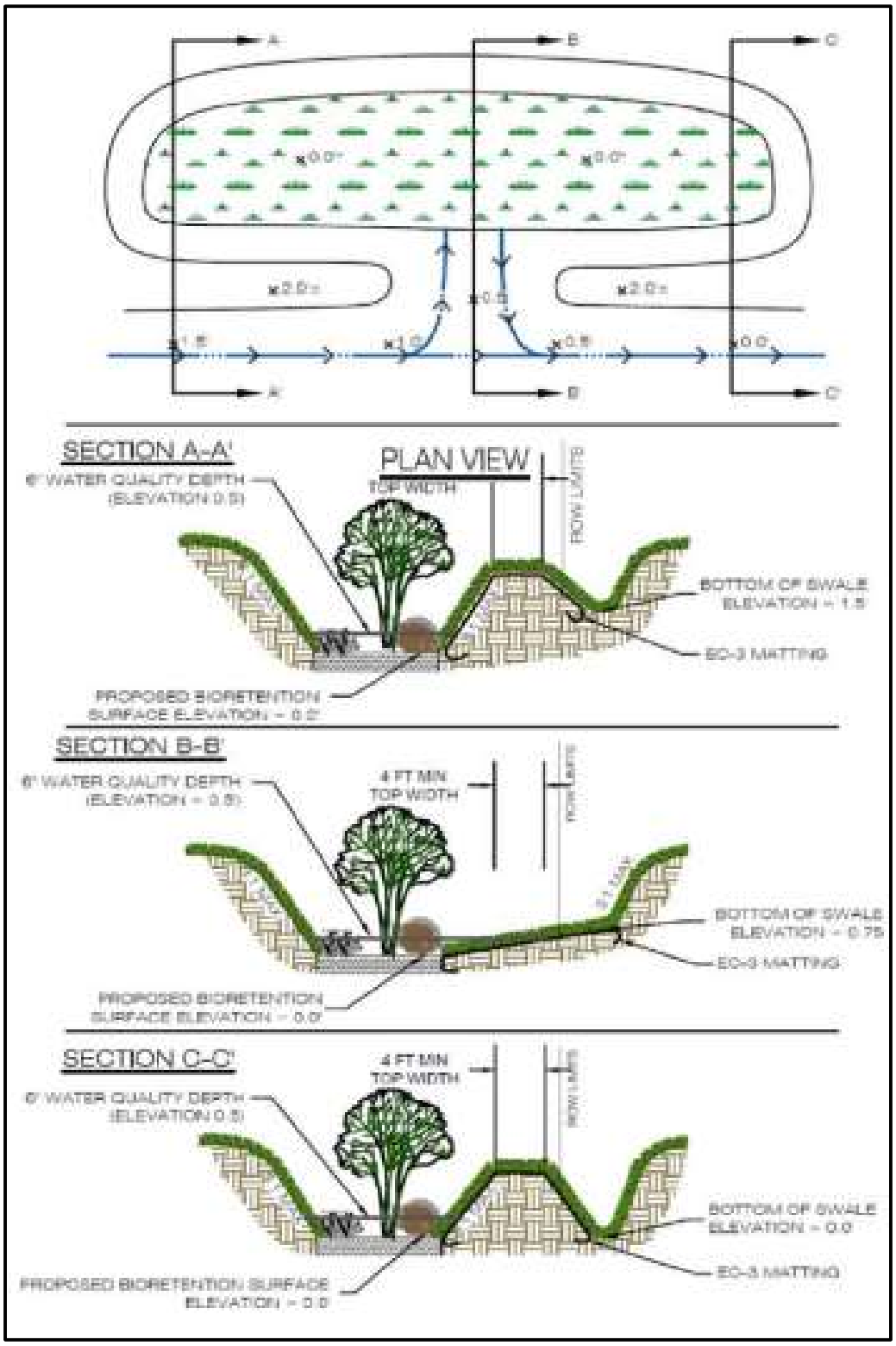


Figure C-27. Typical details for off-line biofiltration (Source: VADCR, 2010).

Another option is to utilize a low-flow diversion or flow splitter at the inlet to allow only the water quality volume to enter the facility. This may be achieved with a weir or curb opening sized for the target flow, in combination with a bypass channel. Using a weir or curb opening helps minimize clogging and reduces the maintenance frequency. (Further guidance on determining the water quality volume design peak flow rate will be necessary to ensure proper design of the diversion structure.)

6.5 Pre-treatment

Pre-treatment of runoff entering bioretention areas is necessary to trap coarse sediment particles before they reach and prematurely clog the filter bed. Pre-treatment measures must be designed to evenly spread runoff across the entire width of the bioretention area. Several pre-treatment measures are feasible, depending on the scale of the bioretention practice and whether it receives sheet flow, shallow concentrated flow, or deeper concentrated flows. The following are appropriate pretreatment options:

For Bioretention Basins:

- ***Pre-treatment Cells (channel flow):*** Similar to a forebay, this cell is located at piped inlets or curb cuts leading to the bioretention area and consists of an energy dissipater sized for the expected rates of discharge. It has a storage volume equivalent to at least 15% of the total water quality volume (inclusive) with a 2:1 length-to-width ratio. The cell may be formed by a stone check dam or an earthen or rock berm. Pretreatment cells do not need underlying engineered soil media, in contrast to the main bioretention cell.
- ***Grass Filter Strips (for sheet flow):*** Grass filter strips extend from the edge of pavement to the bottom of the bioretention basin at a 5:1 slope or flatter. Alternatively, provide a combined 5 ft of grass filter strip at a maximum 5% (20:1) slope and 3:1 or flatter side slopes on the bioretention basin. (See Figure C-20)
- ***Gravel or Stone Diaphragms (sheet flow).*** A gravel diaphragm located at the edge of the pavement shall be oriented perpendicular to the flow path to pre-treat lateral runoff, with a 2-to-4-inch drop. The stone must be sized to resist erosion based on to the expected rate of discharge. (See Figure C-21)
- ***Gravel or Stone Flow Spreaders (concentrated flow).*** The gravel flow spreader is located at curb cuts, downspouts, or other concentrated inflow points, and shall have a 2-to-4-inch elevation drop from a hard-edged surface into a gravel or stone diaphragm. The gravel shall extend the entire width of the opening and create a level stone weir at the bottom or treatment elevation of the basin. (See Figure C-22)
- ***Innovative or Proprietary Structure:*** An approved proprietary structure with demonstrated capability of reducing sediment and hydrocarbons may be used to provide pre-treatment.

6.6 Filter Media and Surface Cover

The filter media and surface cover are the two most important elements of a bioretention facility in terms of long-term performance. The following are key factors to consider in determining an acceptable soil media mixture.

- **General Filter Media Composition.** Two soil mix designs are provided in Appendix C at the end of this specification. These mix designs are based on regional availability of materials and the desired infiltration characteristics. The post-construction bioretention feature must provide adequate infiltration capacity, however, so pre- and post-construction infiltration testing based on the mix design should be performed using the available materials. Where possible, on-site soils should be used for economy provided infiltration characteristics can be met. In this case, additional preconstruction tests should include particle size, or gradation analyses of these soils in accordance with ASTM D-422.
- **Cation Exchange Capacity (CEC).** The CEC of a soil refers to the total amount of positively charged elements that a soil can hold; it is expressed in milliequivalents per 100 grams (meq/100g) of soil. Soils with CECs exceeding 10 meq/100g are preferred for pollutant removal. For agricultural purposes, these cations are basic cations of calcium (Ca2+), magnesium (Mg2+), potassium (K+) and sodium (Na+) and the acidic cations of hydrogen (H+) and aluminum (Al3+). The CEC of the soil is determined in part by the amount of clay and/or humus or organic matter present. Increasing the organic matter content of any soil will help to increase the CEC, since it also holds cations as does clay.

- **Infiltration Rate.** The bioretention soil media shall have a minimum infiltration rate of 1 to 2 inches per hour (a proper soil mix will have an initial infiltration rate that is significantly higher).
- **Depth.** The standard minimum filter bed depth ranges from 24 and 36 inches for Level 1 and Level 2 designs, respectively, (18 to 24 inches for rain gardens). If trees are included in the bioretention planting plan, tree planting holes in the filter bed must be hand-excavated and extend at least 4 ft deep to provide enough soil volume for the root structure of mature trees. Use turf, perennials, or shrubs instead of trees to landscape shallower filter beds.
- **Mulch.** A 3-to-4-inch layer of mulch on the surface of the filter bed enhances plant survival, suppresses weed growth, and pre-treats runoff before it reaches the filter media. Shredded, aged hardwood mulch or pine straw make very good surface cover, as they retain a significant amount of nitrogen and typically will not float away.
- **Alternative to Mulch Cover.** In some situations, designers may consider alternative surface covers such as turf, native groundcover, erosion control matting (coir or jute matting), river stone, or pea gravel. The decision regarding the type of surface cover to use should be based on function, cost, and maintenance.

6.7 Underdrain and Underground Storage Layer

Level 1 designs require an underdrain surrounded by a jacket of 1 inch stone unless the minimum of 0.5 inch/hour infiltration rate is met. For some Level 2 designs on an underdrain will not be required (where soil infiltration rates meet minimum standards; see Section 6.2 and Section 3 design tables). For Level 2 designs with an underdrain, an underground storage layer of at 12 inches shall be incorporated below the invert of the underdrain. The depth of the storage layer will depend on the target treatment and storage volumes needed to meet water quality, channel protection, and/or flood protection criteria. However, the bottom of the storage layer must be at least 2 ft above the seasonally high-water table. The storage layer shall consist of approved clean, washed stone or an approved infiltration module.

All bioretention basins shall include observation wells. The observation wells shall be tied into any T's or Y's in the underdrain system, and shall extend upwards to be flush with the surface, with a vented cap. In addition, cleanout pipes shall be provided if the contributing drainage area exceeds 1 acre.

6.8 Bioretention Planting Plans

A landscaping plan must be provided for each bioretention area. Minimum plan elements shall include the proposed bioretention template to be used, delineation of planting areas, the planting plan, including the size, the list of planting stock, sources of plant species and the planting sequence, including post-nursery care and initial maintenance requirements. The planting plan must be prepared by a licensed landscape architect in order to tailor the planting plan to the site-specific conditions.

Native plant species are preferred over non-native species, but some ornamental species may be used for landscaping effect if they are not aggressive or invasive. The planting template refers to the form and combination of native trees, shrubs, and perennial ground covers that maintain the appearance and function of the bioretention area. Planting templates may be of the following types:

Ornamental planting. This option includes perennials, sedges, grasses and/or trees in a mass bed planting. This template is acceptable for commercial sites where visibility is important. This template requires maintenance much like traditional landscape beds.

Forest-type planting. This option plants a variety of seedlings and saplings modeled from characteristics of a forest ecosystem. This template is appropriate for large bioretention areas located at wooded edges or where a wooded buffer is desired.

Meadow Planting. This is a lower maintenance approach that focuses on the herbaceous layer and may resemble a wildflower meadow or prairie (e.g., with Joe Pye weed, black-eyed Susan, sedges, grasses, etc.). The goal is to establish a more natural look that may be appropriate if the facility is located in a lower maintenance area (e.g., further from buildings and parking lots). Shrubs and trees may be incorporated. Erosion control matting may be used in lieu of the conventional mulch layer.

Turf. This option is typically restricted to on-lot micro-bioretention applications, such as a front yard rain garden. Grass species should be selected that have dense cover, are relatively slow growing, and require the least mowing and chemical inputs (e.g., fine fescue, tall fescue).

Perennial garden. This option uses herbaceous plants and native grasses to create a garden effect with seasonal cover. It may be employed in both micro-scale and small scale bioretention applications. This option is attractive, but it requires more maintenance in the form of weeding.

Perennial garden with shrubs. This option provides greater vertical form by mixing native shrubs and perennials together in the bioretention area. This option is frequently used when the filter bed is too shallow to support tree roots. Shrubs shall have a minimum height of 30 inches.

Tree, shrub, and herbaceous plants. This is the traditional landscaping option for bioretention. It produces the most natural effect, and it is highly recommended for bioretention basin applications. The landscape goal is to simulate the structure and function of a native forest plant community.

Turf and tree. This option is a lower maintenance version of the tree-shrub-herbaceous option 4, where the mulch layer is replaced by turf cover. Trees are planted within larger mulched islands to prevent damage during mowing operations.

The choice of which planting template to use depends on the scale of bioretention, the context of the site in the urban environment, the filter depth, the desired landscape amenities, and the future owner's capability to maintain the landscape. In general, the vegetative goal is to cover up the filter surface with vegetation in a short amount of time. This means that the herbaceous layer is equally or more important than widely spaced trees and shrubs.

The following additional guidance is provided regarding developing an effective bioretention landscaping plan:

- Plants shall be selected based on a specified zone of hydric tolerance and must be capable of surviving both wet and dry conditions.
- "Hydrophytic" or "water tolerant" species shall be planted near the center, whereas upland species do better when planted near the edge.
- Woody vegetation shall not be located at points of inflow; trees shall not be planted directly above underdrains but shall be located closer to the perimeter.
- If trees are part of the planting plan, a tree density of approximately one tree to 580 - 900 sq ft (i.e., 24 - 30 ft on-center) is recommended.
- Shrubs and herbaceous vegetation shall generally be planted in clusters and at higher densities than trees (10 ft on-center and 1 to 1.5 ft on-center, respectively).
- Temporary or supplemental irrigation may be needed for the bioretention plantings in order for plant installers to provide a warranty regarding plant material survival.
- Supplemental irrigation by a rain tank system is also recommended. Designers shall also remember that planting holes for trees must be at least 4 ft deep to provide enough soil volume for the root structure of mature trees. This applies even if the remaining soil media layer is shallower than 4 ft.
- If trees are used, plant shade-tolerant ground covers within the drip line.
- Maintenance is an important consideration in selecting plant species. Plant selection differs if the area will be frequently mowed, pruned, and weeded, in contrast to a site which will receive minimum annual maintenance.

6.9 Bioretention Material Specifications

Table C-8 outlines the standard material specifications used to construct bioretention areas.

Table C-8. Bioretention material specifications.

Material	Specification	Notes
Filter Media Composition	Filter Media shall consist of material specified in Appendix C	The volume of filter media based on 110% of the plan volume, to account for settling or compaction.
Filter Media Testing	CEC greater than 10 meq/100g	---
Mulch Layer	Use mulch meeting requirements specified above.	Provide a 3- to 4-inch layer on the surface of the filter bed.
Alternative Surface Cover	Use river stone or pea gravel, coir, and jute matting.	Provide a 2- to 3- inch layer of cover to suppress weed growth.
Geotextile/Liner	Use a non-woven geotextile fabric with a flow rate of > 110 gal./min./sq. ft. (e.g., Geotex 351 or equivalent)	Apply only to the sides and above the underdrain. For potential high pollutant areas, use an appropriate liner on bottom.
Filter Layer	Lay a 2- to 4-inch layer of sand over a 2 in. layer of filter or choker stone (a finer stone intended to impede flow of sediment into voids in larger stone layer, typically #8 or #89 washed gravel), which is laid over the underdrain stone.	---
Stone Jacket for Underdrain and/or Storage Layer	1-in. stone shall be double-washed and clean and free of all fines (e.g., #57 stone).	12 inches for the underdrain; 12 to 18 inches for the stone storage layer, if needed
Underdrains, Cleanouts, and Observation Wells	Use 6 inch PVC or corrugated HDPE pipe with 3/8-in. perforations at 6 in. on center; position each pipe on a 1% or 2% slope located no more than 20 ft. from the next pipe. Inter-connect pipe network to ensure drainage.	Lay the perforated pipe under the length of the bioretention cell and install nonperforated pipe as needed to connect with the storm drain system. Install T's and Y's as needed, depending on the underdrain configuration. Extend cleanout pipes to the surface with vented caps at the T's and Y's.
Plant Materials	**Shrubs:** Minimum 3 gal or 18-24 inches ht at 4 ft o.c. (or 1plant/15 sq ft). **Grasses, sedges, perennials:** Size and max. spacing as called for in Tables C-9 and C-10 herein **Trees:** Minimum size 2 in. caliper, maximum spacing for Level 2 design of 1 tree/580 sq.ft. (NOTE: the 2 in. cal, minimum allows it to be counted toward landscape ordinance requirements)	Establish plant materials as specified in the landscaping plan and the recommended plant list.

7. Special Case Design Adaptations

7.1 Shallow Bedrock and Groundwater Connectivity

Many areas within Little Rock have shallow bedrock (less than 5 ft below ground surface, which can constrain the application of deeper bioretention areas (particularly Level 2 designs). In such settings, the following design adaptations may be helpful:

- A linear approach to bioretention, using multiple cells leading to the ditch system, helps conserve hydraulic head.
- The minimum depth of the filter bed may be 18 to 24 inches. It is useful to limit surface ponding to 6 to 9 inches and avoid the need for additional depth by establishing a turf cover rather than using mulch. The shallower media depth and the turf cover generally comply with the Water Quality Swale specification, and therefore will be credited with a slightly lower pollutant removal (See Water Quality Swales WQ-14).
- It is important to maintain at least a 0.5% slope in the underdrain to ensure positive drainage.
- The underdrain shall be tied into the ditch or conveyance system.

7.2 Steep Terrain

In steep terrain, land with a slope of up to 15% may drain to a bioretention area, as long as a two-cell design is used to dissipate erosive energy prior to filtering. The first cell, between the slope and the filter media, functions as a forebay to dissipate energy and settle any sediment that migrates downslope. Designers may also want to terrace a series of bioretention cells to manage runoff across or down a slope. The drop in slope between cells shall be limited to 1 foot and shall be armored with river stone or a suitable equivalent to resist erosion and piping.

8. Construction

8.1 Construction Sequence Construction

Stage Erosion and Sediment Controls. Small-scale bioretention areas should be fully protected from construction impacts by silt fence or construction fencing, particularly if they will rely on infiltration (i.e., have no underdrains.) Completed bioretention cells should remain outside the limit of disturbance during construction to prevent soil compaction by heavy equipment. Bioretention basin locations may be used as small sediment traps or basins during construction. However, these must be accompanied by notes and graphic details on the appropriate plan specifying that (1) the maximum excavation depth at the construction stage must be at least 1 foot above the completed installation, and (2) the completed bioretention facility must contain an underdrain. The plans and specifications must also show the proper procedures for converting the temporary sediment control practice to a permanent bioretention facility, including dewatering, cleanout, and stabilization.

8.2 Bioretention Installation

The following is a typical construction sequence to properly install a bioretention basin. The construction sequence for a rain garden is more simplified. These steps may be modified to reflect different bioretention applications or expected site conditions:

Step 1. Construction of the bioretention area may only begin after the entire contributing drainage area has been stabilized with vegetation. THIS IS THE MOST IMPORTANT FACTOR DETERMINING THE SUCCESS OR FAILURE OF THE BIORETENTION AREA. BIORETENTION AREAS WILL FAIL IF SEDIMENT IS ALLOWED TO FLOW INTO THEM. It may be necessary to block certain curb or other inlets while the bioretention area is being constructed. The proposed site shall be checked for existing utilities prior to any excavation.

Step 2. The designer and the installer shall have a preconstruction meeting, checking the boundaries of the contributing drainage area and the actual inlet elevations to ensure they conform to original design. Since other contractors may be responsible for constructing portions of the site, it is quite common to find subtle differences in site grading, drainage and paving elevations that can produce hydraulically important differences for the proposed bioretention area. The designer should clearly communicate, in writing, any project changes agreed upon during the preconstruction meeting to the installer and to the City as required, together with an explanation of the impact of such changes.

Step 3. Temporary erosion and sedimentation controls are needed during construction of the bioretention area to divert stormwater away from the bioretention area until it is completed. Special protection measures such as erosion control fabrics may be needed to protect vulnerable side slopes from erosion during the construction process. At a minimum, these measures shall comply with current ADEQ standards.

Step 4. Any pre-treatment cells shall be excavated first and then sealed to trap sediments.

Step 5. Excavators or backhoes shall work from the sides to excavate the bioretention area to its appropriate design depth and dimensions. Excavating equipment should have scoops with adequate reach and should not be inside the footprint of the bioretention area. Contractors may use a cell construction approach in larger bioretention basins, whereby the basin is split into 500 to 1,000 sq ft temporary cells with a 10-15 ft earth bridge in between, so that cells may be excavated from the side.

Step 6. It may be necessary to rip the bottom soils to a depth of 6 to 12 inches to promote greater infiltration.

Step 7. Place geotextile fabric on the sides of the bioretention area with a minimum 6-inch overlap on the sides. If a stone storage layer will be used, place the appropriate depth of stone on the bottom, install the perforated underdrain pipe, pack stone to 3 inches above the underdrain pipe, and add approximately 3 inches of choker stone/pea gravel as a filter between the underdrain and the soil media layer. If no stone storage layer is used, start with 6 inches of stone on the bottom, and proceed with the layering as described herein.

Step 8. Deliver or prepare the soil media and store it on an adjacent impervious area or plastic sheeting. Apply the media in 12-inch lifts without additional compaction until the desired top elevation of the bioretention area is achieved. Wait a few days to check for settlement, and add additional media, as needed, to achieve the design elevation. Where outlet structures are used, approved structural fill must be placed around the structure in accordance with design details. Structural earthen fill shall be used and compacted to a minimum of 95% Standard Proctor and +/-2 % of optimum moisture content within 2 ft of the structure. If piping or clogging occurs due to sediment transport, adjacent fill must be removed and replaced in the piping or clogged areas and the feature properly reconstructed to address the problem.

Step 9. Prepare planting holes for any trees and shrubs, install the vegetation, and water accordingly. Install any temporary irrigation.

Step 10. Place the surface cover in both cells (mulch, river stone or turf), depending on the design. If coir or jute matting will be used in lieu of mulch, the matting will need to be installed prior to planting (Step 9), and holes or slits will have to be cut in the matting to install the plants.

Step 11. Install the plant materials as shown in the landscaping plan and as per City of Little Rock Standards, and water them during weeks of no rain for the first two months. This should be continued until the plants are fully established. PLANT SURVIVAL AND MAINTENANCE IS CRITICAL TO PROPER PERFORMANCE OF THE BIORETENTION SYSTEM.

Step 12. Conduct the final construction inspection (see Section 9.2). Then log the GPS coordinates for each bioretention facility and submit them to the City Planning/Engineering Department.

8.3 Construction Inspection

Construction inspections shall be performed by the design engineer and the landscape architect to ensure that the bioretention system is installed in accordance with the plans and specifications. Items to inspect during construction include but are not limited to the following:

- Pretreatment systems (if applicable).
- Subgrade (check to make sure subgrade has not been compacted).
- Filter fabric installation.
- Underdrain system (if applicable).

- Gravel media.
- Engineered soil media.
- Plant installation.
- Inlet and Outlet elevations.
- Overflow structures.

9. Maintenance

9.1 Maintenance

The requirements for maintenance include the development of a Long-Term Maintenance Plan (LTMP) by the design engineer. The LTMP contains a description of the stormwater system components and information on the inspection and maintenance activities.

9.2 First Year Maintenance Operations

Successful establishment of bioretention areas requires that the following tasks be undertaken in the first year following installation:

- Initial inspections. For the first 6 months following construction, the site shall be inspected at least twice after storm events that exceed 0.5 inch of rainfall.
- Spot Reseeding. Inspectors shall look for bare or eroding areas in the contributing drainage area or around the bioretention area, and make sure they are immediately stabilized with grass cover.
- Fertilization. One-time, spot fertilization may be needed for initial plantings. A better choice may be a slow-release fertilizer that can be incorporated into the planting medium.
- Watering. Watering is needed once a week during the first 2 months, and then as needed during first growing season (April-October), depending on rainfall.
- Remove invasives and replace dead plants. Since up to 10% of the plant stock may die off in the first year, construction contracts shall include a care and replacement warranty to ensure that vegetation is properly established and survives during the first growing season following construction. The typical thresholds below which replacement is required are 85% survival of plant material and 100% survival of trees. Long-term survival shall be in accordance with City of Little Rock Tree and Landscape requirements.

9.3 Maintenance Inspections

It is highly recommended that a spring maintenance inspection and cleanup be conducted at each bioretention area. The following is a list of some of the key maintenance problems to look for:

- Check to see if 75% to 90% cover (mulch plus vegetative cover) has been achieved in the bed and measure the depth of the remaining mulch. Remove invasives.
- Check for sediment buildup at curb cuts, gravel diaphragms or pavement edges that prevents flow from getting into the bed and check for other signs of bypassing.
- Check for any winter- or salt-killed vegetation and replace it with hardier species.
- Note presence of accumulated sand, sediment and trash in the pre-treatment cell or filter beds and remove it.
- Inspect bioretention side slopes and grass filter strips for evidence of any rill or gully erosion and repair it.
- Check the bioretention bed for evidence of mulch flotation, excessive ponding, dead plants, or concentrated flows, and take appropriate remedial action.
- Check inflow points for clogging and remove any sediment.
- Look for any bare soil or sediment sources in the contributing drainage area and stabilize them immediately.
- Check for clogged or slow-draining soil media, a crust formed on the top layer, inappropriate soil media, or other causes of insufficient filtering time, and restore proper filtration characteristics.

9.4 Routine and Non-Routine Maintenance Tasks

Maintenance of bioretention areas shall be integrated into routine landscape maintenance tasks. If landscaping contractors will be expected to perform maintenance, their contracts shall contain specifics on unique bioretention landscaping needs, such as maintaining elevation differences needed for ponding, proper mulching, sediment and trash removal, and limited use of fertilizers and pesticides. A customized maintenance schedule must be prepared for each bioretention facility, since the maintenance tasks will differ depending on the scale of bioretention, the landscaping template chosen, and the type of surface cover. A generalized summary of common maintenance tasks and their frequency is provided in Table C-9.

The most common non-routine maintenance problem involves standing water. If water remains on the surface for more than 48 hours after a storm, adjustments to the grading may be needed or underdrain repairs may be needed. The surface of the filter bed shall also be checked for accumulated sediment or a fine crust that builds up after the first several storm events. There are several methods that can be used to rehabilitate the filter (try the easiest things first, as listed below):

- Open the underdrain observation well or cleanout and pour in water to verify that the underdrains are functioning and not clogged or otherwise in need of repair. The purpose of this check is to see if there is standing water all the way down through the soil. If there is standing water on top, but not in the underdrain, then there is a clogged soil layer. If the underdrain and standpipe indicate standing water, then the underdrain must be clogged and will need to be snaked.
- Remove accumulated sediment and till 2 to 3 inches of sand into the upper 8 to 12 inches of soil.
- Install sand wicks from 3 inches below the surface to the underdrain layer. This reduces the average concentration of fines in the media bed and promotes quicker drawdown times. Sand wicks can be installed by excavating or auguring (using a tree auger or similar tool) down to the gravel storage zone to create vertical columns which are then filled with a clean open-graded coarse sand material (ASTM C-33 concrete sand or similar approved sand mix for bioretention media). A sufficient number of wick drains of sufficient dimension shall be installed to meet the design dewatering time for the facility.
- Remove and replace some or all the soil media and plants.

Table C-9. Suggested Annual Bioretention Maintenance Activities.

Maintenance Materials	Frequency
Mowing of grass filter strips and bioretention turf cover	At least 4 times a year
Spot weeding, erosion repair, trash removal, and mulch raking	Twice during growing season
Add reinforcement planting to maintain the desired vegetation density	As needed
Remove invasive plants using recommended control methods	As needed
Stabilize the contributing drainage area to prevent erosion	As needed
Spring inspection and cleanup	Annually
Supplement mulch to maintain a 3-inch minimum layer	Annually
Prune trees and shrubs	Annually
Remove sediment in pre-treatment cells and inflow point	Once every 2 to 3 years
Replace the mulch layer	Every 3 years

10. As-Built Requirements

After the bioretention area has been constructed, the developer must provide the city engineers an as-built certification of the bioretention area prepared by a registered Professional Engineer. The as-built certification verifies that the BMP was installed as designed and approved. The following components are vital to ensure that the bioretention area works properly and they must be addressed in the as-built certification:

1. Pretreatment, such as a grass filter strip, for coarser sediments must be provided to prevent premature clogging of the system. Design guidance for grass filter strips used as pretreatment is provided in herein.
2. Surrounding drainage areas must be stabilized to prevent sediment from clogging the filter media.
3. Correct ponding depths and infiltration rates must be maintained to prevent killing vegetation.
4. The landscape plan must be provided.

A mechanism for overflow for storm events exceeding design capacity must be provided.

11. Right of Way Constraints

Bioretention can be used in the right of way and is a preferred practice for constrained right of ways when designed as a series of individual on-line or off-line cells. In these situations, the final design closely resembles that of water quality swales. Stormwater can be conveyed to the bioretention area by sheet flow, curb cuts, or grass channels. The base of the bioretention feature shall be positioned on a 2H:1V slope (or gentler) from edge of pavement (or curb, if applicable). If installation is within 10 ft of pavement subgrade, also refer to Urban Bioretention Practice WQ-05 for additional details.

Table C-10. Potential Areas of High Pollutant Loading.

Potential Area of High Pollutant Loading	Restricted Infiltration	No Infiltration[1]
Facilities w/NPDES Industrial permits	■	■
Public works yard		✓
Railroads/ equipment storage		✓
Auto and metal recyclers/scrap yards	✓	✓
Petroleum storage facilities		✓
Highway maintenance facilities	✓	
Wastewater, solid waste, and composting facilities	✓	
Industrial machinery and equipment	✓	
Trucks and trailers	✓	
Airfields	✓	
Aircraft maintenance areas		✓
Fleet storage areas	✓	
Parking lots (40 or more parking spaces)	✓	
Gas stations		✓
Highways (2500 ADT)	✓	
Construction business (paving, heavy equipment storage and maintenance)	✓	
Retail/wholesale vehicle/ equipment dealers	✓	
Convenience stores/fast food restaurants	✓	
Vehicle maintenance facilities		✓
Car washes		✓
Nurseries and garden centers	✓	
Golf courses	✓	

Note: For a full list of potential stormwater hotspots. Consult Schueler et al. (2004).

■ depends on facility.

✓ criterion applies.

1. For some facilities, infiltration practices will be permitted for certain areas such as employee parking and roof drainage.

12. Community and Environmental Concerns

Potential areas of high pollutant loading are operations or activities that are known to produce higher concentrations of stormwater pollutants and/or have a greater risk for spills, leaks or illicit discharges. Table C-10 presents a list of potential land uses or operations that may be designated as such areas. It should be noted that the actual area may only occupy a portion of the entire proposed use, and that some "clean" areas (such as rooftops or landscaping) can be diverted away to another infiltration or runoff reduction practice. Development proposals should be carefully reviewed to determine if any future operation, on all or part of the site, will be designated as an area of high pollutant loading. Based on this designation, infiltration may be restricted or prohibited within such areas. For the restricted category, use of infiltration may be permitted for employee parking and rooftop drainage areas, and if low permeability linings are used with an underdrain.

13. References

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WQ-05 Urban Bioretention

1. Description

Urban bioretention practices are similar in function to regular bioretention practices (WQ-04) except they are adapted to fit into “containers” within urban landscapes. Typically, urban bioretention is installed within an urban streetscape or city street Right of Way (ROW), urban landscaping beds, tree planters, and plazas. Urban bioretention is not intended for large commercial areas, nor shall it be used to treat small sub-areas of a large drainage area such as a parking lot. Rather, urban bioretention is intended to be incorporated into small, fragmented drainage areas such as shopping or pedestrian plazas within a larger urban development. Urban Bioretention within the ROW can only be used to treat water that falls in the ROW.

Urban bioretention features hard edges, often with vertical concrete sides, as contrasted with the gentler earthen slopes of regular bioretention. If these practices are outside of the ROW, they may be open bottomed, to allow some infiltration of runoff into the sub-grade, but they generally are served by an underdrain.

2. Pollutant Removal Capabilities

The following design pollutant removal rates are conservative average pollutant reduction percentages for design purposes derived from sampling data, modeling, and professional judgment.

- Total Suspended Solids – 80%
- Total Phosphorus – 60%
- Total Nitrogen – 50%
- Fecal Coliform – insufficient data
- Heavy Metals – 80%

3. Design Considerations

Each urban bioretention variant is planted with a mix of trees, shrubs, and grasses as appropriate for its size and landscaping context.

Stormwater planters (also known as vegetative box filters or foundation planters) take advantage of limited space available for stormwater treatment by placing a soil filter in a container located above ground or at grade in landscaping areas between buildings and roadways, provided with waterproof liner protection (Figure C-28). The small footprint of foundation planters is typically contained in a precast or cast-in-place concrete vault. Other materials may include molded polypropylene cells and precast modular block systems. Stormwater planters must be outside the ROW if they are runoff from areas outside of the ROW as additional surface drainage shall not be directed into the ROW.



Figure C-28. Stormwater planters (Source: City of Portland, OR).

Green Street swales and planters are installed in the sidewalk zone near the street where urban street trees are normally installed. The soil volume for the tree pit is increased and used as a stormwater storage area (Figure C-29). Treatment is increased by using a series of connected tree planting areas together in a row. The surface of the enlarged planting area may be mulch, overlain with grates or conventional pavement (subject to approval of the City Arborist). Large and shared rooting space and a reliable water supply increase the growth and survival rates in this otherwise harsh planting environment.



Figure C-29. Deaderick Street tree planters (Source: City of Nashville, TN).

4. Typical Details

Figures C-30 through C-36 provide some typical details for several bioretention configurations. Planting plans shall be prepared by a licensed Landscape Architect.

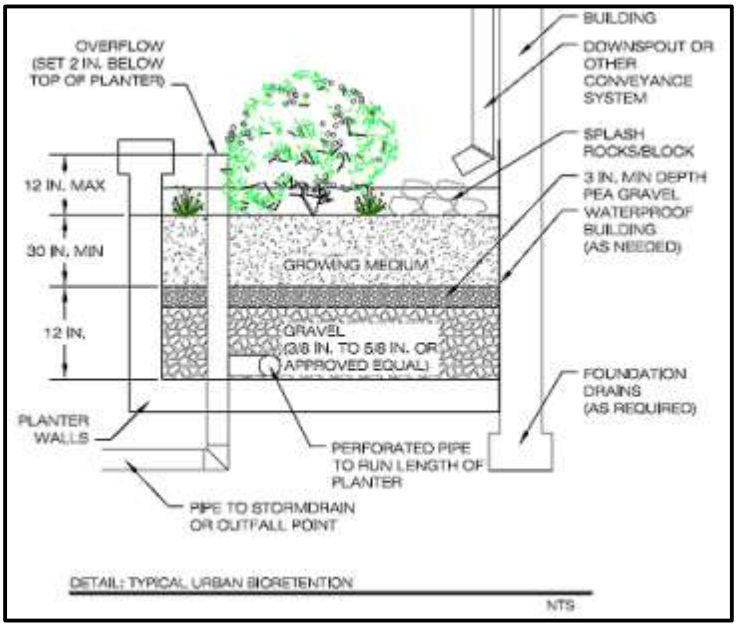


Figure C-30. Stormwater planter cross-section (Source: VADCR, 2010).

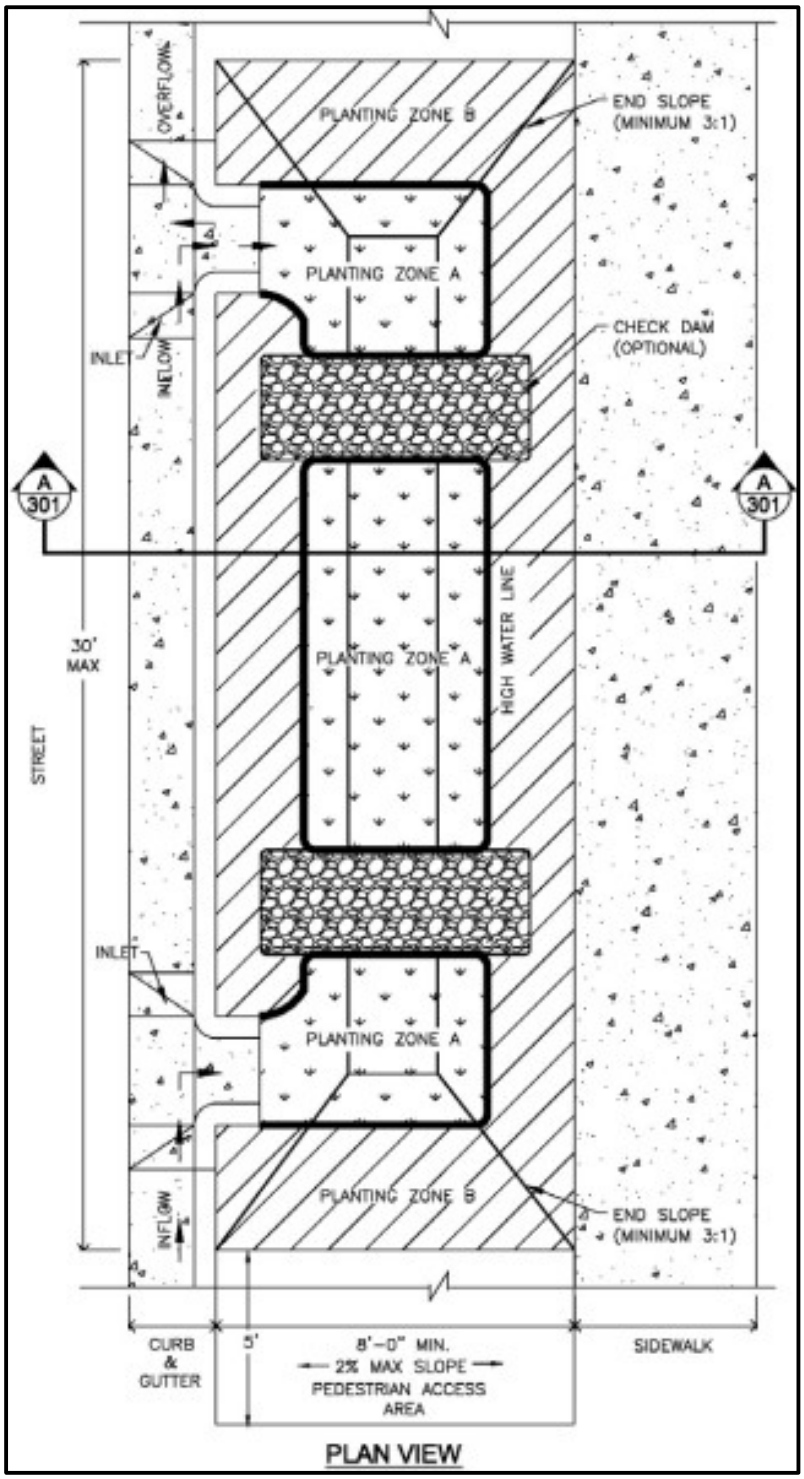


Figure C-31. Green streets swale plan view (Source: Portland, 2011).

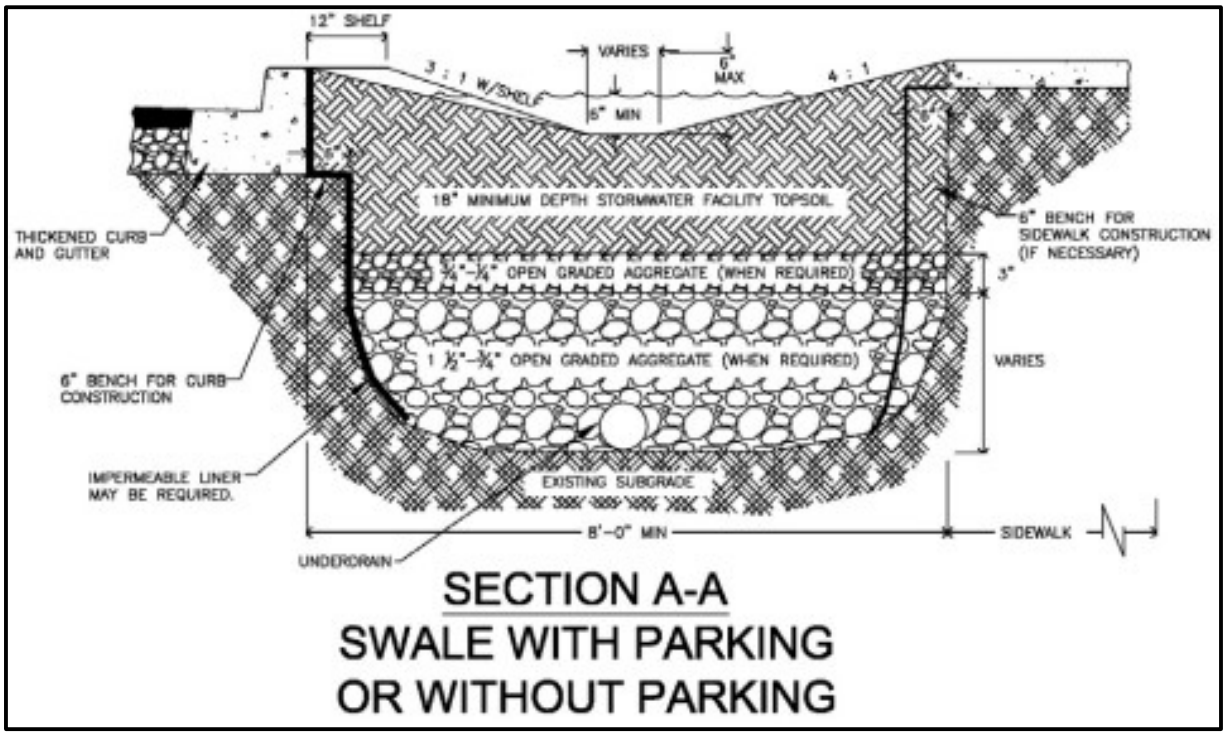


Figure C-32. Green streets swale section view (Source: Portland, 2011).

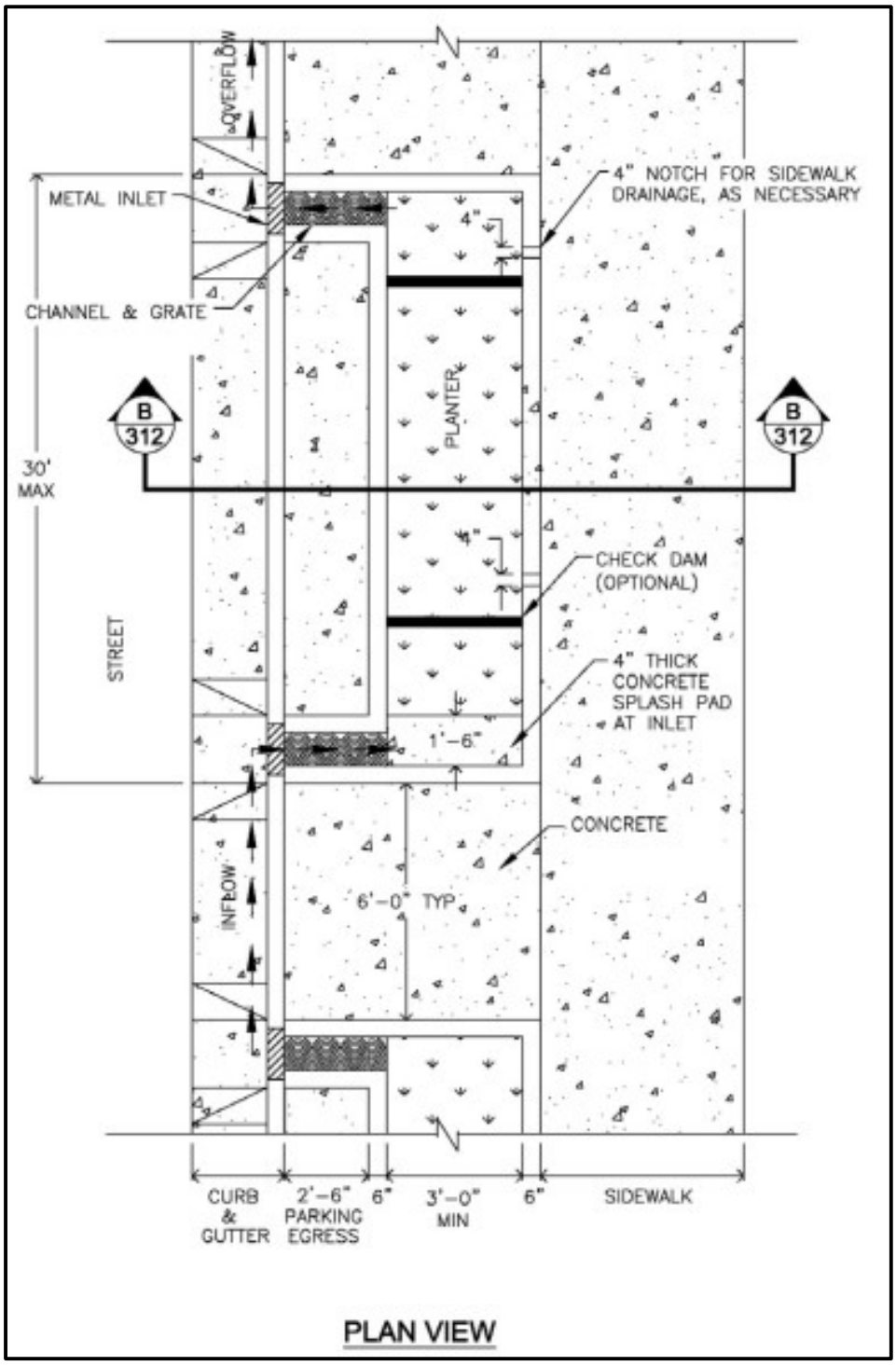


Figure C-33. Green streets planter plan view with parking (Source: Portland, 2011).

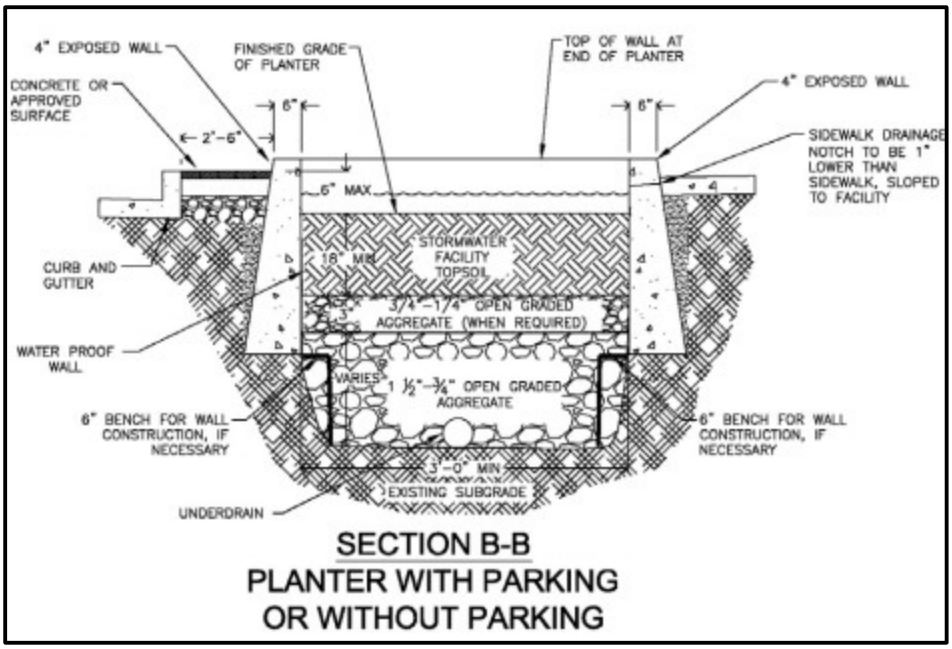


Figure C-34. Green streets planter section view with or without parking (Source: Portland, 2011).

5. Physical Feasibility and Design Applications

Table C-11. Urban Bioretention Design Criteria.

Level 1 Design Only (RR: 60)
Sizing (Refer to Section 6.1): Surface Area (sq ft) = WQv/Storage Depth[1] = [(1.05 inch) (Rv)(A)/12 – the volume reduced by an upstream BMP]/Storage Depth[1]
Underdrain = Slotted corrugated HDPE with clean-outs (Refer to WQ-04, Section 6.7)
Maximum Drainage Area = 2,500 sq ft
Maximum Ponding Depth = 6 inches
Filter media depth minimum = 30 inches; maximum = 48 inches;
Media and Surface Cover (Refer to WQ-04, Section 6.6)
Sub-soil testing (Refer to WQ-04, Section 6.2)
Inflow = sheet flow, curb cuts, trench drains, roof drains, concentrated flow, or equivalent
Building setbacks (Refer to Section 5)
Long-term maintenance plan (Refer to WQ-04, Section 9)

1. Storage depth is based on the Void Ratios (V_r) of the soil media and gravel layers and their respective thicknesses, plus the surface ponding depth. Compare as described in Section 6.1.

In general, urban bioretention has the same constraints as regular bioretention (See WQ-04), along with a few additional constraints as noted below:

Contributing Drainage Area. Urban bioretention is classified as a micro-bioretention practice and is therefore limited to 2,500 sq ft of drainage area to each unit. However, this is considered a general rule; larger drainage areas may be allowed with sufficient flow controls and other mechanisms to ensure proper function, safety, and community acceptance. The drainage areas in these urban settings are typically considered to be 100% impervious. While multiple units can be installed to maximize the treatment area in ultra-urban watersheds, urban bioretention is not intended to be used as treatment for large impervious areas (such as parking lots).

Adequate Drainage. Urban bioretention practice elevations must allow the untreated stormwater runoff to be discharged at the surface of the filter bed and ultimately connect to the local storm drain system.

Available Hydraulic Head. In general, 3 ft of elevation difference is needed between the downstream storm drain invert and finished grade of the urban bioretention practice. A positive hydraulic differential must be maintained through all design storm and runoff events. This is generally not a constraint, due to the standard depth of most storm drain systems. Utilities, Infrastructure, and Setbacks. Urban bioretention practices frequently compete for space with a variety of utilities. Since they are often located parallel to the road right-of-way, care shall be taken to accommodate utility-specific horizontal and vertical setbacks. However, conflicts with water and sewer laterals (e.g., house connections) may be unavoidable, and the construction sequence must be altered as necessary to avoid impacts to existing service. Tree Box Filter details may be found in the University of Arkansas Community Design Center Low Impact Development Manual. Adjacent to buildings, an impermeable liner and an underdrain shall be used and properly installed. Otherwise, the urban bioretention practice shall be positioned a minimum of 10 ft from the building. For ground-level installations, maximum pool elevation shall be at least 0.5 ft below the lowest adjacent grade of the building. Designers shall also plan future tree canopy heights achieved in conjunction with urban bioretention practices to not conflict with existing overhead telephone, cable communications and power lines.

Minimizing External Impacts. Because urban bioretention practices are installed in highly urban settings, individual units may be subject to higher public visibility, greater trash loads, pedestrian use traffic, vandalism, and even vehicular loads. These practices shall be designed in ways that prevent, or at least minimize, such impacts. In addition, designers shall clearly recognize the need to perform frequent landscaping maintenance to remove trash, check for clogging, and

maintain vigorous vegetation. The urban landscape context may feature naturalized landscaping or a more formal design. When urban bioretention is used in sidewalk areas of high foot traffic, designers shall not impede pedestrian movement or create a safety hazard and maintain the Americans with Disabilities Act (ADA) required path of travel. Designers may also install low fences (such as a low garden fence), grates or other measures to prevent damage from pedestrian short-cutting across the practices.

6. Design Criteria

Urban bioretention practices are similar in function to regular bioretention practices except they are adapted to fit into "containers" within urban landscapes. Therefore, special sizing accommodations are made to allow these practices to fit in very constrained areas where other surface practices may not be feasible.

6.1 Sizing of Urban Bioretention

The required surface area of the urban bioretention filter is calculated by dividing the water quality volume by the Equivalent Storage Depth (Equation C-4 below), in the same manner as it is calculated for traditional bioretention. The equivalent storage depth is computed as the depth of media, gravel, or surface ponding (in feet) multiplied by the accepted ratio. The maximum allowable Void Ratios (V_r) are shown in Table C-12.

Table C-12. Maximum Allowable Void Ratios.

Type	V_r
Bioretention Soil Media (See Section 6.6 of WQ-04)	0.40 (sandy loam, loamy sand, or loam)
Gravel	0.35
Surface Storage	1.0

The equivalent storage depth for an urban bioretention facility with, for example, a 6-inch surface ponding depth, a 30-inch soil media depth, and a 12-inch gravel layer is therefore computed as:

Weighted Storage Depth = D_W = Vr1(D1) + Vr2(D2) + ⋯ **Eq. C-4**

(1.0 x 0.5 ft) + (0.4 x 2.5 ft) + (0.35 x 1.0 ft) = 1.85 ft

Where: V_{r1} and D_1 are for the first layer, etc.

The Surface Area (SA) corresponding to this example may be computed as:

SA (sq ft) = WQv (cu ft) / DW **Eq. C-5**

Where: SA = Minimum surface area of the urban bioretention facility (sq ft)

D_W = ESD (Equivalent Storage Depth)

WQv = Required Water Quality Volume (in cu ft)

WQv = [(1.05 inch) (Rv) (A)] / 12 **Eq. C-6**

Where: WQv = Water Quality Volume (cu ft)

A = the contributing drainage area (sq ft)

Rv = the volumetric runoff coefficient

Equations C-4 and C-6 should be modified with the addition of any surface or subsurface storage components (e.g., additional area of surface ponding, subsurface storage chambers, etc.).

6.2 General Design Criteria for Urban Bioretention

Design of urban bioretention shall follow the general guidance presented in this design specification. The actual geometric design of urban bioretention is usually dictated by other landscape elements such as buildings, sidewalk widths, utility corridors, retaining walls, etc. Designers can divert fractions of the runoff volume from small impervious surfaces into micro-bioretention units that are integrated with the overall landscape design. Inlets and outlets should be located as far apart as possible. The following is additional design guidance that applies to all variations of urban bioretention:

- The ground surface of the micro-bioretention cell shall slope a minimum of 1% towards the outlet unless a stormwater planter is used.
- The soil media depth shall be a minimum of 30 inches.
- If large trees and shrubs are to be installed, soil media thickness shall be a minimum of 4 ft.
- All urban bioretention practices shall be designed to fully drain within 24 hours.
- Any grates used above urban bioretention areas must be removable to allow maintenance access, be ADA compliant and, if for tree planters, be approved by the Urban Forester.
- The inlet(s) to urban bioretention shall be stabilized using coarse aggregate stone (cobblestone), splash block, river stone or other acceptable energy dissipation measures. The following forms of inlet stabilization are recommended:
    - Stone energy dissipators.
    - Sheet flow over a depressed curb with a 3-inch drop.
    - Curb cuts allowing runoff into the bioretention area.
    - Covered drains that convey flows under sidewalks from the curb or from downspouts (if the bioretention area is outside of the ROW).
    - Grates or trench drains that capture runoff from the sidewalk or plaza area.

Pre-treatment options overlap with those of regular bioretention practices. However, the materials used may be chosen based on their aesthetic qualities in addition to their functional properties. For example, river rock may be used in lieu of rip rap. Other pretreatment options may include one of the following:

- A trash rack between the pre-treatment cell and the main filter bed. This will allow trash to be collected from one location.
- A trash rack across curb cuts. While this trash rack may clog occasionally, it keeps trash in the gutter, where it can be picked up by street sweeping equipment.
- A pre-treatment area above ground, or with a manhole or grate directly over the pre-treatment area.

Overflows can either be diverted from entering the bioretention cell or dealt with via an overflow inlet. Optional methods include the following:

- Size curb openings to capture only the water quality volume and bypass higher flows through the existing gutter.
- Use landscaping type inlets or standpipes with trash guards as overflow devices.
- Use a pre-treatment chamber with a weir design that limits flow to the filter bed area.

6.3 Specific Design Issues for Stormwater Planters

Since stormwater planters are often located near building foundations, waterproofing by using a watertight concrete shell or an impermeable liner that is designed and constructed to prevent seepage is required.

6.4 Specific Design Issues for Green Streets Swales and Planters

- The bottom of the soil media must be a minimum of 4 inches below the root ball of plants to be installed, 12 inches for shrubs and 24 inches for trees.

- Green street designs sometimes cover portions of the filter media with pervious pavers (if outside the ROW) or cantilevered sidewalks. In these situations, it is important that the filter media is connected beneath the surface so that stormwater and tree roots can share this space.
- Installing a tree pit grate over filter bed media is one possible solution to prevent pedestrian traffic and trash accumulation.
- Low, wrought iron fences can help restrict pedestrian traffic across the tree pit bed and serve as a protective barrier if there is a drop-off from the pavement to the micro-bioretention cell.
- A removable grate capable of supporting typical H-20 loads may be used to allow the tree to grow through it. Tree grates shall be approved by the City Arborist.
- Each tree needs a minimum of 120 sq ft of shared root space.
- Proprietary tree pit devices are acceptable, provided they conform to this specification, comply with the ordinance, and are approved by the Urban Forester.

6.5 Planting and Landscaping Considerations

The degree of landscape maintenance that can be provided will determine some of the planting choices for urban bioretention areas. The planting cells can be formal gardens or naturalized landscapes but shall comply with existing city ordinances. Landscaping in the ROW shall be designed to limit visual obstructions for pedestrian and vehicular traffic.

In areas where less maintenance will be provided and where trash accumulation in shrubbery or herbaceous plants is a concern, consider a “turf and trees” landscaping model. Spaces for herbaceous flowering plants can be included. This may be attractive at a community entrance location.

Native trees or shrubs are preferred for urban bioretention areas, although some ornamental species may be used. As with regular bioretention, selected perennials, shrubs, and trees must be tolerant of drought, and inundation. The landscape designer shall also take into account that de-icing materials may accumulate in the bioretention areas in winter and could kill vegetation. Additionally, tree species selected shall be those that are known to survive well in the compacted soils and polluted air and water of an urban landscape. For additional information regarding recommended planting densities see WQ-04 Bioretention, Section 6.8: Bioretention Planting Plans for shrubs.

7. Material Specifications

Please consult WQ-04, Table C-8 for the typical materials needed for filter media, stone, mulch, and other bioretention components. In urban planters, pea gravel or river stone may be a more appropriate and attractive mulch than shredded hardwood.

The unique components for urban bioretention may include the inlet control device, a concrete box or other containing shell, impermeable liner, protective grates, and an underdrain that daylights to another stormwater practice or connects to the storm drain system. The underdrain shall:

- Consist of slotted pipe greater than or equal to 4 inches in diameter, placed in a layer of washed (less than 1% passing a #200 sieve) crushed stone.
- Have a minimum of 3 inches of gravel laid above and below the pipe. Be laid at a minimum slope of 0.5 %.
- Extend the length of the box filter from one wall to within 6 inches of the opposite wall and may be either centered in the box or offset to one side.
- Be separated from the soil media by non-woven, geotextile fabric or a 2-to-3-inch filter layer of 1/8 to 3/8-inch pea gravel.

8. Construction

The construction sequence and inspection requirements for urban bioretention are generally the same as other bioretention practices. Consult the construction sequence and inspection guidance provided in WQ-04. In cases where urban bioretention is constructed in the road or ROW, the construction sequence may need to be adjusted to account for traffic control, pedestrian access, and utility notification.

Additionally, urban bioretention areas shall only be constructed after the drainage area to the facility is completely stabilized and the inlet areas completely protected from construction traffic. The specified growth media shall be placed and spread by hand, to avoid compaction and maintain the porosity of the media. The media shall be placed in 12-inch lifts with no machinery allowed directly on the media during or after construction. The media shall be overfilled above the proposed surface elevation, as needed, to allow for natural settling. Lifts may be lightly watered to encourage settling. After the final lift is placed, the media shall be raked (to level it), saturated, and allowed to settle for at least one week prior to installation of plant materials.

9. Maintenance

Routine operation and maintenance are essential to gain public acceptance of highly visible urban bioretention areas. Weeding, pruning, the removal and replacement of dead vegetation and trash removal shall be done as needed to maintain the aesthetics necessary for community acceptance. During drought conditions, it may be necessary to water the plants, as would be necessary for any landscaped area. The requirements for maintenance include the development of a Long-Term Maintenance Plan (LTMP) by the design engineer. The LTMP contains a description of the stormwater system components and information on the inspection and maintenance activities.

To ensure proper performance, installers shall check that stormwater infiltrates properly into the soil within 24 hours after a storm. If excessive surface ponding is observed, corrective measures include inspection for soil compaction and underdrain clogging. Consult the maintenance guidance outlined in WQ-04.

10. As-Builts

After urban bioretention has been constructed, the developer must provide the city engineer with an as-built certification prepared by a registered Professional Engineer. The as-built certification verifies that the BMP was installed as designed and approved. The following components are vital to ensure that the bioretention area works properly and they must be addressed in the as-built certification:

1. The proper media and gravel depths were installed per plan. Photographs taken during phases of construction shall be included to demonstrate.
2. Surrounding drainage areas must be stabilized to prevent sediment from clogging the filter media.
3. Correct ponding depths and infiltration rates must be maintained to prevent killing vegetation.
4. The landscape plan must be provided.

11. Right of Way Considerations

Green Street swales and planters are applicable along roads. They can be used along curbside in urban areas with stormwater being conveyed by sheet flow or curb cuts. Green Street swales and planters can also be designed as a series of cells running parallel to roadway. Successful designs in other locations have also featured an impermeable membrane separating the road subgrade from the bioretention feature. Please consult WQ-04, Table C-8 for material slopes, types, and sizing. Connections to storm drain shall be consistent with connection requirements in the City of Little Rock Drainage Criteria Manual.

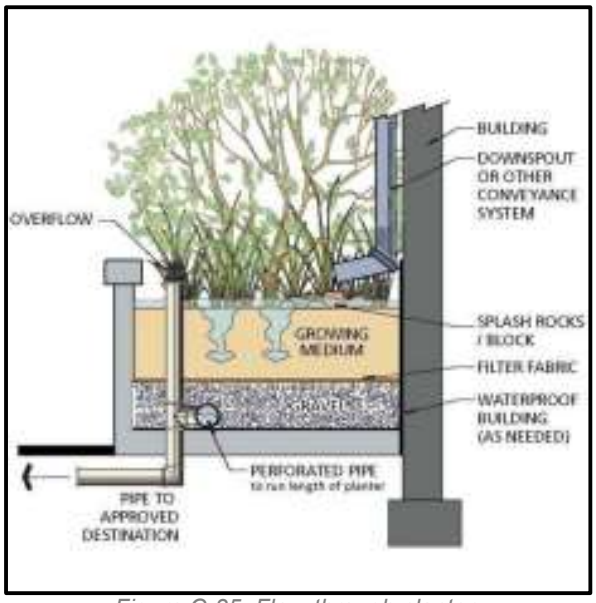


Figure C-35. Flow-through planter.

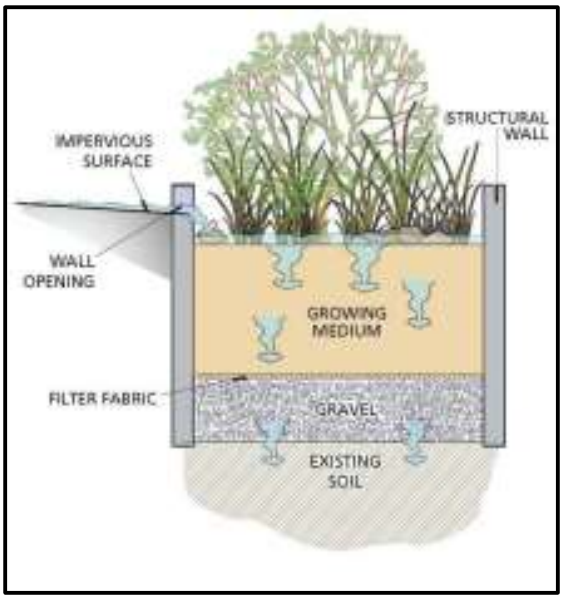


Figure C-36. Infiltration planter.



Figure C-37. Portland State University Street planters.



Figure C-38. Deaderick Street planters.

12. References

Center for Watershed Protection, 2006, *Urban Watershed Forestry Manual. Part 2: Conserving and Planting Trees at Development Sites.* Ellicott City, MD.

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WQ-06 Sand Filters

1. Description

Sand filters (also referred to as filtration basins) are structural stormwater controls that capture and temporarily store stormwater runoff and pass it through a filter bed of sand. Most sand filter systems consist of two-chamber structures. The first chamber is a sediment forebay or sedimentation chamber, which removes floatables and heavy sediments. The second is the filtration chamber, which removes additional pollutants by filtering the runoff through a sand bed. The filtered runoff is typically collected and returned to the conveyance system, though it can also be partially or fully exfiltrated into the surrounding soil in areas with porous soils.

Because they have few site constraints beside head requirements, sand filters can be used on development sites where the use of other structural controls may be precluded. However, sand filter systems can be relatively expensive to construct and install.

There are two primary sand filter system designs, the surface sand filter, and the perimeter sand filter. Below are descriptions of these filter systems:

- **Surface Sand Filter** – The surface sand filter is a ground-level open air structure that consists of a pretreatment sediment forebay and a filter bed chamber. This system can treat drainage areas up to 10 acres in size and is typically located off-line. Surface sand filters can be designed as an excavation with earthen embankments or as a concrete or block structure.
- **Perimeter Sand Filter** – The perimeter sand filter is an enclosed filter system typically constructed just below grade in a vault along the edge of an impervious area such as a parking lot. The system consists of a sedimentation chamber and a sand bed filter. Runoff flows into the structure through a series of inlet grates located along the top of the control.

A third design variant, the underground sand filter, is intended primarily for extremely space limited and high-density areas and is thus considered a limited application structural control.



Figure C-39. Sand Filter Examples.

2. Stormwater Management Suitability

Sand filter systems are designed primarily as off-line systems for stormwater quality (i.e., the removal of stormwater pollutants) and will typically need to be used in conjunction with another structural control to provide downstream channel protection, overbank flood protection, and extreme flood protection, if required. However, under certain circumstances, filters can provide limited runoff quantity control, particularly for smaller storm events.

Water Quality

In sand filter systems, stormwater pollutants are removed through a combination of gravitational settling, filtration, and adsorption. The filtration process effectively removes suspended solids and particulates, biochemical oxygen demand (BOD), fecal coliform bacteria, and other pollutants. Surface sand filters with a grass cover have additional opportunities for bacterial decomposition as well as vegetation uptake of pollutants, particularly nutrients.

Downstream Flood Protection

Another structural control must be used in conjunction with a sand filter system to reduce the post-development peak flow of the design storm events to pre-development levels (detention). Sand filter facilities must provide flow diversion and/or be designed to safely pass extreme storm flows (100-year storm event) and protect the filter bed and facility.

3. Pollutant Removal Capabilities

Both the surface and perimeter sand filters are presumed to be able to remove 80% of the total suspended solids load in typical urban post-development runoff when sized, designed, constructed, and maintained in accordance with the recommended specifications. Undersized or poorly designed sand filters can reduce TSS removal performance.

The following design pollutant removal rates are conservative average pollutant reduction percentages for design purposes derived from sampling data, modeling, and professional judgment. In a situation where a removal rate is not deemed sufficient, additional controls may be put in place at the given site in a series or “treatment train” approach.

- Total Suspended Solids – 80%
- Total Phosphorus – 50%
- Total Nitrogen – 25%
- Fecal Coliform – 40%
- Heavy Metals – 50%

For additional information and data on pollutant removal capabilities for sand filters, see the National Pollutant Removal Performance Database (3rd Edition) available at www.cwp.org and the National Stormwater Best Management Practices (BMP) Database at www.bmpdatabase.org

4. Typical Details

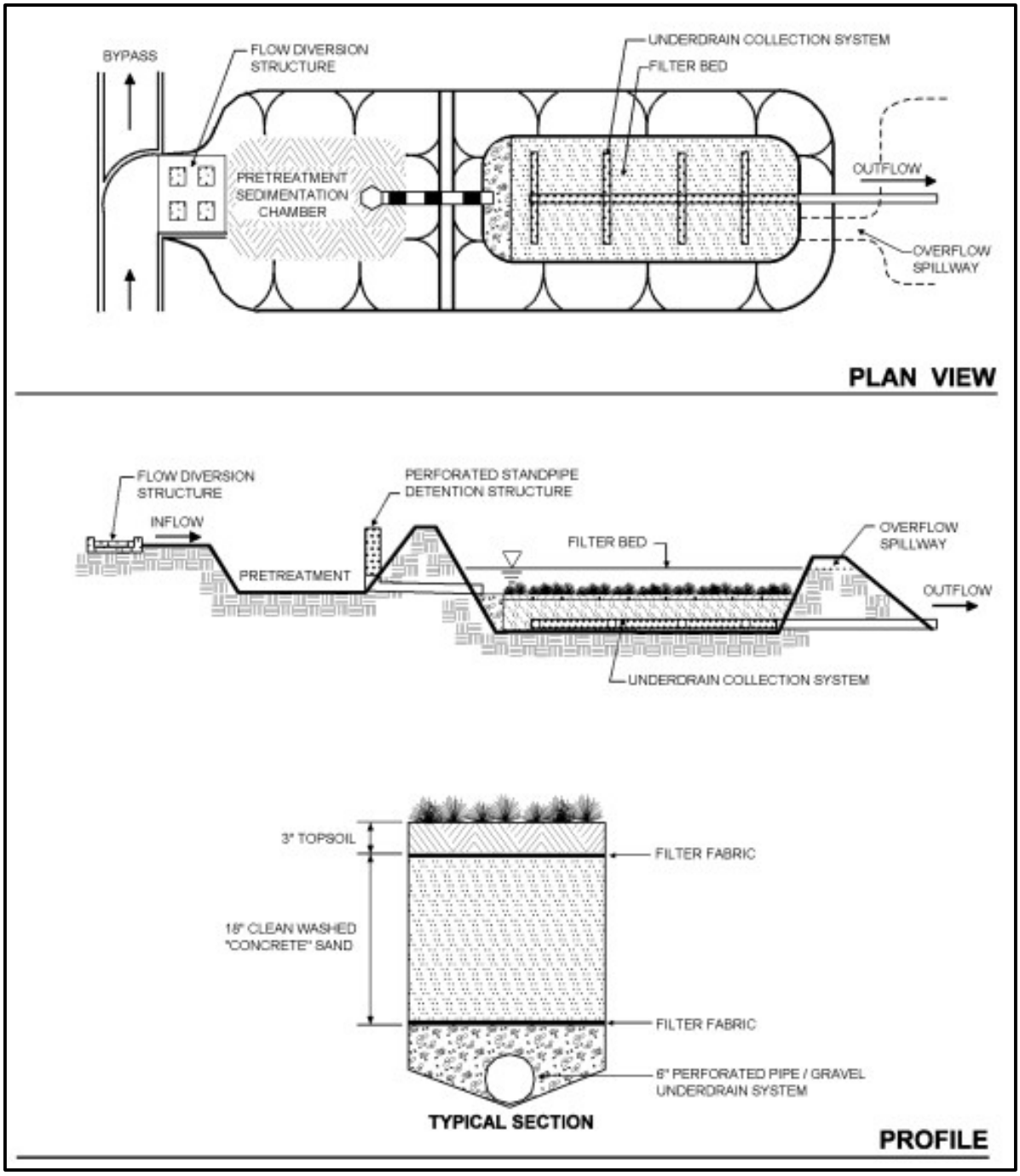


Figure C-40. Schematic of Surface Filter.

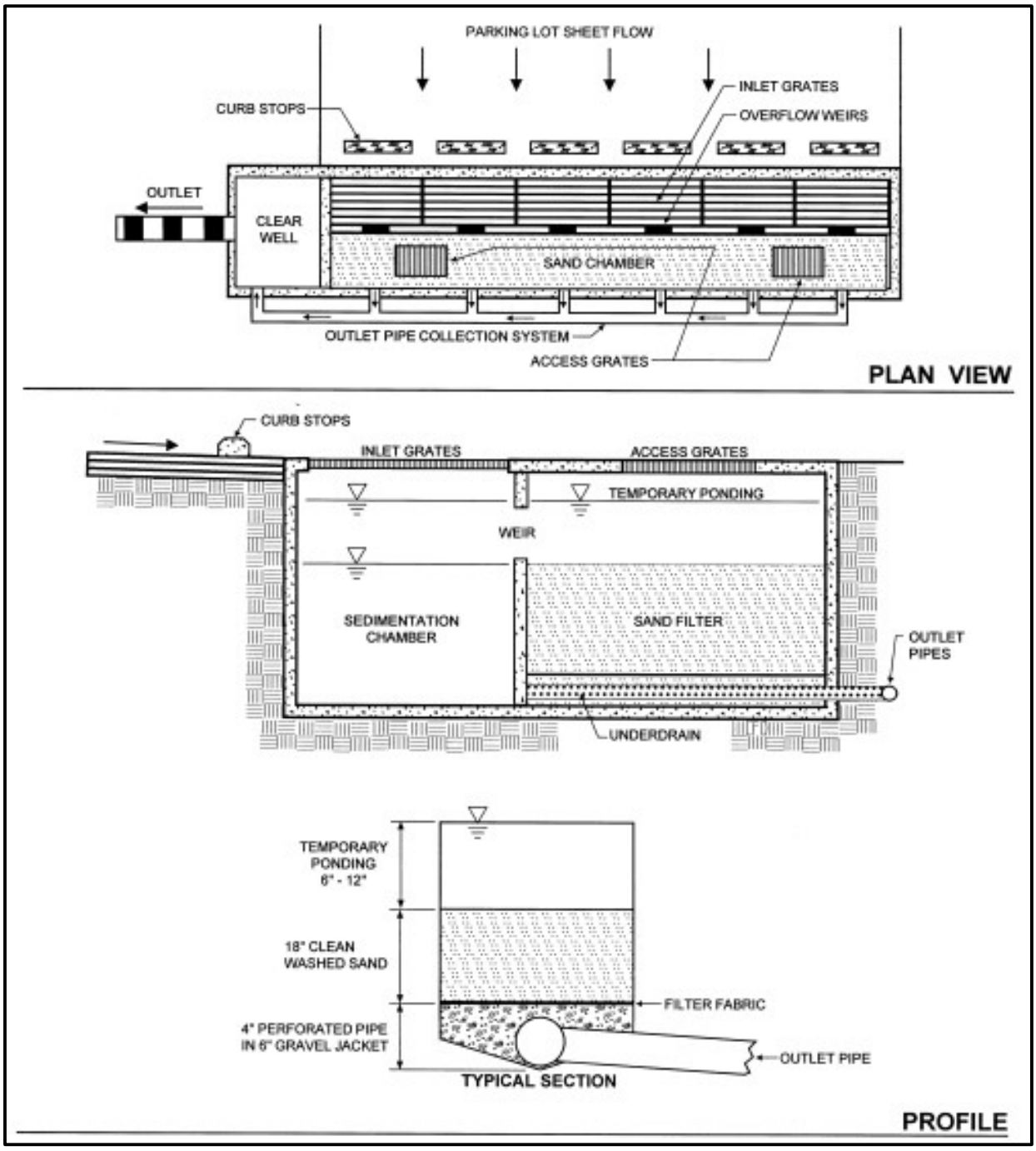


Figure C-41. Schematic of Perimeter Sand Filter.

5. Site Feasibility and Design Applications

Sand filter systems are well suited for highly impervious areas where land available for structural controls is limited. Sand filters should primarily be considered for new construction or retrofit opportunities for commercial, industrial, and institutional areas where the sediment load is relatively low, such as: parking lots, driveways, loading docks, gas stations, garages, airport runways/taxiways, and storage yards. Sand filters may also be feasible and appropriate in some multi-family or higher density residential developments.

To avoid rapid clogging and failure of the filter media, the use of sand filters should be avoided in areas with less than 50% impervious cover, or high sediment yield sites with clay/silt soils.

The following basic criteria should be evaluated to ensure the suitability of a sand filter facility for meeting stormwater management objectives on a site or development.

General Feasibility

- Suitable for Residential Subdivision Usage – NO
- Suitable for High Density/Ultra Urban Areas – YES
- Regional Stormwater Control – NO

Physical Feasibility - Physical Constraints at Project Site

- **Drainage Area** – 10 acres maximum for surface sand filter; 2 acres maximum for perimeter sand filter
- **Space Required** – Function of available head at site.
- **Site Slope** – No more than 6% slope across filter location.
- **Minimum Head** – Elevation difference needed at a site from the inflow to the outflow: 5 ft for surface sand filters; 2 to 3 ft for perimeter sand filters.
- **Minimum Depth to Water Table** – For a surface sand filter with exfiltration (earthen structure), 2 ft are required between the bottom of the sand filter and the elevation of the seasonally high-water table.
- **Soils** – No restrictions; Group “A” soils generally required to allow exfiltration (for surface sand filter earthen structure).

Other Constraints / Considerations

- Aquifer Protection – Do not allow exfiltration of filtered runoff into groundwater from areas with potential for high pollutant loading.

6. Planning and Design Criteria

The following criteria are to be considered minimum standards for the design of a sand filter facility.

Location and Siting

- Surface sand filters should have a contributing drainage area of 10 acres or less. The maximum drainage area for a perimeter sand filter is 2 acres.
- Sand filter systems are generally applied to land uses with a high percentage of impervious surfaces. Sites with less than 50% imperviousness or high clay/silt sediment loads must not use a sand filter without adequate pretreatment due to potential clogging and failure of the filter bed. Any disturbed areas within the sand filter facility drainage area should be identified and stabilized. Filtration controls should only be constructed after the construction site is stabilized.
- Surface sand filters are generally used in an off-line configuration where the Water Quality Volume (WQ_v) is diverted to the filter facility through the use of a flow diversion structure and flow splitter. Stormwater flows greater than the WQ_v are diverted to other controls or downstream using a diversion structure or flow splitter.

- Perimeter sand filters are typically sited along the edge, or perimeter, of an impervious area such as a parking lot.
- Sand filter systems are designed for intermittent flow and must be allowed to drain and reaerate between rainfall events. They should not be used on sites with a continuous flow from groundwater, sump pumps, or other sources.

General Design

- ***Surface Sand Filter***
  A surface sand filter facility consists of a two-chamber open-air structure, which is located at ground-level. The first chamber is the sediment forebay (a.k.a. sedimentation chamber) while the second chamber houses the sand filter bed. Flow enters the sedimentation chamber where settling of larger sediment particles occurs. Runoff is then discharged from the sedimentation chamber through a perforated standpipe into the filtration chamber. After passing though the filter bed, runoff is collected by a perforated pipe and gravel underdrain system. Figure C-19 provides plan view and profile schematics of a surface sand filter.

- ***Perimeter Sand Filter***
  A perimeter sand filter facility is a vault structure located just below grade level. Runoff enters the device through inlet grates along the top of the structure into the sedimentation chamber. Runoff is discharged from the sedimentation chamber through a weir into the filtration chamber. After passing though the filter bed, runoff is collected by a perforated pipe and gravel underdrain system. Figure C-20 provides plan view and profile schematics of a perimeter sand filter.

Physical Specificiations / Geometry

- ***Surface Sand Filter***
  The entire treatment system (including the sedimentation chamber) must temporarily hold at least 75% of the WQ_v prior to filtration. Figure C-21 illustrates the distribution of the Water Quality Volume (0.75 WQ_v) among the various components of the surface sand filter, including:
    - V_s – volume within the sedimentation basin
    - V_f – volume within the voids in the filter bed
    - $V_{f\text{-}temp}$ – temporary volume stored above the filter bed
    - A_s – the surface area of the sedimentation basin
    - A_f – surface area of the filter media
    - h_s – height of water in the sedimentation basin
    - h_f – average height of water above the filter media
    - d_f – depth of filter media
  - The sedimentation chamber must be sized to at least 25% of the computed WQ_v and have a length-to-width ratio of at least 2:1. Inlet and outlet structures should be located at opposite ends of the chamber.
  - The filter area is sized based on the principles of Darcy's Law. A coefficient of permeability (k) of 3.5 ft/day for sand should be used. The filter bed is typically designed to completely drain in 40 hours or less.
  - The filter media consists of an 18-inch layer of clean washed medium sand (meeting ASTM C-33 concrete sand) on top of the underdrain system. Three inches of topsoil are placed over the sand bed. Permeable filter fabric is placed both above and below the sand bed to prevent clogging of the sand filter and the underdrain system. Figure C-23 illustrates a typical media cross section.
    - The filter bed is equipped with a 6-inch perforated PVC pipe (AASHTO M 252) underdrain in a gravel layer. The underdrain must have a minimum grade of 1/8-inch per foot (1% slope). Holes should be 3/8-inch diameter and spaced approximately 6 inches on center. Gravel should be clean washed aggregate with a maximum diameter of 3.5 inches and a minimum diameter of 1.5 inches with a void space of about 40%. Aggregate contaminated with soil shall not be used.

- The structure of the surface sand filter may be constructed of impermeable media such as concrete or using excavations and earthen embankments. When constructed with earthen walls/embankments, filter fabric should be used to line the bottom and side slopes of the structures before installation of the underdrain system and filter media.

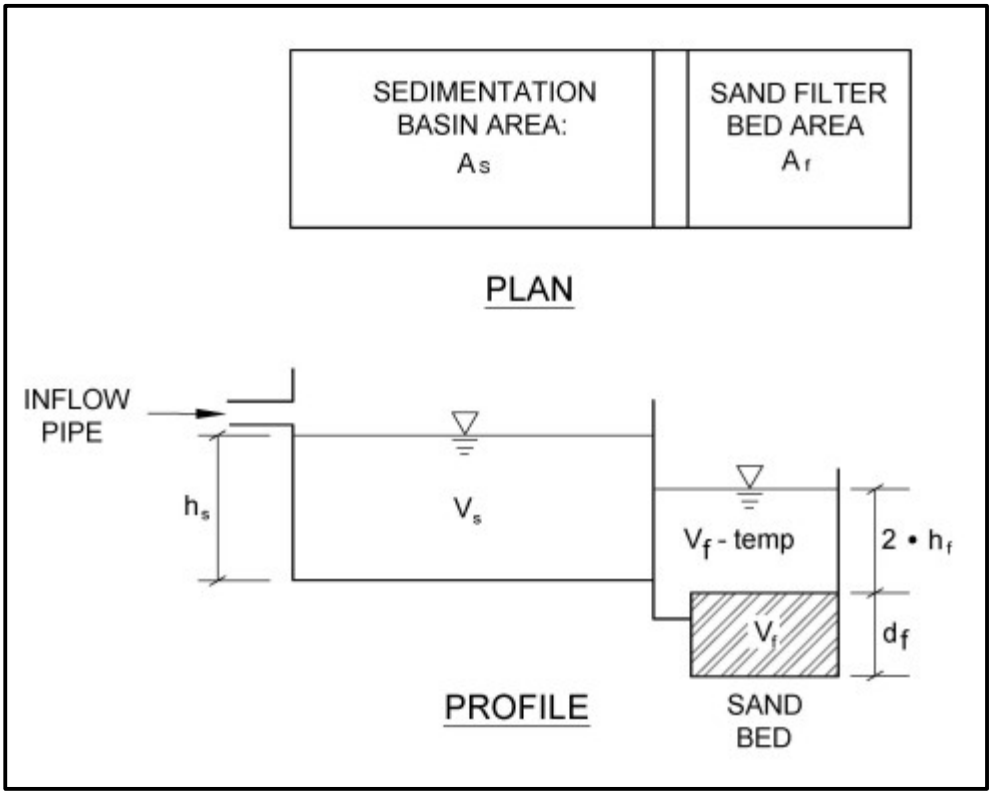


Figure C-42. Surface Sand Filter Volumes.

- ***Perimeter Sand Filter***
  - The entire treatment system (including the sedimentation chamber) must temporarily hold at least 75% of the WQ_v prior to filtration. Figure C-22 illustrates the distribution of the Water Quality Volume (0.75 WQ_v) among the various components of the perimeter sand filter, including:
    - Vw – wet pool volume within the sedimentation basin
    - Vf – volume within the voids in the filter bed
    - Vtemp – temporary volume stored above the filter bed
    - As – the surface area of the sedimentation basin
    - Af – surface area of the filter media
    - hf – average height of water above the filter media (1/2 h_{temp})
    - df – depth of filter media
  - The sedimentation chamber must be sized to at least 50% of the computed WQ_v.
  - The filter area is sized based on the principles of Darcy's Law. A coefficient of permeability (k) of 3.5 ft/day for sand should be used. The filter bed is typically designed to completely drain in 40 hours or less.
    - The filter media should consist of a 12- to 18-inch layer of clean washed medium sand (meeting ASTM C-33 concrete sand) on top of the underdrain system. Figure C-23 illustrates a typical media cross section.

- The perimeter sand filter is equipped with a 4-inch perforated PVC pipe (AASHTO M 252) underdrain in a gravel layer. The underdrain must have a minimum grade of 1/8 inch per foot (1% slope). Holes should be 3/8-inch diameter and spaced approximately 6 inches on center. A permeable filter fabric should be placed between the gravel layer and the filter media. Gravel should be clean washed aggregate with a maximum diameter of 3.5 inches and a minimum diameter of 1.5 inches with a void space of about 40%. Aggregate contaminated with soil shall not be used.

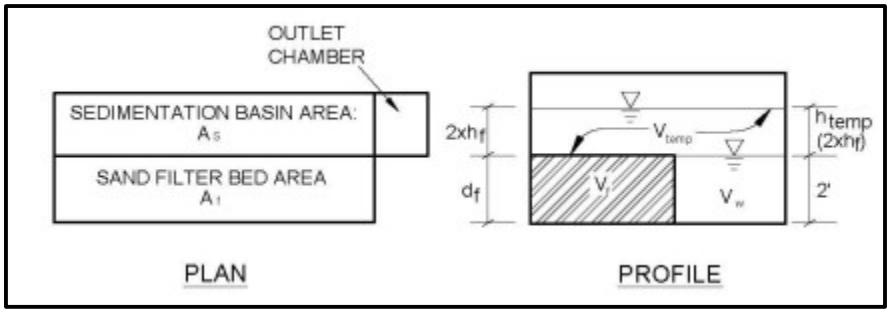


Figure C-43. Perimeter Sand Filter Volumes.

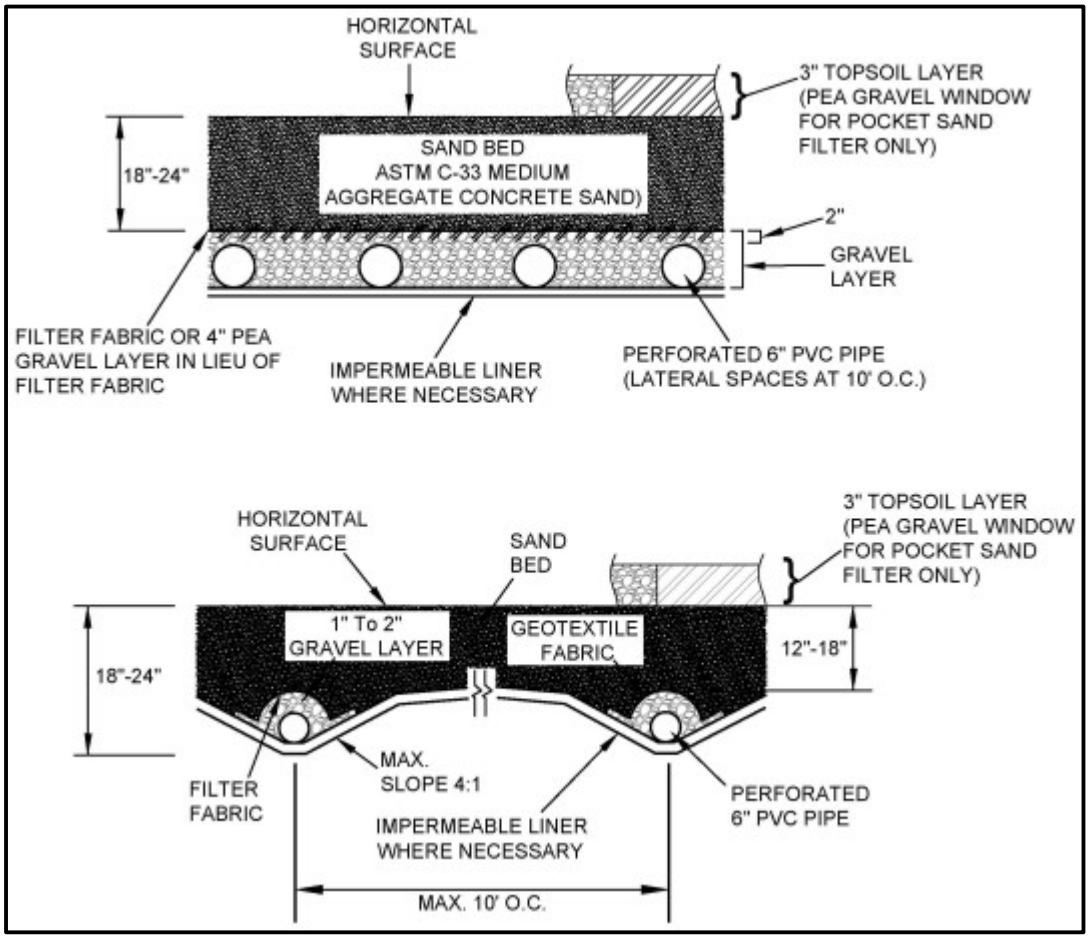


Figure C-44. Typical Sand Filter Media Cross Section.

Pretreatment / Inlets

- Pretreatment of runoff in a sand filter system is provided by the sedimentation chamber.
- Inlets to surface sand filters are to be provided with energy dissipators. Exit velocities from the sedimentation chamber must be nonerosive.
- Figure C-24 shows a typical inlet pipe from the sedimentation basin to the filter media basin for the surface sand filter.

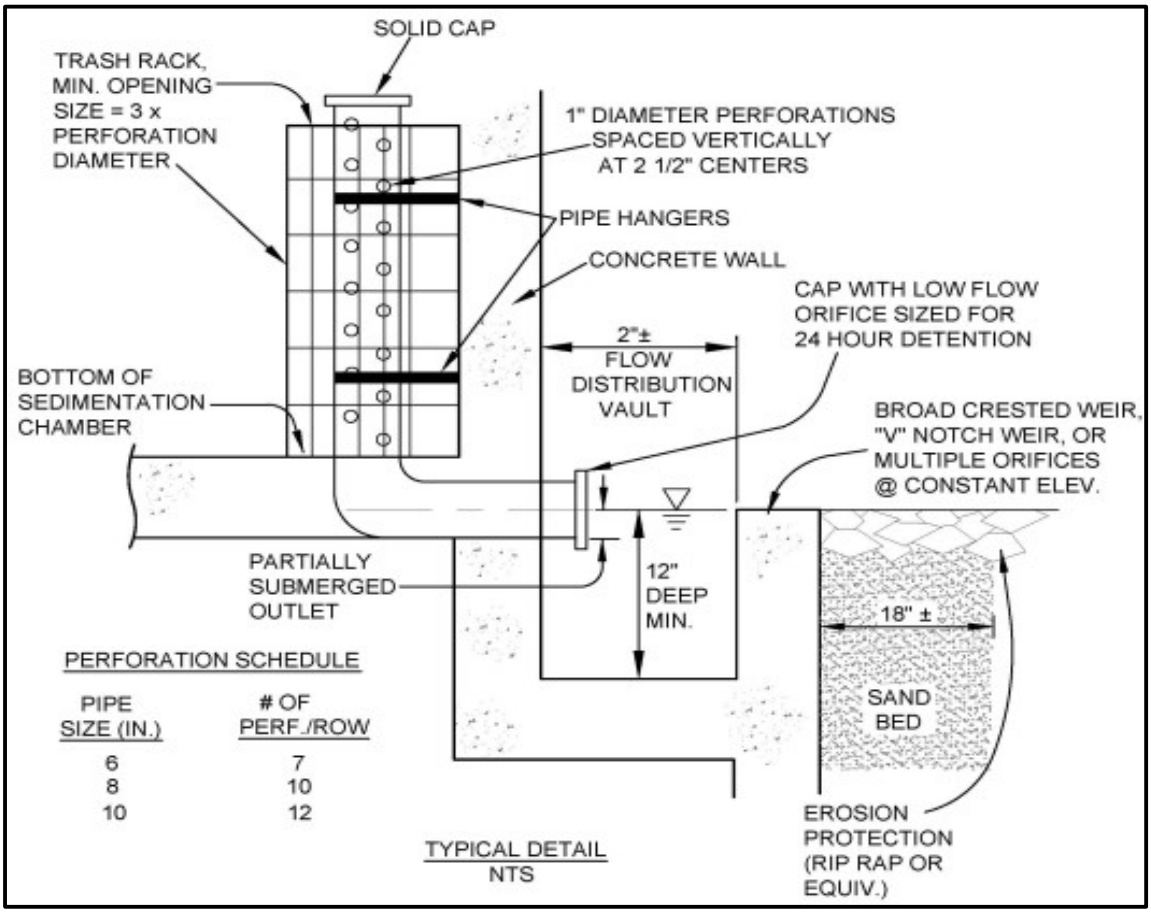


Figure C-45. Surface Sand Filter Perforated Standpipe.

Outlet Structures

- Outlet pipe is to be provided from the underdrain system to the facility discharge. Due to the slow rate of filtration, outlet protection is generally unnecessary (except for emergency overflows and spillways).

Emergency Spillway

- An emergency or bypass spillway must be included in the surface sand filter to safely pass flows that exceed the design storm flows. The spillway prevents filter water levels from overtopping the embankment and causing structural damage. The emergency spillway should be located so that downstream buildings and structures will not be impacted by spillway discharges.

Maintenance Access

- Adequate access must be provided for all sand filter systems for inspection and maintenance, including the appropriate equipment and vehicles. Access grates to the filter bed need to be included in a perimeter sand filter design. Facility designs must enable maintenance personnel to easily replace upper layers of the filter media.

Safety Features

- Surface sand filter facilities can be fenced to prevent access. Inlet and access grates to perimeter sand filters may be locked.

Landscaping

- Surface filters can be designed with a grass cover to aid in pollutant removal and prevent clogging. The grass should be capable of withstanding frequent periods of inundation and drought.

Additional Site-Specific Design Criteria and Issues

- Physiographic Factors - Local terrain design constraints
    - Low Relief – Use of surface sand filter may be limited by low head.
    - High Relief – Filter bed surface must be level.
- Soils
    - No restrictions
- Special Downstream Watershed Considerations
    - Aquifer Protection – Use polyliner or impermeable membrane to seal bottom of earthen surface sand filter or use watertight structure, no exfiltration of filter runoff into groundwater.

7. Design Procedures

Step 1. Compute runoff control volumes from the Unified Stormwater Sizing Criteria

Calculate the Water Quality Volume (WQ_v) and design storm events. Details on the Stormwater Sizing Criteria are found in Chapter 2.

Step 2. Determine if the development site and conditions are appropriate for the use of a surface or perimeter sand filter.

Consider the Application and Site Feasibility Criteria in the Planning and Design Criteria section of this specification (Location and Siting).

Step 3. Confirm local design criteria and applicability.

Consider any special site-specific design conditions/criteria from the Planning and Design Criteria section of this specification (Additional Site-Specific Design Criteria and Issues). Check with City Engineer to determine if there are any additional restrictions and/or surface water or watershed requirements that may apply.

Step 4. Compute WQ_v peak discharge (Q_{wq})

The peak rate of discharge for water quality design storm is needed for sizing of off-line diversion structures (see Chapter 3).

1. Using WQ_v, compute CN.
2. Compute time of concentration using TR-55 method.
3. Determine appropriate unit peak discharge from time of concentration.
4. Compute Q_{wq} from unit peak discharge, drainage area, and WQ_v.

Step 5. Size flow diversion structure, if needed

A flow regulator (or flow splitter diversion structure) should be supplied to divert the WQ_v to the sand filter facility. Size low flow orifice, weir, or another device to pass Qw_q.

Step 6. Size filtration basin chamber

The filter area is sized using the following equation (based on Darcy's Law):

$$A_f = [(WQ_v)(d_f)]/[(k)(h_f + d_f)(t_f)] \quad \textbf{Eq. C-9}$$

Where: A_f = surface area of filter bed (ft^2)

d_f = filter bed depth (typically 18 inches, no more than 24 inches)

k = coefficient of permeability of filter media (ft/day)

h_f = average height of water above filter bed (ft)
(1/2 h_{max}, which varies based on site but h_{max} is typically ≤ 6 ft)

t_f = design filter bed drain time (days) (1.67 days or 40 hours is recommended maximum)

Set preliminary dimensions of filtration basin chamber.

Step 7. Size sedimentation chamber

Surface sand filter: The sedimentation chamber should be sized to at least 25% of the computed WQ_v and have a length-to-width ratio of 2:1. The Camp-Hazen equation is used to compute the required surface area:

$$A_s = -\left(\frac{Q_o}{w}\right) * Ln(1 - E) \quad \textbf{Eq. C-10}$$

Where: A_s = sedimentation basin surface area (ft^2)

Q_o = rate of outflow = the WQ_v over a 24-hour period

w = particle settling velocity (ft/sec)

E = trap efficiency

Assuming:

- 90% sediment trap efficiency (0.9)
- Particle settling velocity (ft/sec) = 0.0033 ft/sec for imperviousness < 75%
- Particle settling velocity (ft/sec) = 0.0004 ft/sec for imperviousness ≥ 75%
- Average of 24-hour holding period

Then:

- A_s = (0.066) (WQ_v) ft^2 for I < 75%
- A_s = (0.0081) (WQ_v) ft^2 for I ≥ 75%

Set preliminary dimensions of sedimentation chamber.

Perimeter sand filter: The sedimentation chamber should be sized to at least 50% of the computed WQ_v. Use same approach as for surface sand filter.

Step 8. Compute V_{min}

$$V_{min} = 0.75 * WQ_v \quad \text{Eq. C-11}$$

Step 9. Compute storage volumes within entire facility and sedimentation chamber orifice size

Surface sand filter:

$$V_{min} = 0.75 * WQ_v = V_s + V_f + V_{f-temp} \quad \text{Eq. C-12}$$

1. Compute V_f = water volume within filter bed/gravel/pipe = A_f * d_f * n
   Where: n = porosity = 0.4 for most applications
2. Compute Vf_{-temp} = temporary storage volume above the filter bed = 2 * h_f * A_f
3. Compute V_s = volume within sediment chamber = V_{min} - V_f - V_{f-temp}
4. Compute h_s = height in sedimentation chamber = V_s/A_s
5. Ensure h_s and h_f fit available head and other dimensions still fit – change as necessary in design iterations until all site dimensions fit.
6. Size orifice from sediment chamber to filter chamber to release V_s within 24-hours at average release rate with 0.5 h_s as average head.
7. Design outlet structure with perforations allowing for a safety factor of 10 (see example)
8. Size distribution chamber to spread flow over filtration media – level spreader weir or orifices.

Perimeter sand filter:

1. Compute V_f = water volume within filter bed/gravel/pipe = A_f * d_f * n
2. Where: n = porosity = 0.4 for most applications
3. Compute V_w = wet pool storage volume A_s * 2 ft minimum
4. Compute V_{temp} = temporary storage volume = V_{min} – (V_f + V_w)
5. Compute h_{temp} = temporary storage height = V_{temp} / (A_f + A_s)
6. Ensure h_{temp} ≥ 2 * h_f, otherwise decrease h_f and re-compute. Ensure dimensions fit available head and area – change as necessary in design iterations until all site dimensions fit.
7. Size distribution slots from sediment chamber to filter chamber.

Step 10. Design inlets, pretreatment facilities, underdrain system, and outlet structures.

See the Planning and Design Criteria section of this specification for more detail.

Step 11. Compute overflow weir sizes.

Surface sand filter:

1. Size overflow weir at elevation hs in sedimentation chamber (above perforated standpipe) to handle surcharge of flow through filter system from 25-year storm (see example). Plan inlet protection for overflow from sedimentation chamber and size overflow weir at elevation hf in filtration chamber (above perforated standpipe) to handle surcharge of flow through filter system from 25-year storm (see example).

Perimeter sand filter:

1. Size overflow weir at end of sedimentation chamber to handle excess inflow, set at WQ_v elevation.

8. Inspection and Maintenance Requirements

Table C-16. Typical Maintenance Activities for Sand Filters.

Activity	Schedule
• Ensure that contributing area, facility, inlets, and outlets are clear of debris. • Ensure that the contributing area is stabilized and mowed, with clippings removed. • Remove trash and debris. • Check to ensure that the filter surface is not clogging (also check after moderate and major storms). • Ensure that activities in the drainage area minimize oil/grease and sediment entry to the system. • If permanent water level is present (perimeter sand filter), ensure that the chamber does not leak, and normal pool level is retained.	Monthly
• Check to see that the filter bed is clean of sediment, and the sediment chamber is not more than 50% full or 6 inches, whichever is less, of sediment. Remove sediment as necessary. • Make sure that there is no evidence of deterioration, spalling or cracking of concrete. • Inspect grates (perimeter sand filter). • Inspect inlets, outlets, and overflow spillway to ensure good condition and no evidence of erosion. • Repair or replace any damaged structural parts. • Stabilize any eroded areas. • Ensure that flow is not bypassing the facility. • Ensure that no noticeable odors are detected outside the facility.	Annually
• If filter bed is clogged or partially clogged, manual manipulation of the surface layer of sand may be required. Remove the top few inches of sand, roto-till or otherwise cultivate the surface, and replace media with sand meeting the design specifications. • Replace any filter fabric that has become clogged.	As needed

Additional Maintenance Considerations and Requirements

- A record should be kept of the dewatering time for a sand filter to determine if maintenance is necessary.
- When the filtering capacity of the sand filter facility diminishes substantially (i.e., when water ponds on the surface of the filter bed for more than 48 hours), then the top layers of the filter media (topsoil and 2 to 3 inches of sand) will need to be removed and replaced. This will typically need to be done every 3 to 5 years for low sediment applications, more often for areas of high sediment yield or high oil and grease.
- Removed sediment and media may usually be disposed of in a landfill.

9. References

Atlanta Regional Commission, 2001. *Georgia Stormwater Management Manual, Volume 2: Technical Handbook.* Atlanta, GA. http://www.georgiastormwater.com/GSMMVol2.pdf

Center for Watershed Protection (CWP), 1996. Design of Stormwater Filtering Systems. Prepared for Chesapeake Research Consortium.

City of Fayetteville, 2014. *Drainage Criteria Manual.* Fayetteville, AR. Drainage-Criteria-Manual-Appendix-F-PDF (fayetteville-ar.gov)

Clayton, R., & Schueler, T. (1996). Design of stormwater filtering systems. Ellicott City, MD. Center for Watershed Protection.

Pitt, R., M. Lilbum, S. Nix, S. Durrans, and S. Burian, 1997. *Guidance Manual for Integrated Wet Weather Flow Collection and Treatment Systems for Newly Urbanized Areas.* US EPA. Office of Research and Development.

Watershed Management Institute (WMI), 1997. Operation, Maintenance, and Management of Stormwater Management Systems. Prepared for US EPA, Office of Water.

WQ-07 Organic Filter

1. Description

The organic filter is a design variant of the surface sand filter, which uses organic materials such as leaf compost or a peat/sand mixture as the filter media. The organic material enhances pollutant removal by providing adsorption of contaminants such as soluble metals, hydrocarbons, and other organic chemicals.

As with the surface sand filter, an organic filter consists of a pretreatment chamber, and one or more filter cells. Each filter bed contains a layer of leaf compost or the peat/sand mixture, followed by filter fabric and a gravel/perforated pipe underdrain system. The filter bed and subsoils can be separated by an impermeable polyliner or concrete structure to prevent movement into groundwater.

Organic filters are typically used in high-density applications, or for areas requiring an enhanced pollutant removal ability. Maintenance is typically higher than the surface sand filter facility due to the potential for clogging. In addition, organic filter systems have a higher head requirement than sand filters.

2. Pollutant Removal Capabilities

Peat/sand filter systems provide good removal of bacteria and organic waste metals. The following design pollutant removal rates are conservative average pollutant reduction percentages for design purposes derived from sampling data, modeling, and professional judgment.

- Total Suspended Solids – 80%
- Total Phosphorus – 60%
- Total Nitrogen – 40%
- Fecal Coliform – 50%
- Heavy Metals – 75%

3. Design Criteria and Specifications

- Organic filters are typically used on relatively small sites (up to 10 acres), to minimize potential clogging.
- The minimum head requirement (elevation difference needed at a site from the inflow to the outflow) for an organic filter is 5 to 8 ft.
- Organic filters can utilize a variety of organic materials as the filtering media. Two typical media bed configurations are the peat/sand filter and compost filter (see Figure C-25). The peat filter includes an 18-inch 50/50 peat/sand mix over a 6-inch sand layer and can be optionally covered by 3 inches of topsoil and vegetation. The compost filter has an 18-inch compost layer. Both variants utilize a gravel underdrain system.
- The type of peat used in a peat/sand filter is critically important. Fibric peat in which undecomposed fibrous organic material is readily identifiable is the preferred type. Hemic peat containing more decomposed material may also be used. Sapric peat made up of largely decomposed matter should not be used in an organic filter.
- Typically, organic filters are designed as “off-line” systems, meaning that the water volume (WQ_v) is diverted to the filter facility through the use of a flow diversion structure and flow splitter. Stormwater flows greater than the WQ_v are diverted to other controls or downstream using a diversion structure or flow splitter.
- Consult the design criteria for the surface sand filter (see WQ-04, Sand Filters) for the organic filter sizing and design steps.

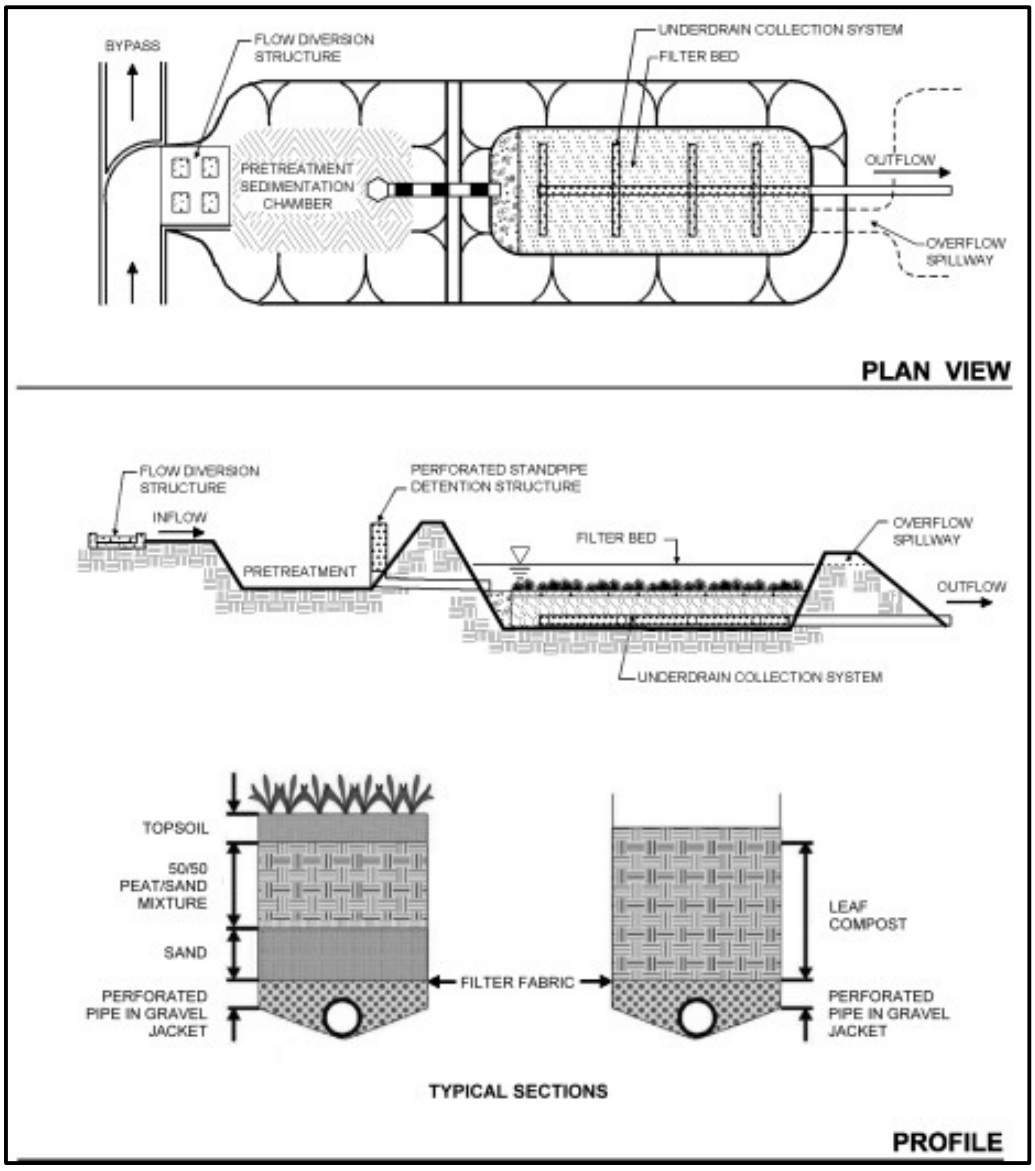


Figure C-46. Schematic of Organic Filter.

4. Inspection and Maintenance Requirements

The inspection and maintenance requirements for organic filters are like those for surface sand filter facilities (see WQ-04, Sand Filter, in Appendix C).

5. References

Atlanta Regional Commission, 2001. *Georgia Stormwater Management Manual, Volume 2: Technical Handbook.* Atlanta, GA. http://www.georgiastormwater.com/GSMMVol2.pdf

Center for Watershed Protection (CWP), 1996. Design of Stormwater Filtering Systems. Prepared for Chesapeake Research Consortium.

City of Fayetteville, 2014. *Drainage Criteria Manual.* Fayetteville, AR. Drainage-Criteria-Manual-Appendix-F-PDF (fayetteville-ar.gov)

WQ-08 Underground Sand Filter

1. Description

The underground sand filter is a design variant of the sand filter located in an underground vault designed for high-density land use or ultra-urban applications where there is not enough space for a surface sand filter or other structural stormwater controls.

The underground sand filter is a three-chamber system. The initial chamber is a sedimentation (pretreatment) chamber that temporarily stores runoff and utilizes a wet pool to capture sediment. The sedimentation chamber is connected to the sand filter chamber by a submerged wall that protects the filter bed from oil and trash. The filter bed is 18 to 24 inches deep and may have a protective screen of gravel or permeable geotextile to limit clogging. The sand filter chamber also includes an underdrain system with inspection and clean out wells. Perforated drainpipes under the sand filter bed extend into a third chamber that collects filtered runoff. Flows beyond the filter capacity are diverted through an overflow weir.

Due to its location below the surface, underground sand filters have a high maintenance burden and should only be used where adequate inspection and maintenance can be ensured.

2. Guidelines for Using Proprietary Systems

Underground sand filter pollutant removal rates are like those for surface and perimeter sand filters (see WQ-04, Sand Filters).

3. Design Criteria and Specifications

- Underground sand filters are typically used on highly impervious sites of 1 acre or less. The maximum drainage area that should be treated by an underground sand filter is 5 acres.
- Underground sand filters are typically constructed on-line but can be constructed off-line. For offline construction, the overflow between the second and third chambers is not included.
- The underground vault should be tested for water tightness prior to placement of filter layers.
- Adequate maintenance access must be provided to the sedimentation and filter bed chambers.
- Compute the minimum wet pool volume required in the sedimentation chamber as:
  $\boldsymbol{V_w = A_s * 3\ ft\ minimum}$
- Consult the design criteria for the perimeter sand filter (see WQ-04) for the rest of the underground filter sizing and design steps.

4. Typical Details

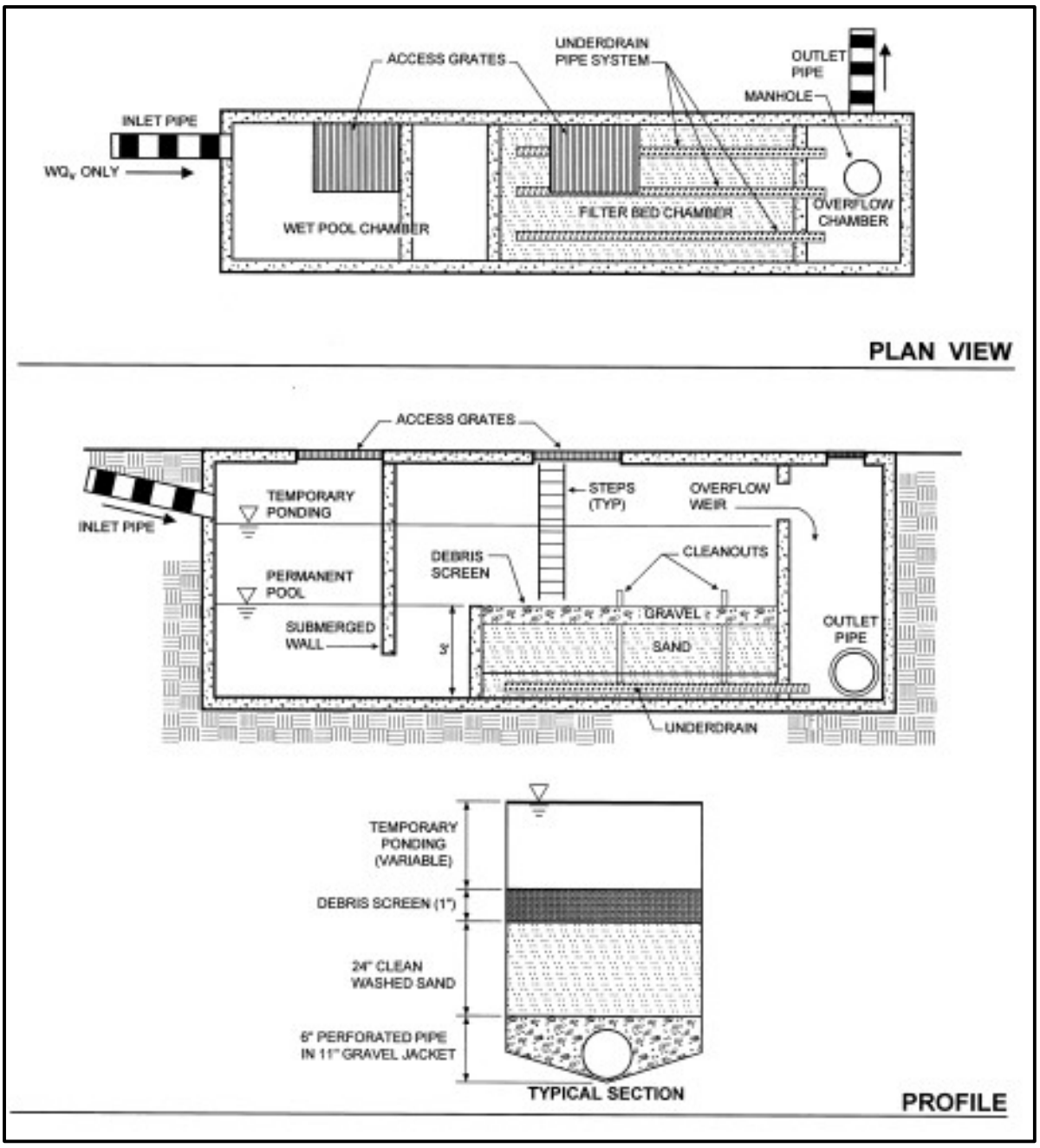


Figure C-47. Schematic of Underground Sand Filter.

5. Inspection and Maintenance Requirements

Table C-17. Typical Maintenance Activities for Underground Sand Filters

Activity	Schedule
• Monitor water level in sand filter chamber.	Quarterly and following large storm events
• Sedimentation chamber should be cleaned out when the sediment depth reaches 12 inches.	As needed
• Remove accumulated oil and floatables in sedimentation chamber.	As needed, (typically every 6 months)

Additional inspection and maintenance requirements for organic filters are similar to those for surface sand filter facilities (see WSC-04, Sand Filters, in Appendix C)

6. References

Atlanta Regional Commission, 2001. *Georgia Stormwater Management Manual, Volume 2: Technical Handbook.* Atlanta, GA. http://www.georgiastormwater.com/GSMMVol2.pdf

Center for Watershed Protection (CWP), 1996. Design of Stormwater Filtering Systems. Prepared for Chesapeake Research Consortium.

City of Fayetteville, 2014. *Drainage Criteria Manual.* Fayetteville, AR. Drainage-Criteria-Manual-Appendix-F-PDF (fayetteville-ar.gov)

WQ-09 Submerged Gravel Wetlands

1. Description

The submerged gravel wetland system consists of one or more treatment cells that are filled with crushed rock or gravel and is designed to allow stormwater to flow subsurface through the root zone of the constructed wetland. The outlet from each cell is set at an elevation to keep the rock or gravel submerged. Wetland plants are rooted in the media, where they can directly take up pollutants. In addition, algae and microbes thrive on the surface area of the rocks. In particular, the anaerobic conditions on the bottom of the filter can foster the denitrification process. Although widely used for wastewater treatment in recent years, only a handful of submerged gravel wetland systems have been designed to treat stormwater. Mimicking the pollutant removal ability of nature, this structural control relies on the pollutant-stripping ability of plants and soils to remove pollutants from runoff.

2. Pollutant Removal Capabilities

The pollution removal efficiency of the submerged gravel wetland is like a typical wetland. Recent data show a TSS removal rate more than the 80% goal. This reflects the settling environment of the gravel media. These systems also exhibit removals of about 60% TP, 20% TN, and 50% Zn. The growth of algae and microbes among the gravel media has been determined to be the primary removal mechanism of the submerged gravel wetland. The following design pollutant removal rates are conservative average pollutant reduction percentages for design purposes derived from sampling data, modeling, and professional judgment.

- Total Suspended Solids – 80%
- Total Phosphorus – 50%
- Total Nitrogen – 20%
- Fecal Coliform – 70%
- Heavy Metals – 50%

3. Design Criteria and Specifications

- Submerged gravel wetlands should be designed as off-line systems designed to handle only water quality volume.
- Submerged gravel wetland systems need sufficient drainage area to maintain vegetation.
- The local slope should be relatively flat (<2%). While there is no minimum slope requirement, there does need to be enough elevation drop from the inlet to the outlet to ensure that hydraulic. conveyance by gravity is feasible (generally about 3 to 5 ft).
- All submerged gravel wetland designs should include a sediment forebay or other equivalent pretreatment measures to prevent sediment or debris from entering and clogging the gravel bed.
- Unless they receive runoff from areas with potential for high pollutant loading, submerged gravel wetland systems can be allowed to intersect the groundwater table.
- See WQ-03, Stormwater Wetlands in Appendix C for additional planning and design guidance.

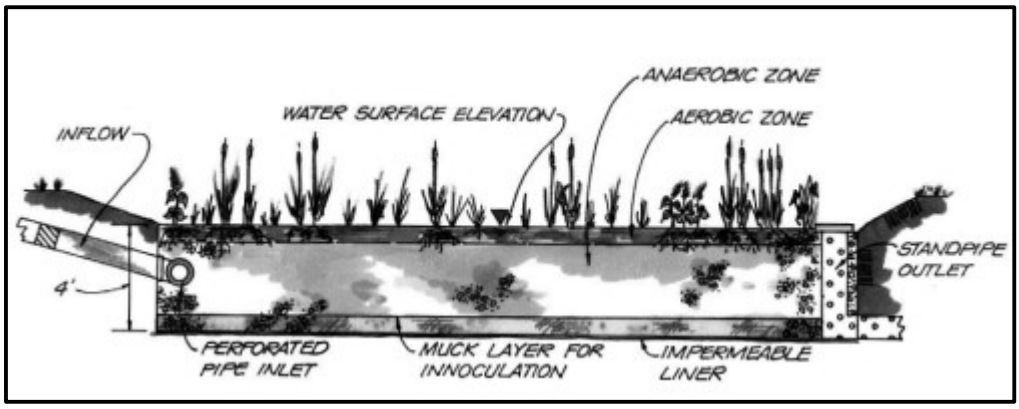


Figure C-48. Schematic of Submerged Gravel Wetland System.

4. Inspection and Maintenance Requirements

The inspection and maintenance requirements for submerged gravel wetlands are similar to those for stormwater wetlands (see WQ-03, Stormwater Wetlands, in Appendix C)

5. References

Atlanta Regional Commission, 2001. *Georgia Stormwater Management Manual, Volume 2: Technical Handbook.* Atlanta, GA. http://www.georgiastormwater.com/GSMMVol2.pdf

Center for Watershed Protection (CWP), 1996. Design of Stormwater Filtering Systems. Prepared for Chesapeake Research Consortium.

City of Fayetteville, 2014. *Drainage Criteria Manual.* Fayetteville, AR. Drainage-Criteria-Manual-Appendix-F-PDF (fayetteville-ar.gov)

WQ-10 Gravity (oil-grit) Separator

1. Description

Gravity separators (also known as oil-grit separators) are hydrodynamic separation devices that are designed to remove grit and heavy sediments, oil and grease, debris, and floatable matter from stormwater runoff through gravitational settling and trapping. Gravity separator units contain a permanent pool of water and typically consist of an inlet chamber, separation/storage chamber, a bypass chamber, and an access port for maintenance purposes. Runoff enters the inlet chamber where heavy sediments and solids drop out. The flow moves into the main gravity separation chamber, where further settling of suspended solids takes place. Oil and grease are skimmed and stored in a waste oil storage compartment for future removal. After moving into the outlet chamber, the clarified runoff is then discharged.

The performance of these systems is based primarily on the relatively low solubility of petroleum products in water and the difference between the specific gravity of water and the specific gravities of petroleum compounds. Gravity separators are not designed to separate other products such as solvents, detergents, or dissolved pollutants. The typical gravity separator unit may be enhanced with a pretreatment swirl concentrator chamber, oil draw-off devices that continuously remove the accumulated light liquids, and flow control valves regulating the flow rate into the unit.

Gravity separators are best used in commercial, industrial and transportation land uses and are intended primarily as a pretreatment measure for high-density or ultra urban sites, or for use in hydrocarbon hotspots, such as gas stations and areas with high vehicular traffic. However, gravity separators cannot be used for the removal of dissolved or emulsified oils and pollutants such as coolants, soluble lubricants, glycols, and alcohols.

Since resuspension of accumulated sediments is possible during heavy storm events, gravity separator units are typically installed off-line. Gravity separators are available as prefabricated proprietary systems from several different commercial vendors.

2. Pollutant Removal Capabilities

Testing of gravity separators has shown that they can remove between 40 and 50% of the TSS loading when used in an off-line configuration (Curran, 1996 and Henry, 1999). Gravity separators also provide removal of debris, hydrocarbons, trash and other floatables. They provide only minimal removal of nutrients and organic matter.

The following design pollutant removal rates are conservative average pollutant reduction percentages for design purposes derived from sampling data, modeling, and professional judgment.

- Total Suspended Solids – 40%
- Total Phosphorus – 5%
- Total Nitrogen – 5%
- Fecal Coliform – insufficient data
- Heavy Metals – insufficient data

Actual field-testing data and pollutant removal rates from an independent source should be obtained before using a proprietary gravity separator system.

3. Design Criteria and Specifications

The use of gravity (oil-grit) separators should be limited to the following applications:

- Pretreatment for other structural stormwater controls.
- High-density, ultra urban, or other space-limited development sites.
- Hotspot areas where the control of grit, floatables, and/or oil and grease are required.
- Gravity separators are typically used for areas less than 5 acres. It is recommended that the contributing area to any individual gravity separator be limited to 1 acre or less of impervious cover.

- Gravity separator systems can be installed in almost any soil or terrain. Since these devices are underground, appearance is not an issue and public safety risks are low.
- Gravity separators are rate-based devices. This contrasts with most other stormwater structural controls, which are sized based on capturing and treating a specific volume.
- Gravity separator units are typically designed to bypass runoff flows more than the design flow rate. Some designs have built-in high flow bypass mechanisms. Other designs require a diversion structure or flow splitter ahead of the device in the drainage system. An adequate outfall must be provided.
- The separation chamber should provide for three separate storage volumes:
    - A volume for separated oil storage at the top of the chamber.
    - A volume for settleable solids accumulation at the bottom of the chamber.
    - A volume required to give adequate flow-through detention time for separation of oil and sediment from the stormwater flow.
- The total wet storage of the gravity separator unit should be at least 400 cu ft per contributing impervious acre.
- The minimum depth of the permanent pools should be 4 ft.
- Horizontal velocity through the separation chamber should be 1 to 3 ft/min or less. No velocities in the device should exceed the entrance velocity.
- A trash rack should be included in the design to capture floating debris, preferably near the inlet chamber to prevent debris from becoming oil impregnated.
- Ideally, a gravity separator design will provide an oil draw-off mechanism to a separate chamber or storage area.
- Adequate maintenance access to each chamber must be provided for inspection and cleanout of a gravity separator unit.
- Gravity separator units should be watertight to prevent possible groundwater contamination.
- The design criteria and specifications of a proprietary gravity separator unit should be obtained from the manufacturer.

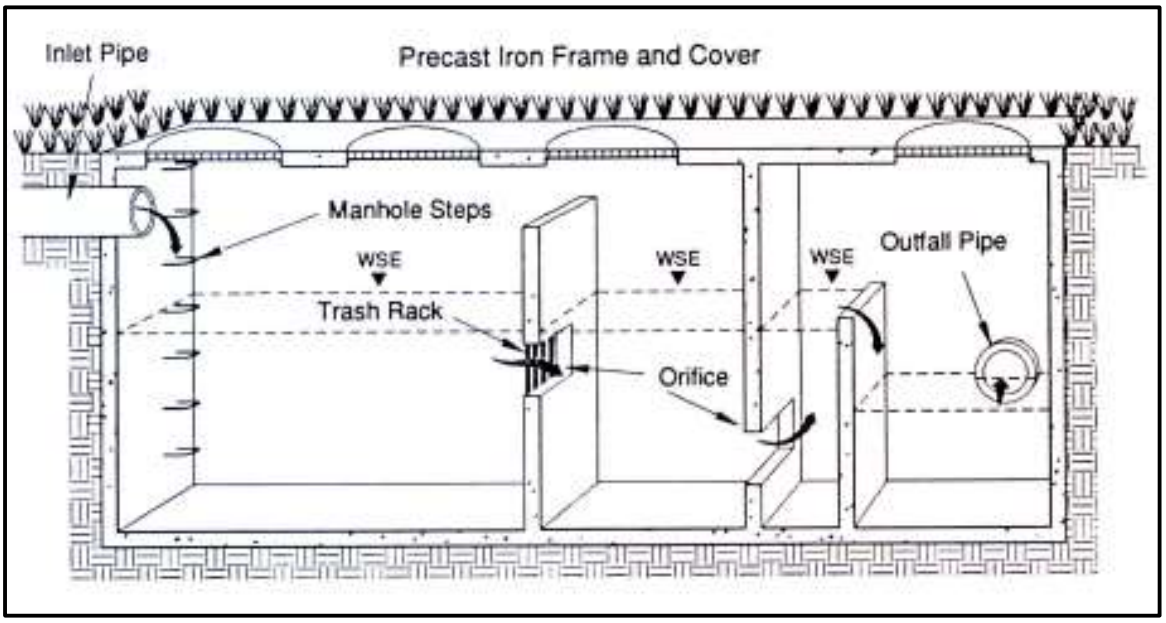


Figure C-49. Schematic of an Example Gravity (oil-grit) Separator.

4. Inspection and Maintenance Requirements

Table C-50. Typical Maintenance Activities for Gravity Separators.

Activity	Schedule
• Inspect the gravity separator unit.	Regularly (quarterly)
• Clean out sediment, oil, and grease, and floatables, using catch basin cleaning equipment (vacuum pumps). Manual removal of pollutants may be necessary.	As Needed

Additional Maintenance Considerations and Requirements

- Additional maintenance requirements for a proprietary system should be obtained from the manufacturer.
- Failure to provide adequate inspection and maintenance can result in the resuspension of accumulated solids. Frequency of inspection and maintenance is dependent on land use, climatological conditions, and the design of gravity separator.
- Proper disposal of oil, solids and floatables removed from the gravity separator must be ensured.

5. References

Atlanta Regional Commission, 2001. *Georgia Stormwater Management Manual, Volume 2: Technical Handbook.* Atlanta, GA. http://www.georgiastormwater.com/GSMMVol2.pdf

Center for Watershed Protection (CWP), 1996. Design of Stormwater Filtering Systems. Prepared for Chesapeake Research Consortium.

City of Fayetteville, 2014. *Drainage Criteria Manual.* Fayetteville, AR. Drainage-Criteria-Manual-Appendix-F-PDF (fayetteville-ar.gov)

Northern Virginia Regional Commission (NVRC), 1992.*The Northern Virginia BMP Handbook.* Annandale, VA.

WQ-11 Proprietary Structural Controls

1. Description

There are many types of commercially available proprietary stormwater structural controls available for both water quality treatment and quantity control. These systems include:

- Hydrodynamic systems such as gravity and vortex separators.
- Filtration systems.
- Catch basin media inserts.
- Chemical treatment systems.
- Package treatment plants.
- Prefabricated detention structures.

Many proprietary systems are useful on small sites and space-limited areas where there is not enough land or room for other structural control alternatives. Proprietary systems can often be used in pretreatment applications in a treatment train. However, proprietary systems are often more costly than other alternatives and may have high maintenance requirements. Perhaps the largest difficulty in using a proprietary system is the lack of adequate independent performance data. Below are general guidelines that should be followed before considering the use of a proprietary commercial system.

2. Guidelines for Using Proprietary Systems

For use as a limited application control, a proprietary system must have a demonstrated capability of meeting the stormwater management goals for which it is being intended. This means that the system must provide:

1. Independent third-party scientific verification of the ability of the proprietary system to meet water quality treatment objectives and/or to provide water quantity control (channel or flood protection).
2. Proven record of longevity in the field.
3. Proven ability to function in Little Rock conditions (e.g., climate, rainfall patterns, soil types, etc.).

For a propriety system to meet (1) above for water quality goals, the following monitoring criteria should be met for supporting studies:

- At least 15 storm events must be sampled.
- The study must be independent or independently verified (i.e., may not be conducted by the vendor or designer without third-party verification).
- The study must be conducted in the field, as opposed to laboratory testing.
- Field monitoring must be conducted using standard protocols which require proportional sampling both upstream and downstream of the device.
- Concentrations reported in the study must be flow weighted.
- The propriety system or device must have been in place for at least one year at the time of monitoring.

Although local data is preferred, data from other regions can be accepted as long as the design accounts for the local conditions.

A poor performance record or high failure rate is valid justification for not allowing the use of a proprietary system or device.

3. References

Atlanta Regional Commission, 2001. *Georgia Stormwater Management Manual, Volume 2: Technical Handbook.* Atlanta, GA. http://www.georgiastormwater.com/GSMMVol2.pdf

City of Fayetteville, 2014. *Drainage Criteria Manual.* Fayetteville, AR.
Drainage-Criteria-Manual-Appendix-F-PDF (fayetteville-ar.gov)

WQ-12 Permeable Pavement

1. Description

Permeable pavements are alternative paving surfaces that allow stormwater runoff to filter through voids in the pavement surface into an underlying stone reservoir, where it is temporarily stored and/or infiltrated. Permeable pavements consist of a surface pavement layer, an underlying stone aggregate reservoir layer and a filter layer or fabric installed on the bottom (See Figure C-50 below).

The thickness of the reservoir layer is determined by both a structural and hydrologic design analysis. The reservoir layer serves to retain stormwater and supports the design traffic loads for the pavement. In low-infiltration soils, some or all the filtered runoff is collected in an underdrain and returned to the storm drain system. If infiltration rates in the underlying soils permit, permeable pavement can be designed without an underdrain, to enable full infiltration of runoff. A combination of these methods can be used to infiltrate a portion of the filtered runoff.

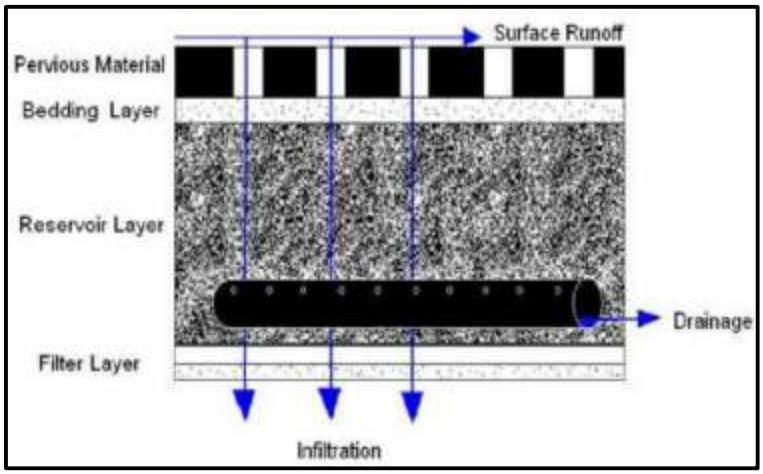


Figure C-50. Cross section of typical permeable pavement (Source: Hunt & Collins, 2008).

Permeable pavement is typically designed to treat stormwater that falls on the actual pavement surface area, but it may also be used to accept run-on from small adjacent impervious areas, such as impermeable driving lanes or rooftops. However, careful sediment control is needed for any run-on areas to avoid clogging of the down-gradient permeable pavement. Permeable pavement has been used at commercial, institutional, and residential sites in spaces that are traditionally impervious. Pervious pavement is well suited for parking lots, walking paths, sidewalks, playgrounds, plazas, tennis courts, and other similar uses. Permeable pavement promotes a high degree of runoff volume reduction, and it can also reduce the effective of impervious cover on a development site.

The various types of permeable pavement are listed below along with brief descriptions and photos.

Pervious Concrete:

Pervious concrete (Figure C-51) is produced by substantially reducing the number of fines in the concrete mix design in order to establish voids for drainage. Pervious concrete shall always be underlain by a stone subbase designed for stormwater management and shall never be placed directly onto a soil subbase. Pervious concrete has a coarser appearance than conventional concrete. Also, care must be taken during placement to avoid working the surface and creating an impervious layer. See Table C-21 for typical design information. A typical cross section is shown in Figure C-57.





Figure C-51. Pervious concrete walkways (Source: EPA 2009).

Pervious Asphalt:

Pervious Asphalt (Figure C-52) consists of standard bituminous asphalt in which the fines have been screened and reduced, allowing water to pass through small voids. Pervious asphalt is placed directly on the stone bedding layer above an open graded subbase. See Table C-22 for typical design information. A typical cross section is shown in Figure C-58.





Figure C-52. Pervious asphalt parking and recreation area (Source: PennDEP 2006).

Modular Block Paver Systems:

Pervious Paver Blocks (Figure C-22) consist of interlocking units (often concrete) that provide some portion of surface area that may be filled with a pervious material such as gravel. These units are often very attractive and are especially well suited to plazas, patios, small parking areas, etc. Several manufactured products are available for use as permeable pavement. The designer is encouraged to evaluate the benefits of various products with respect to the specific application and the desired stormwater management goals. See Table C-23 for typical design information. A typical cross section is shown in Figure C-58.

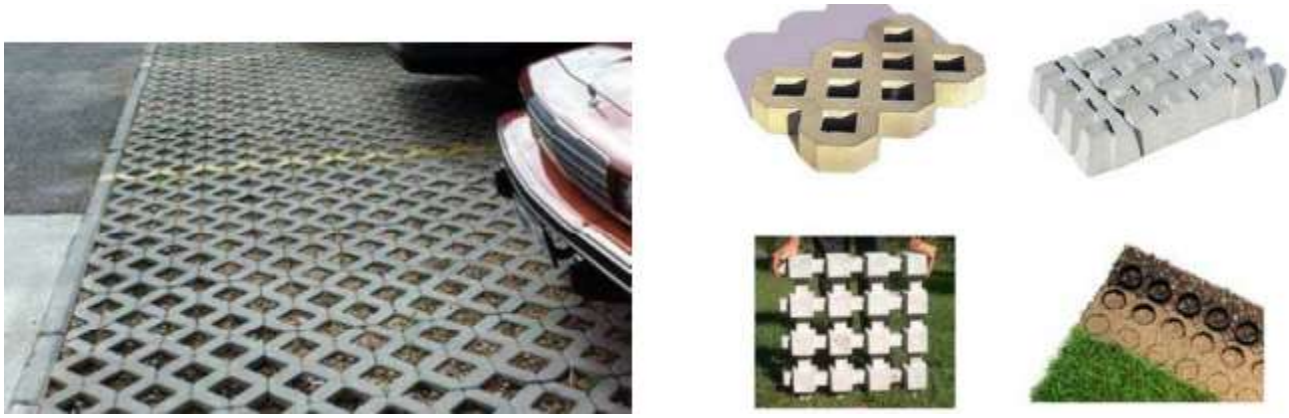


Figure C-53. Pervious asphalt parking and recreation area (Source: Atlanta Regional Commission 2001).

Grass Pavers:

Plastic rings in a flexible grid system are placed on a bedding layer above a base of No. 57 stone. The rings or grid system are then filled with a sandy topsoil such as sandy loam and planted with vegetation. This pavement gives designers a turfgrass alternative to asphalt or concrete for such low-traffic areas as fire lanes, overflow and event parking, golf cart paths, residential driveways, and maintenance and utility access lanes (see Figure C-54) The support base and the walls of the rings prevent soil compaction and reduce rutting and erosion by supporting the weight of traffic and concentrated loads, while the large void spaces in the rings allow a strong root network to develop. The result is a load-bearing surface covered with natural grass and which is typically around 90% pervious. See Table C-24 for typical design information. A typical cross section is shown in Figure C-60.



Figure C-54. Reinforced grass parking (Source: PennDEP 2006).

Gravel pavers:

This pavement option (Figures C-55 and C-56) is intended for high frequency, low speed traffic areas. The same ring structure as with the grass paver is used, but the voids in the rings are filled with gravel in order to provide greater load bearing support for unlimited traffic volumes and/or parking durations. Manufacturers provide specifications on the aggregate gradation that shall be used to generate the clean gravel fill for the rings, and a geotextile fabric is used to

prevent the gravel infill from migrating to the soil subbase. See Table C-25 for typical design information. A typical cross section is shown in Figure C-61.



Figure C-55. Reinforced gravel walkway (Source: PennDEP 2006).



Figure C-56. Gravel parking with concrete drive lanes (Source: FTN Associates, Ltd).

2. Typical Details

For each type of permeable pavement, a design table and typical design detail are provided below.

Table C-21. Pervious concrete pavement.

Typical Cross Section:	
Pavement thickness:	5 inches to 8 inches (Depth shall be determined by Professional Engineer)
Bedding Layer (Choker Course):	2 inches No. 8 stone
Reservoir Layer:	3 inches to 4 inches No. 57 stone 6 inches to 12 inches No. 2 stone (Depth shall be determined by Professional Engineer)
Filter Layer:	6 inches to 8 inches coarse sand (ASTM C33) on top of 2 inches to 4 inches No. 8 stone
Optional Geotextile Fabric:	4-oz minimum Non-Woven filter fabric

Design References: ACI 522R-06, Pervious Concrete

ACI522.1-08, “Specification for Pervious Concrete Pavement”

NRMCA, Pervious Concrete Pavements

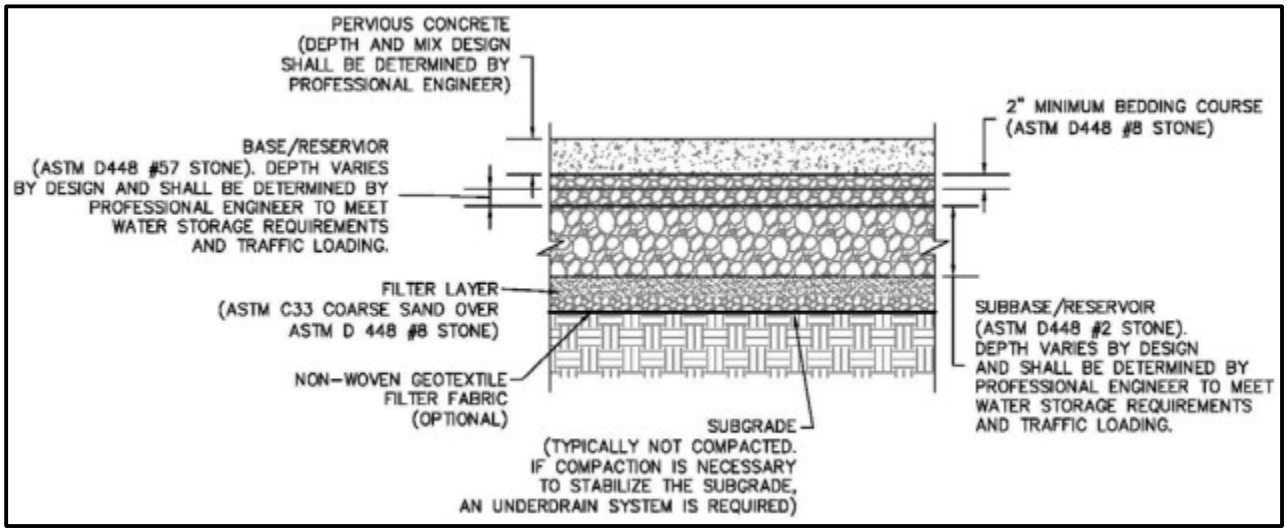


Figure C-57. Typical permeable concrete pavement section (Source: FTN Associates, Ltd).

Table C-22. Pervious Asphalt Pavement.

Typical Cross Section:	
Pavement thickness:	2-1/2 inches to 4 inches (Depth shall be determined by Professional Engineer)
Bedding Layer (Choker Course):	2 inches No. 8 stone
Reservoir Layer:	3 inches to 4 inches No. 57 stone 12 inches to 18 inches No. 2 stone (Depth of layers shall be determined by Professional Engineer)
Filter Layer:	6 inches to 8 inches coarse sand (ASTM C33) on top of 2 inches to 4 inches No. 8 stone
Geotextile Fabric:	4-oz minimum Non-Woven filter fabric

Design References: National Asphalt Pavement Association (NAPA), “Porous Asphalt Pavements for Stormwater Management, Design, Construction, and Maintenance Guide” (Revised November 2008).

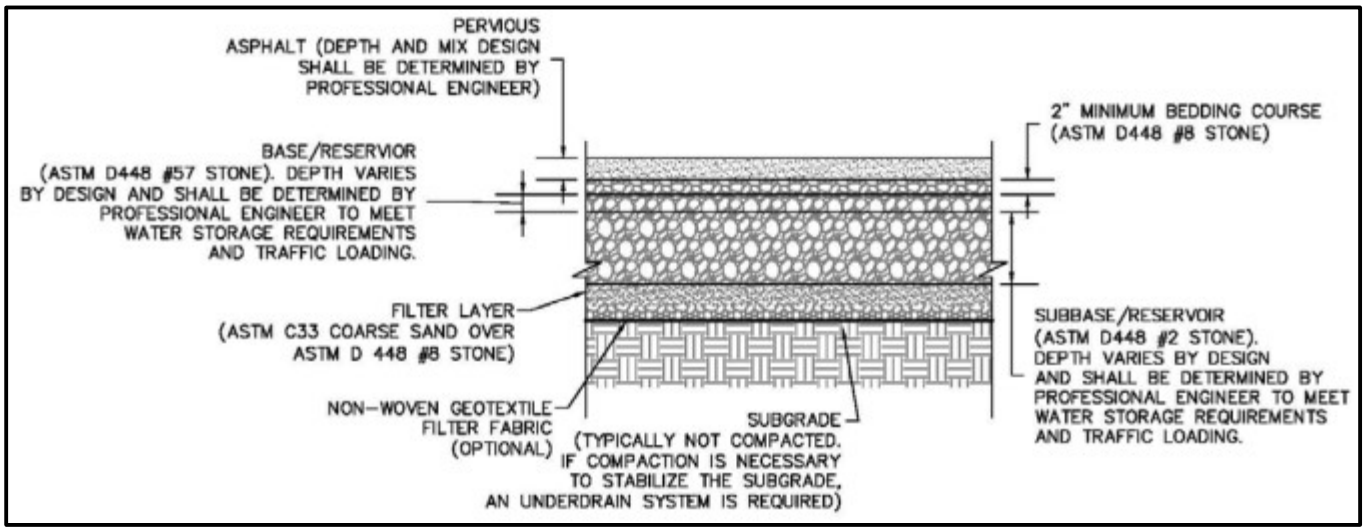


Figure C-58. Typical permeable asphalt pavement section (Source: FTN Associates, Ltd).

Table C-23. Modular Block Pavement System

Typical Cross Section:	
Pavement thickness:	Varies by product
Joint Material:	Pervious material, Sand or No. 8 stone gravel
Bedding Layer (Choker Course):	2 inches No. 8 stone or sand with geotextile fabric
Reservoir Layer:	3 inches to 4 inches No. 57 stone 12 inches to 18 inches No. 2 stone (Depth of layers shall be determined by Professional Engineer)
Filter Layer:	6 inches to 8 inches coarse sand (ASTM C33) on top of 2 inches to 4 inches No. 8 stone
Geotextile Fabric:	4-oz minimum Non-Woven filter fabric

References: Interlocking Concrete Pavement Institute (ICPI), “Permeable Interlocking Concrete Pavements”, Fourth Edition. Refer to Manufacturer specifications for proprietary modular block systems.

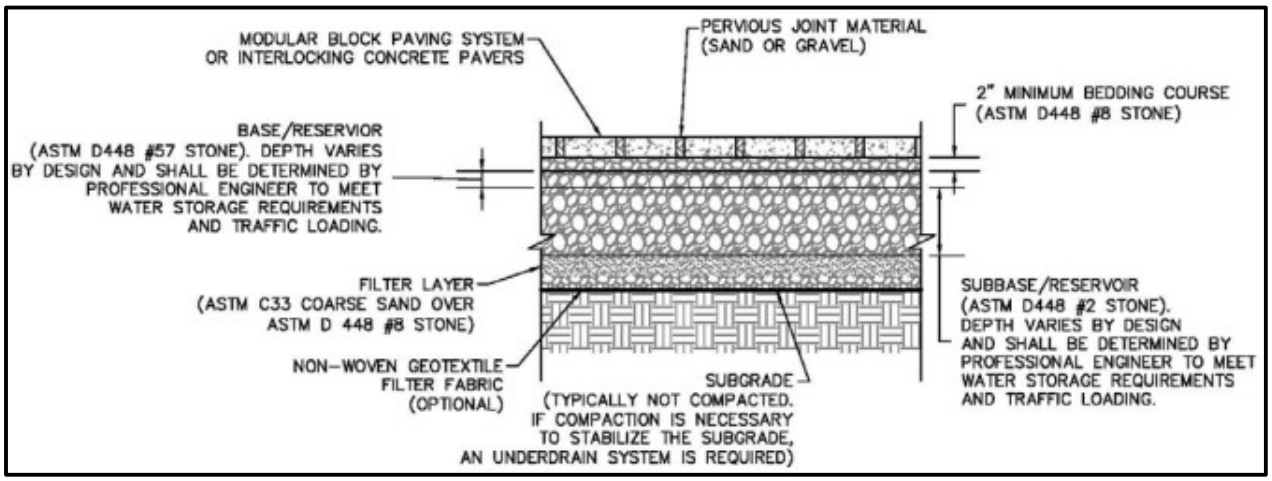


Figure C-59. Typical modular block pavement section (Source: FTN Associates, Ltd).

Table C-24. Reinforced grass pavement system.

Typical Cross Section:	
Pavement thickness:	Varies by product
Joint Material:	Fill cells with sandy soil growing medium (60% minimum sand)
Bedding Layer (Choker Course):	2 inches No. 8 stone
Reservoir Layer:	12 inches to 18 inches No. 57 or No. 67 stone (Depth of layers shall be determined by Professional Engineer)
Filter Layer:	6 inches to 8 inches coarse sand (ASTM C33) on top of 2 inches to 4 inches No. 8 stone
Geotextile Fabric:	4-oz minimum Non-Woven filter fabric

References: Refer to Manufacturer specifications for proprietary systems.

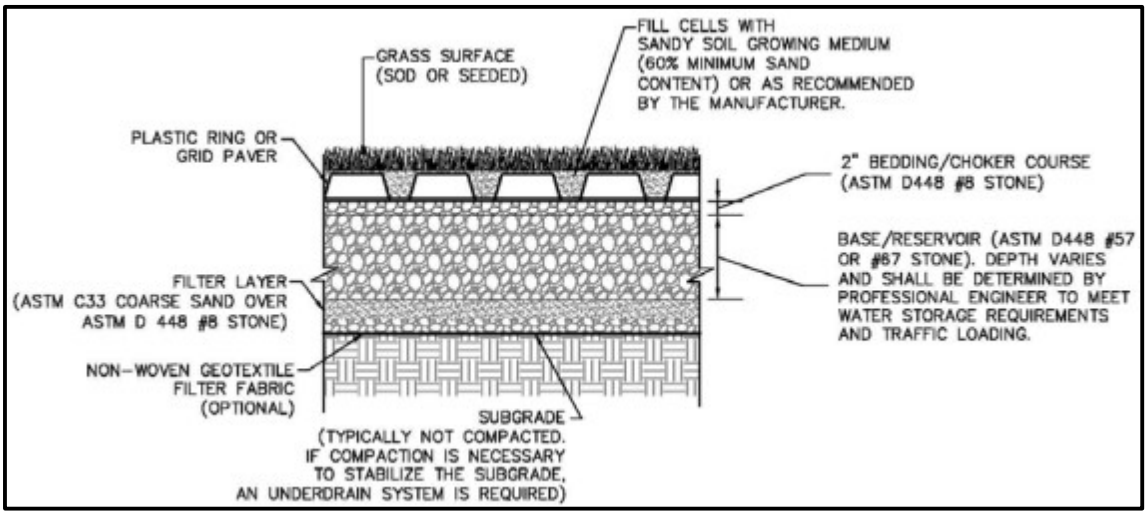


Figure C-60. Typical grass paver section (Source: FTN Associates, Ltd).

Table C-25. Reinforced gravel pavement System

Typical Cross Section:	
Pavement thickness:	Varies by product
Joint Material:	Fill cells with washed gravel (No. 8 stone)
Reservoir Layer:	12 inches to 18 inches No. 57 or No. 67 stone (Depth of layers shall be determined by Professional Engineer)
Filter Layer:	6 inches to 8 inches coarse sand (ASTM C33) on top of 2 inches to 4 inches No. 8 stone
Geotextile Fabric:	4-oz minimum Non-Woven filter fabric

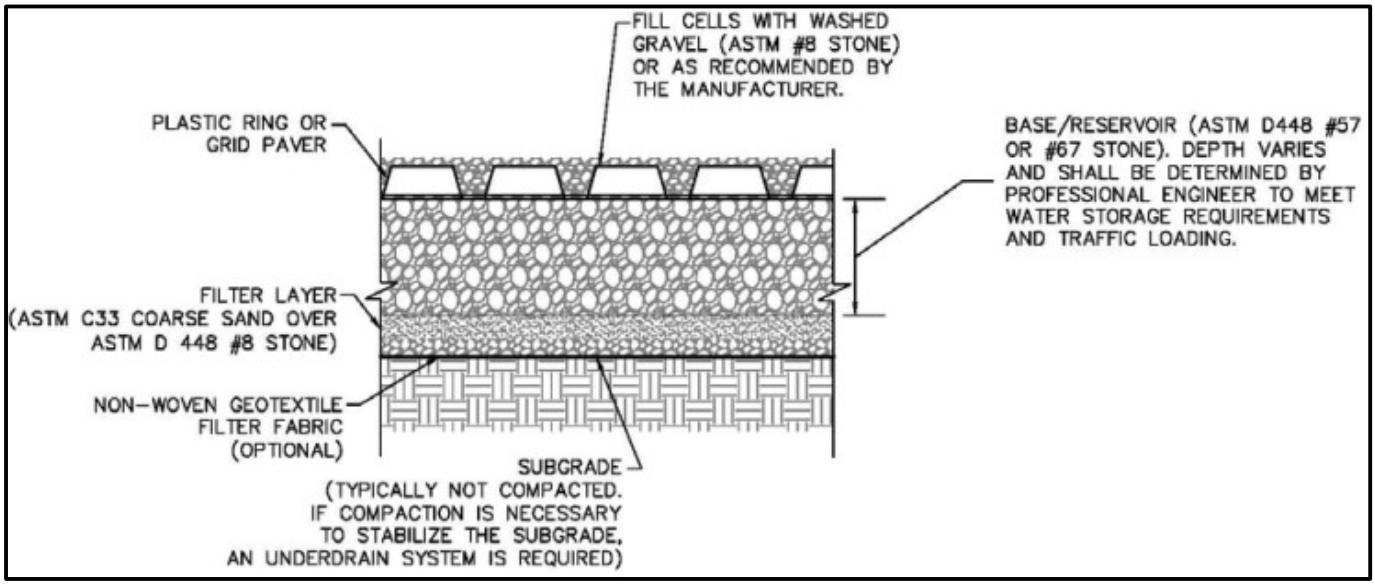


Figure C-61. Typical reinforced gravel pavement section (Source: FTN Associates, Ltd).

Figure C-62 shows a typical underdrain. Refer to Section 6.1 of this specification to determine the flow within the underdrain. Refer to Section 6.8 of this specification for details regarding the design and location of underdrains. If the pavement surface is sloped, the pavement subbase shall be terraced in cells with underdrains that collect stormwater from each cell. The subbase slope shall not exceed 1%. Figure C-63 shows a typical installation of a terraced subbase on a sloped pavement.

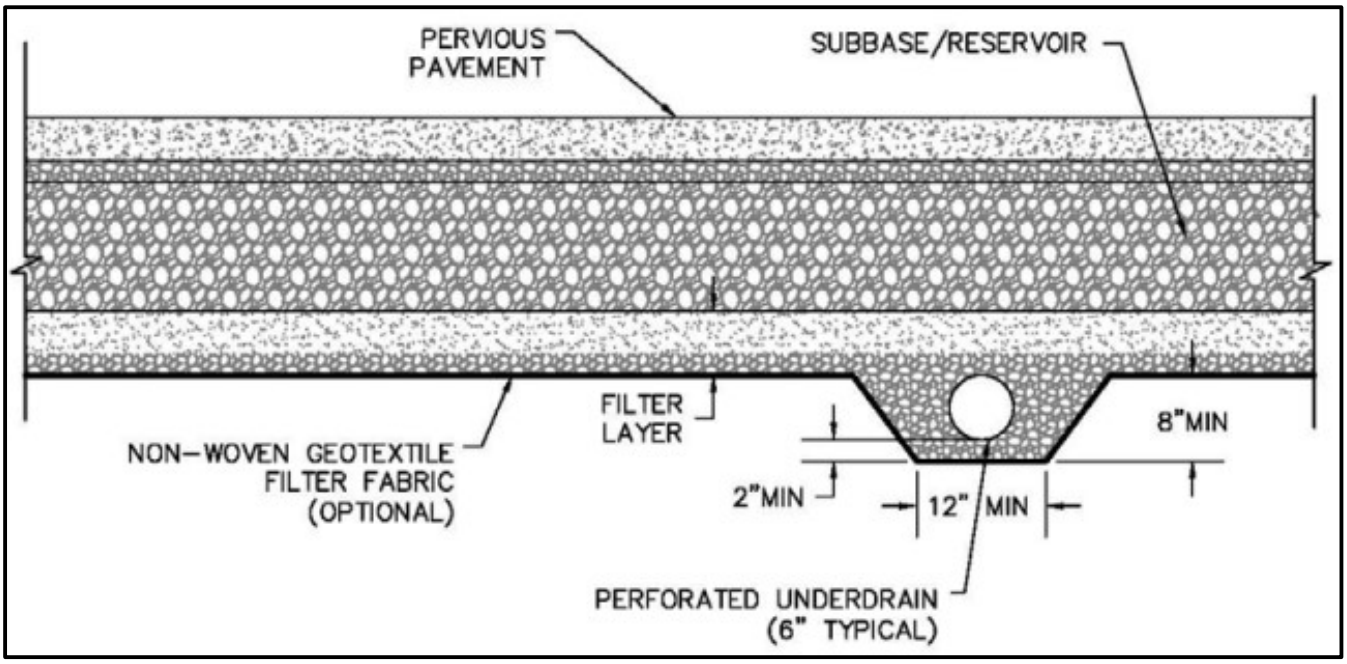


Figure C-62. Typical underdrain detail (Source: FTN Associates, Ltd).

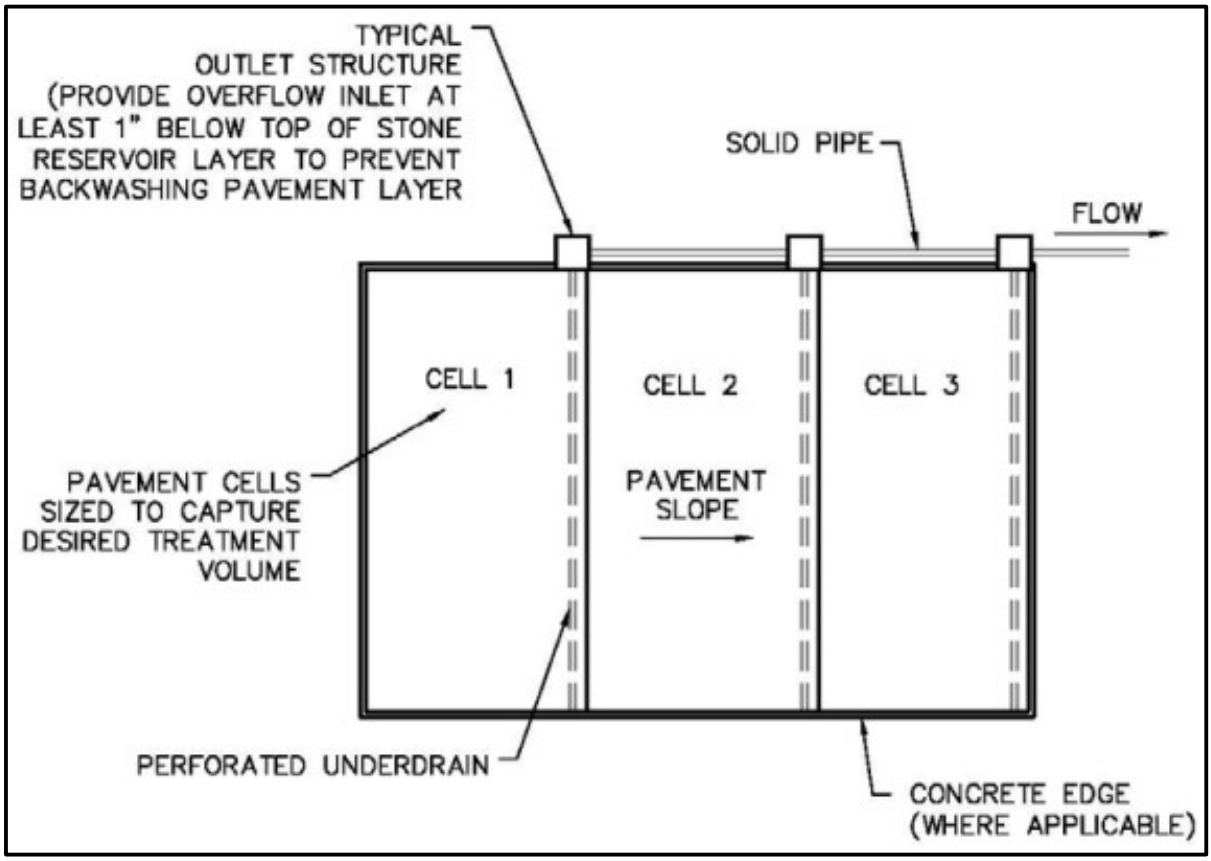


Figure C-63. Typical sloped pavement underdrain layout (Source: UDFCD 2010).

3. Physical Feasibility and Design Applications

Since permeable pavement has a very high runoff reduction capability, it should generally be considered as an alternative to conventional pavement. Permeable pavement is subject to the same feasibility constraints as most infiltration practices, as described below.

Available Space. A prime advantage of permeable pavement is that it does not normally require additional space at a new development or redevelopment site, which can be important for tight sites or areas where land prices are high.

Soils. Soil conditions do not constrain the use of permeable pavement, although they do determine whether an underdrain is needed. Impermeable soils in Hydrologic Soil Groups (HSG) C or D usually require an underdrain, whereas HSG A and B soils often do not. In addition, permeable pavement above fill soils shall be designed with an impermeable liner and underdrain.

If the proposed permeable pavement area is designed to infiltrate runoff without underdrains, it must have a minimum infiltration rate of 0.5 inches per hour. Initially, projected soil infiltration rates can be estimated from USDA-NRCS soil data, but they must be confirmed by on-site infiltration measurements.

> ***Designers shall evaluate existing soil properties during initial site layout and seek to configure the site to conserve and protect the soils with the greatest recharge and infiltration rates. Areas of HSG A or B soils shown on NRCS soil surveys should be considered as primary locations for all types of infiltration.***

External Drainage Area. Any external drainage area contributing runoff to permeable pavement shall generally not exceed twice the surface area of the permeable pavement (for Level 1 design), and it shall be as close to 100% impervious as possible. Some field experience has shown that an upgradient drainage area (even if it is impervious) can contribute particulates to the permeable pavement and should not lead to clogging (Hirschman, et al., 2009). Therefore, careful sediment source control and/or a pre-treatment strip or sump (e.g., stone or gravel) shall be used to control sediment run-on to the permeable pavement section.

Pavement Slope. Steep slopes can reduce the stormwater storage capability of permeable pavement and may cause shifting of the pavement surface and base materials. Designers should consider using a terraced design for permeable pavement in sloped areas. The maximum pavement surface slope is 3%. However, the bottom of the subbase slope shall be as flat as possible (0% to 1.0% longitudinal slope) to enable even distribution and infiltration of stormwater. An underdrain system shall be employed to collect stormwater that does not infiltrate if the bottom of the subbase is sloped. Lateral slopes shall be 0%.

Minimum Hydraulic Head. The elevation difference needed for permeable pavement to function properly is generally nominal, although 2 to 4 feet of head may be needed to drive flows through underdrains. Flat terrain may affect proper drainage of Level 1 permeable pavement designs, so underdrains shall have a minimum 0.5% slope.

Minimum Depth to Water Table. A high groundwater table may cause ponding at the bottom of the permeable pavement system. Therefore, a minimum vertical distance of 2 feet above the bottom of the permeable pavement installation (i.e., the bottom invert of the reservoir layer) and the seasonal high-water table.

Setbacks. To prevent seepage, permeable pavement shall not be hydraulically connected to structure foundations. Setbacks to structures and roads vary, based on the scale of the permeable pavement installation (see Table C-26 below). At a minimum, small- and large-scale pavement applications shall be located a minimum horizontal distance of 100 feet from any water supply well, 50 feet from septic systems, and at least 5 feet down-gradient from dry or wet utility lines.

Informed Owner. The property owner should clearly understand the unique maintenance responsibilities inherent with permeable pavement, particularly for parking lot applications. The owner should be capable of performing periodic

routine and long-term actions (e.g., vacuum sweeping) to maintain the pavement's hydrologic functions, and avoid future practices (e.g., winter sanding, seal coating or repaving) that diminish or eliminate them.

High Loading Situations. Permeable pavement is not intended to treat sites with high sediment or trash/debris loads, since such loads will cause the practice to clog and fail.

Groundwater Protection. Section 10 of the Bioretention specification (WQ-04) presents a list of potential areas with high pollutant loading that pose a risk of groundwater contamination. Infiltration of runoff from designated areas with high pollutant loading is highly restricted or prohibited.

Limitations. Permeable pavement can be used as an alternative to most types of conventional pavement at residential, commercial, and institutional developments; however, it is not currently approved for use within the Right of Way (ROW).

Design Scales. Permeable pavement can be installed at the following three scales:

1. The smallest scale is termed Micro-Scale Pavements, which applies to converting impervious surfaces to permeable ones on small lots and redevelopment projects, where the installations may range from 250 to 1000 square feet in total area. Where redevelopment or retrofitting of existing impervious areas results in a larger footprint of permeable pavers (small-scale or large- scale, as described below), the designer shall implement the Load Bearing, Observation Well, Underdrain, Soil Test, and Building Setback criteria associated with the applicable scale.
2. Small-scale pavement applications treat portions of a site between 1,000 and 10,000 square feet in area and include areas that only occasionally receive heavy vehicular traffic.
3. Large scale pavement applications exceed 10,000 square feet in area and typically are installed within portions of a parking lot.

Table C-26 outlines the different design requirements for each of the three scales of permeable pavement installation.

Table C-26. The Three Design Scales for Permeable Pavement

Design Factor	Micro-Scale Pavement	Small-Scale Pavement	Large-Scale Pavement
Impervious Area Treated	250 to 1000 sq ft	1000 to 10,000 sq ft	More than 10,000 sq ft
Typical Applications	Driveways, Walkways, Courtyards Plazas Individual Sidewalks	Sidewalk Network, Fire Lanes Road Shoulders, Spill-Over Parking, Plazas	Parking Lots with more than 40 spaces Low Speed Residential Streets
Load Bearing Capacity	Foot traffic Light vehicles	Light vehicles	Heavy vehicles (moving & parked)
Reservoir Size	Infiltrate or detain some or all the WQv	Infiltrate or detain the full WQv pavement area	
External Drainage Area?	No	Impervious cover up to twice pavement area with Level 1 design.	Impervious cover up to twice pavement area with Level 1 design.
Observation Well	No	No	Yes
Underdrain?	Depends on soil	Depends on soil	Back-up underdrain
Required Soil Tests	One per practice	Two per practice	One per 5000 sq ft of proposed practice
Building Setbacks	5 ft[1] 25 ft[2]	10 ft[1] 50 ft[2]	25 ft[1] 100 ft[2]

1.Minimum setback required for features down-gradient of structure

2.Minimum setback required for features up-gradient

Regardless of the design scale of the permeable pavement installation, the designer shall carefully consider the expected traffic load at the proposed site and the consequent structural requirements of the pavement system. Sites with heavy traffic loads will require a thick aggregate base. In contrast, most micro-scale applications should have little or no traffic flow to contend with.

4. Design Criteria

4.1 Sizing of Permeable Pavement

Structural Design. If permeable pavement will be used in a parking lot or other setting that involves vehicles, the pavement section must be able to support the maximum anticipated traffic load. The structural design process will vary according to the type of pavement selected, and the manufacturer's specific recommendations should be consulted. The thickness of the permeable pavement and reservoir layer must be sized to support loading and to temporarily store the design storm volume (e.g., the water quality, channel protection, and/or flood control volumes). On most new development and redevelopment sites, the structural support requirements will dictate the depth of the underlying stone reservoir.

The structural design of permeable pavements involves consideration of four main site elements:

- Total traffic
- In-situ soil strength characteristics based on design maximum water loads
- Environmental elements, and
- Bedding and Reservoir layer design

The resulting structural requirements may include, but are not limited to, the thickness of the pavement, filter, and reservoir layer. Designers should note that if the underlying soils have a low California Bearing Ratio (CBR), they may need to be compacted to at least 95% of the Standard Proctor Density, which rules out their use for infiltration.

Designers shall determine structural design requirements by consulting transportation design guidance sources, such as the following:

- City of Little Rock Roads Master Plan, Minimum Street Standards
- AASHTO Guide for Design of Pavement Structures (1993), and
- AASHTO Supplement to the Guide for Design of Pavement Structures (1998)

Hydraulic Design. Permeable pavement is typically sized to store the complete Water Quality Volume (WQv) or another design storm volume in the reservoir layer. Modeling indicates that this simplified sizing rule approximates an 80% average annual rainfall volume removal for subsurface soil infiltration rates up to one inch per hour. More conservative values are established for this practice as experience has shown that clogging of the permeable material can be an issue long-term, especially with larger contributing areas carrying significant soil materials onto the permeable surface.

The sub-surface soil infiltration rate typically will be less than the flow rate through the pavement, so that some underground reservoir storage will usually be required. Designers shall initially assume that there is no outflow through underdrains, using Equation C-13 to determine the depth of the reservoir layer, assuming runoff fully infiltrates into the underlying soil:

$$d_p = \frac{\left[(d_c \times R) + P - \left(i/2 \times t_f\right)\right]}{V_r} \qquad \textbf{Eq. C-13}$$

Where: dp = The depth of the reservoir layer (ft)

dc = The depth of runoff from the contributing drainage area (not including the permeable paving surface) for the Water Quality Volume (WQv/Ac), or other design storm (feet)

R = Ac /Ap = The ratio of the contributing drainage area (Ac, not including the permeable paving surface) to the permeable pavement surface area (Ap).

P = The rainfall depth for the Water Quality Volume (1.05 inches), or other design storm (feet)
i = The field-verified infiltration rate for native soils (feet/day)

tf = The time to fill the reservoir layer (day) typically 2 hours or 0.083 day

Vr = The void ratio for the reservoir layer (0.35)

The maximum allowable depth of the reservoir layer is constrained by the maximum allowable drainage time, which is calculated using Equation C-14:

$$d_{p-max} = \frac{(i/2 \times t_d)}{V_r} \qquad \textbf{Eq. C-14}$$

Where: dp-max = The maximum depth of the reservoir layer (feet)

i = The field-verified infiltration rate for native soils (feet/day)

td = The maximum allowable time to drain the reservoir layer, typically 1 to 2 days

Vr = The void ratio for the reservoir layer (0.35)

The following design assumptions apply to Equations C-13 and C-14:

- The contributing drainage area (Ac) should not contain pervious areas.
- For conservative design, the native soil infiltration rate or the field-tested soil infiltration rate is divided by a factor of safety of 2. The minimum acceptable native soil infiltration rate is 0.5 inches/hour.
- The void ratio (Vr) of 0.35 shall be used in the equations for the stone reservoir.
- Maximum drain time for the reservoir layer shall be not less than 24 hours and no more than 48 hours.

If the depth of the reservoir layer is too great (i.e. dp exceeds dp-max), or the verified soil infiltration rate is less than 0.5 inches per hour, then the design method typically changes to account for underdrains. The storage volume in the pavements must account for the underlying infiltration rate and outflow through the underdrain. In this case, the design storm shall be routed through the pavement to accurately determine the required reservoir depth. Alternatively, the designer may use Equations C-15 through C-17 to approximate the depth of the reservoir layer for designs using underdrains.

Equation C-15 can be used to approximate the outflow rate from the underdrain. The hydraulic conductivity, k, of gravel media is very high (~17,000 ft/day). However, the permeable pavement reservoir layer will drain more slowly as the storage volume decreases (i.e. the hydraulic head decreases). To account for this change, a conservative permeability coefficient of 100 ft/day can be used to approximate the average underdrain outflow rate.

$$q_u = k \times m \qquad \textbf{Eq. C-15}$$

Where: q_u = Outflow through the underdrain (per outlet pipe, assumed 6-inch diameter) (feet/day)

k = Hydraulic conductivity for the reservoir layer (ft/day – assume 100 feet/day)

m = Underdrain pipe slope (feet/foot)

Once the outflow rate through the underdrain has been approximated, Equation C-16 is used to determine the depth of the reservoir layer needed to store the design storm.

$$d_p = \frac{[(d_c \times R) + P - (i/2 \times t_f) - (q_u \times t_f)]}{V_r}$$ **Eq. C-16**

Where: d_p = Depth of the reservoir layer (feet)

d_c = Depth of runoff from the contributing drainage area (not including the permeable pavement surface) for the Water Quality Volume (WQv/Ac), or other design storm (feet)

R = Ac/Ap = The ratio of the contributing drainage area (Aj (not including the permeable pavement surface) to the permeable pavement surface area (Ap)

P = The rainfall depth for the Water Quality Volume (1.05”), or other design storm (feet)

i = The field-verified infiltration rate for the native soils (feet/day)

t_f = The time to fill the reservoir layer (day) – typically 2 hours or 0.083 day

V_r = The void ratio for the reservoir layer (0.35)

q_u = Outflow through Underdrain (feet/day)

The maximum allowable depth of the reservoir layer is constrained by the maximum allowable drain time, which is calculated using Equation C-17.

$$d_{p-max} = \frac{[(i/2 \times t_d) - (q_u \times t_d)]}{V_r}$$ **Eq. C-17**

Where: dp-max = The maximum depth of the reservoir layer (feet)

i = The field-verified infiltration rate for the native soils (feet/day)

Vr = The void ratio for the reservoir layer (0.35)

td = The time to drain the reservoir layer (day – typically 1 to 2 days)

qu = Outflow through Underdrain (feet/day)

If the depth of the reservoir layer is still too great (i.e. dp exceeds dp-max), the number of underdrains can be increased, which will increase the total outflow rate. Permeable pavement can also be designed to address, in whole or in part, the detention storage needed to comply with channel protection and/or flood control requirements. The designer can model various approaches by factoring in storage within the stone aggregate layer, expected infiltration, and any outlet structures used as part of the design. Routing calculations properly used can also provide a more accurate solution of the peak discharge and required storage volume.

Once runoff passes through the surface of the permeable pavement system, designers shall calculate outflow pathways to handle subsurface flows. Subsurface flows can be regulated using underdrains, the volume of storage in the reservoir layer, the bed slope of the reservoir layer, a control structure at the outlet, and/or appropriately designed natural filter layers.

4.2 Soil Infiltration Rate Testing

To design a permeable pavement system without an underdrain, the measured infiltration rate of subsoils must be 0.5 inch per hour or greater and the design must accommodate the 1-to-2-day reservoir drain time. On-site soil infiltration rate testing procedures are outlined in Appendix C. Alternately, a double ring infiltrometer test may be used. Test quantities shall be based on Table C-26. In most cases, a single soil test is sufficient for micro-scale and small-scale applications. At least one soil boring must be taken to confirm the underlying soil properties at the depth where infiltration

is designed to occur (i.e., to ensure that depth to water table or depth to bedrock is defined). Soil infiltration testing shall be conducted above a confining layer, if found, within 4 feet of the bottom of a proposed permeable pavement system.

4.3 Type of Surface Pavement

The type of pavement shall be selected based on a review of the factors described in Sections 1 and 2, the project objectives, and designed according to the product manufacturer's recommendations.

4.4 Internal Geometry and Drawdowns

- ***Elevated Underdrain.*** To promote greater runoff reduction for permeable pavement located on lower filtration subgrade soils, an elevated underdrain shall be installed with a stone jacket that creates a 12- to 18-inch-deep storage layer below the underdrain invert. The void storage in this layer can help qualify a site to achieve Level 2 design.
- ***Rapid Drawdown.*** When possible, permeable pavement shall be designed so that the target runoff reduction volume stays in the reservoir layer for at least 24 hours before being discharged through an underdrain.
- ***Conservative Infiltration Rates.*** Designers shall always decrease the measured infiltration rate by a factor of 2 during design, to approximate long term infiltration rates.

4.5 Pretreatment

Pretreatment for most permeable pavement applications is not necessary since the surface acts as pretreatment to the reservoir layer below. Additional pretreatment may be appropriate if the pavement receives run-on from an adjacent pervious or impervious area.

4.6 Conveyance and Overflow

Permeable pavement designs may include methods to convey larger storms (e.g., 50- and 10-percent annual chance) to the storm drain system. The following is a list of methods that can be used to accomplish this:

- Place a perforated pipe horizontally near the top of the reservoir layer to pass excess flows after water has filled the base. The placement and/or design shall be such that the incoming runoff is not captured (e.g., placing the perforations on the underside only).
- Increase the thickness of the top of the reservoir layer by as much as 6 inches over initial design requirement (i.e., create freeboard). The design computations used to size the reservoir layer often assume that no freeboard is present.
- Create underground detention within the reservoir layer of the permeable pavement system. Reservoir storage may be augmented by corrugated metal pipes, plastic, or concrete arch structures, etc.
- Route excess flows to another detention or conveyance system that is designed for the management of discharges due to larger storms.
- Set the storm drain inlets flush with the elevation of the permeable pavement surface to effectively convey excess stormwater runoff past the system (typically in remote areas). The design shall also make allowances for outlet structures or overflows to prevent unacceptable ponding depths during larger rainfall events.

4.7 Reservoir layer

The thickness of the reservoir layer is determined by runoff storage needs, the infiltration rate of in situ soils, structural requirements of the pavement sub-base, depth to water table and bedrock, and frost depth conditions. The engineer shall inspect subgrade for suitability.

- The reservoir below the permeable pavement surface shall be composed of clean, washed stone aggregate and sized for both the storm event to be treated and the structural requirements of the expected traffic loading.
- The storage layer may consist of clean washed No. 57 stone, although No. 2 stone is preferred because it provides additional storage and structural stability.
- The bottom of the reservoir layer shall be horizontal to allow runoff to infiltrate evenly through the entire surface.

4.8 Underdrains

The use of underdrains is recommended when there is a reasonable potential for infiltration rates to decrease over time, when underlying soils have an infiltration rate of less than 0.5 inch per hour, when shallow bedrock is present, or when subgrade soils must be compacted to achieve a desired Proctor density. Underdrains can also be used to manage extreme storm events to keep detained stormwater from backing up above the permeable pavement.

- Underdrains will be required in most locations within the City of Little Rock since the majority of existing soils are HSG type C or D.
- Underdrains shall be 4 to 6-inch perforated pipe and shall be placed to collect excess stormwater that cannot be infiltrated into the ground within the allowable reservoir drain time.
- The number of underdrains required depends upon the size of the pavement area, the depth of the storage reservoir needed, the slope of pavement section, and the infiltration rate of the underlying soils.
- Underdrains shall be placed within the reservoir and encased by 8 to 12 inches of clean, washed stone as measured from the outside of the underdrain to the outer edge of the encasing layer.
- The underdrain outlet can be fitted with a flow-reduction orifice as a means of regulating the stormwater detention time. The minimum diameter of any orifice shall be 0.5 inch. Orifices must be inspected regularly to ensure the system is functioning properly.
- Underdrains can also be installed and capped at the downstream outlet as an option for future use if maintenance observations indicate a reduction in the soil permeability.

4.9 Maintenance Reduction Features

Maintenance is a crucial element to ensure the long-term performance of permeable pavement. The most frequently cited maintenance problem is surface clogging caused by organic matter and sediment, which can be reduced by the following measures:

- ***Periodic Vacuum Sweeping.*** The pavement surface is the first line of defense in trapping and eliminating sediment that may otherwise enter the stone base and soil subgrade. The rate of sediment deposition shall be monitored, and vacuum sweeping done once or twice a year. This frequency shall be adjusted according to the intensity of use and deposition rate on the permeable pavement surface. At least one sweeping pass shall occur at the end of winter.
- ***Protecting the Bottom of the Reservoir Layer.*** There are two options to protect the bottom of the reservoir layer from intrusion by underlying soils. The first method involves covering the bottom with nonwoven, polypropylene geotextile that is permeable, although some practitioners recommend avoiding the use of filter fabric since it may become a future plane of clogging within the system. Permeable filter fabric is still recommended to protect the excavated sides of the reservoir layer, to prevent soil piping. The second design method is to form a natural filter barrier of choker stone and sand. In this case, underlying native soils shall be separated from the reservoir base/subgrade layer by one or more filter layers of sand and gravel based on gradation sizing of the subgrade and subsequent filter layers to prevent piping and undermining of the subgrade soils.
- ***Observation Well.*** An observation well, consisting of a well-anchored, perforated 2-to-4 inch (diameter) slotted PVC pipe that extends vertically to the bottom of the reservoir layer, shall be installed at the downstream end of all large-scale permeable pavement systems. The observation well shall be fitted with a lockable cap installed flush with the ground surface (or under the pavers) to facilitate periodic inspection and maintenance. The observation well is used to observe the rate of drawdown within the reservoir layer following storm events.
- ***Overhead Landscaping***. Plan and design landscaping to account for permeable pavement maintenance limitations. Large-scale permeable pavement applications shall be carefully planned to integrate this landscaping in a manner that maximizes runoff treatment and minimizes the risk that sediment, mulch, grass clippings, leaves, nuts, and fruits will clog the paving surface.

4.10 Material Specifications

Permeable pavement material specifications vary according to the specific pavement product selected. Table C-27 describes general material specifications for the component structures installed beneath the permeable pavement. Table -28 provides specifications for general categories of permeable pavements. Designers shall consult manufacturer's technical specifications for specific criteria and guidance.

Table C-27. Material specifications for underneath the pavement surface.

Material	Specification	Notes
Bedding Layer	**Pervious Concrete, Asphalt, and Interlocking pavers**: 2 inches of No. 8 stone over 3 to 4 inches No. 57 **Reinforced Grass Pavers:** 2 inches No. 8 stone **Reinforced Gravel Pavers:** 2 inches No. 8 stone (fill pavers)	ASTM D448 size No. 8 stone (e.g., 3/8 to 3/16 inch in size). Shall be double-washed and clean and free of all fines.
Reservoir Layer	**Pervious Concrete, Asphalt, and Interlocking pavers:** No. 2 stone **Reinforced Grass Pavers**: No. 57 Stone	ASTM D448 size No. 57 stone (e.g., 1 1/2 to ½ inch in size); No. 2 Stone (e.g., 2-1/2 in. to 3/4 in. in size). Depth is based on the pavement structural and hydraulic requirements. Shall be double-washed, clean, and free of all fines.
Underdrain	Use 4- to 6-inch diameter perforated HDPE or PVC (AASHTO M 252) pipe, with maximum 3/8-inch perforations at 6 inches on center; each underdrain installed at a minimum 0.5% slope spaced 20 ft on centers (equivalent corrugated HDPE may be used for smaller load-bearing applications). Perforated pipe installed for the full length of the permeable pavement cell, and non-perforated pipe, as needed, is used to connect with the storm drain system in accordance with Fayetteville Drainage Manual. T's and Y's installed as needed, depending on the underdrain configuration. Extend cleanout pipes to the surface with vented caps at Ts and Ys.	
Filter Layer	The underlying native soils shall be separated from the stone reservoir by a thin, 2 to 4 inches layer of choker stone (e.g., No. 8) covered by a 6 to 8 inches layer of coarse sand (e.g., ASTM C 33, 0.02-0.04 inches).	The sand shall be placed between the stone reservoir and the choker stone, which shall be placed on top of the underlying native soils.
Filter Fabric (optional)	Use a needled, non-woven, polypropylene geotextile with Grab Tensile Strength equal to or greater than 120 pounds (ASTM D4632), with a Mullen Burst Strength equal to or greater than 225 pounds/square inch (ASTM D3786), with a Flow Rate greater than 125 gpm/sq ft (ASTM D4491), and an Apparent Opening Size (AOS) equivalent to a US # 70 or # 80 sieve (ASTM D4751). The geotextile AOS selection is based on the percent passing the No. 200 sieve in "A" Soil subgrade, using FHWA or AASHTO selection criteria.	

Table C-28. Additional Permeable Pavement Specifications

Material	Specification	Notes
Permeable Interlocking Concrete Pavers	**Surface open area:** 5% to 15%. **Thickness:** 3.125 inches for vehicles. **Compressive strength:** 55 Mpa (~8000 psi). **Open void fill media:** aggregate	Must conform to ASTM C936 specifications. Reservoir layer required to support the structural load.
Concrete Grid Pavers Reservoir Layer	**Open void content:** 20% to 50%. **Thickness:** 3.5 inches. **Compressive strength:** 35 Mpa(~5000 psi). **Open void fill media:** aggregate, topsoil and grass, coarse sand.	Must conform to ASTM C 1319 specifications. Reservoir layer required to support the structural load.
Plastic Reinforced Grid Pavers	**Void content:** depends on fill material. **Compressive strength:** varies, depending on fill material. **Open void fill media:** aggregate, topsoil and grass, coarse sand.	Reservoir layer required to support the structural load.
Impermeable Liner	Use a thirty mil (minimum) PVC Geomembrane liner covered by 8 to 12 oz./sq. yd. non-woven geotextile (both sides)	
Observation Well	Use a perforated or slotted 2 to 4 in. vertical PVC pipe (AASHTO M 252) with a lockable cap, installed flush with the surface.	

5. Special Case Design Adaptations

The design adaptations described below permit permeable pavement to be used on a wider range of sites. However, it is important not to force this practice onto marginal sites. Other runoff reduction practices are often preferred alternatives for difficult sites.

Shallow Bedrock

Underdrains must be used in locations in which bedrock is encountered less than 2 feet beneath the planned invert of the reservoir layer.

6. Construction

Experience has shown that proper installation is critical to the effective operation of a permeable pavement system. Installation shall be overseen by a qualified engineer or engineer's representative.

6.1 Necessary Erosion & Sediment Controls

- All permeable pavement areas shall be fully protected from sediment intrusion by silt fence or construction fencing, particularly if they are intended to infiltrate runoff.
- Permeable pavement areas shall remain outside the limit of disturbance during construction to prevent soil compaction by heavy equipment. Permeable pavement areas shall be clearly marked on all construction documents and grading plans. To prevent soil compaction, heavy vehicular and foot traffic shall be kept out of permeable pavement areas during and immediately after construction.
- During construction, care shall be taken to avoid tracking sediments onto any permeable pavement surface to avoid clogging.
- Any area of the site intended ultimately to be a permeable pavement area shall not be used as the site of a temporary sediment basin.
- Where locating a sediment basin on an area intended for permeable pavement is unavoidable, the invert of the sediment basin must be a minimum of 2 feet above the final design elevation of the bottom of the aggregate reservoir course.

- All sediment deposits in the excavated area shall be carefully removed prior to completion of the excavation and installation of the subbase, base, and surface materials.

6.2 Permeable Pavement Construction Sequence

The following is a typical construction sequence to properly install permeable pavement:

Step 1. Construction of the permeable pavement shall only (or) begin after the entire contributing drainage area has been stabilized. The proposed site shall be checked for existing utilities prior to any excavation. Do not install the system in rain or snow, and do not install frozen bedding materials.

Step 2. As noted above, temporary erosion and sediment (E&S) controls are needed during installation to divert stormwater away from the permeable pavement area until it is completed. Special protection measures such as erosion control fabrics may be needed to protect vulnerable side slopes from erosion during the excavation process. The proposed permeable pavement area must be kept free from sediment during the entire construction process. Construction materials that are contaminated by sediments must be removed and replaced with clean materials.

Step 3. Excavators or backhoes shall work from the sides to excavate the reservoir layer to its appropriate design depth and dimensions. For micro-scale and small-scale pavement applications, excavating equipment shall have arms with adequate extension so they do not have to work inside the footprint of the permeable pavement area (to avoid compaction). Contractors can utilize a cell construction approach, whereby the proposed permeable pavement area is split into 500 to 1000 sq ft temporary cells with a 10-to-15-foot earth bridge in between, so that cells can be excavated from the side. Excavated material shall be placed away from the open excavation to not jeopardize the stability of the side walls.

Step 4. The native soils along the bottom and sides of the permeable pavement system shall be scarified or tilled to a depth of 3 to 4 inches prior to the placement of the filter layer or filter fabric. In large scale paving applications with weak soils, the soil subgrade may need to be compacted to 95% of the Standard Proctor Density to achieve the desired load-bearing capacity. (NOTE: This improves strength characteristics and significantly reduces the permeability of the subgrade, and it will typically require installation of an underdrain. The reduced infiltration characteristics must be addressed during hydrologic design.)

Step 5. Filter fabric shall be installed on the bottom and the sides of the reservoir layer. Filter fabric strips shall overlap down-slope by a minimum of 2 feet and be secured a minimum of 4 feet beyond the edge of the excavation. Where the filter layer extends beyond the edge of the pavement (to convey runoff to the reservoir layer), install an additional layer of filter fabric 1 foot below the surface to prevent sediments from entering the reservoir layer. Excess filter fabric shall not be trimmed until the site is fully stabilized.

Step 6. Provide a minimum of 2 inches of aggregate above and below the underdrains. The underdrains shall slope down towards the outlet at a grade of 0.5% or steeper. The up-gradient end of underdrains in the reservoir layer shall be capped. Where an underdrain pipe is connected to a structure, there shall be no perforations within 1 foot of the structure. Ensure that there are no perforations in clean-outs and observation wells within 1 foot of the surface.

Step 7. Moisten and spread 6-inch lifts of the appropriate clean, washed stone aggregate (usually No. 2 or No. 57 stone). Place at least 4 inches of additional aggregate above the underdrain, and then compact it using a vibratory roller in static mode until there is no visible movement of the aggregate. Do not crush the aggregate with the roller.

Step 8. Install the desired depth of the bedding layer, depending on the type of pavement, as follows:

- ***Pervious Concrete, Asphalt, and Interlocking Pavers:*** The bedding layer for porous asphalt pavement consists of 2 inches of clean, washed ASTM D 448 No.8 stone above 3 to 4 inches of ASTM D 448 No. 57 stone. The filter course must be leveled and pressed (choked) into the reservoir base with at least four (4) passes of a 10-ton steel drum static roller. For interlocking pavers, the thickness of the bedding layer is to be based on the block manufacturer's recommendation or a minimum of 2 inches whichever is greater.

- ***Reinforced Grass and Gravel Pavers:*** The bedding layer for reinforced grass and gravel pavers pavement consists of 2 inches of clean, washed ASTM D 448 No.8 stone.

Step 9. Paving materials shall be installed in accordance with manufacturer or industry specifications for the pavement.

- **Installation of Porous Asphalt.** The following has been excerpted from various documents, most notably Jackson (2007).
    - Install porous asphalt pavement similarly to regular asphalt pavement. The pavement shall be laid in a single lift over the filter course. The laying temperature shall be between 230°F and 260°F, with a minimum air temperature of 50°F, to ensure that the surface does not stiffen before compaction.
    - Complete compaction of the surface course when the surface is cool enough to resist a 10-ton roller. One or two passes of the roller are required for proper compaction. More rolling could cause a reduction in the porosity of the pavement.
    - The mixing plant must provide certification of the aggregate mix, abrasion loss factor, and asphalt content in the mix. Test the asphalt mix for its resistance to stripping by water using ASTM 1664. If the estimated coating area is not above 95%, additional antistripping agents must be added to the mix.
    - Transport the mix to the site in a clean vehicle with smooth dump beds sprayed with a non- petroleum release agent. The mix shall be covered during transportation to control cooling. o Test the full permeability of the pavement surface by application of clean water at a rate of at least five gallons per minute over the entire surface. All water must infiltrate directly, without puddle formation or surface runoff.
    - Inspect the facility 18 to 30 hours after a significant rainfall (greater than 0.5 inch) or artificial flooding, to determine that the facility is draining properly.
- **Installation of Pervious Concrete.** The basic installation sequence for pervious concrete is outlined by the American Concrete Institute (2008). It is strongly recommended that concrete installers successfully complete a recognized pervious concrete installers training program, such as the Pervious Concrete Contractor Certification Program offered by the NRMCA. The basic installation procedure is as follows:
    - Drive the concrete truck as close to the project site as possible.
    - Water the underlying aggregate (reservoir layer) before the concrete is placed, so that the aggregate does not draw moisture from the freshly laid pervious concrete.
    - After the concrete is placed, approximately 3/8 to 1/2 inch is struck off, using a vibratory screed. This is to allow for compaction of the concrete pavement.
    - Compact the pavement with a steel pipe roller. Care shall be taken so that over compaction does not occur.
    - Cut joints for the concrete to a depth of 1/4 inch.
    - The curing process is very important for pervious concrete. Cover the pavement with plastic sheeting within 20 minutes of the strike-off, and keep it covered for at least seven (7) days. Do not allow traffic on the pavement during this time.
- **Installation of Interlocking Pavers.** The basic installation process is described in greater detail by Smith (2006).
    - Place edge restraints for open jointed pavement blocks before the bedding layer and pavement blocks are installed. Permeable interlocking concrete pavement (IP) systems require edge restraints to prevent vehicle loads from moving the paver blocks. Edge restraints may be standard City of Little Rock curbs or gutter pans, or precast or cast-in-place reinforced concrete borders a minimum of 6 inches wide and 18 inches deep, constructed with Class A3 concrete. Edge restraints along the traffic side of a permeable pavement block system are recommended.
    - Place the No. 57 stone in a single lift. Level the filter course and compact it into the reservoir course beneath with at least four (4) passes of a 10-ton steel drum static roller until there is no visible movement. The first two (2) passes are in vibratory mode, with the final two (2) passes in static mode. The filter aggregate shall be moist to facilitate movement into the reservoir course.

- Place and screed the bedding course material (typically No. 8 stone).
- Fill gaps at the edge of the paved areas with cut pavers or edge units. When cut pavers are needed, cut the pavers with a paver splitter or masonry saw. Cut pavers no smaller than one- third (1/3) of the full unit size.
- Pavers may be placed by hand or with mechanical installers. Fill the joints and openings with stone. Joint openings must be filled with No. 8 stone, although smaller stone may be used where needed to fill narrower joints. Remove excess stones from the paver surface.
- Compact and seat the pavers into the bedding course with a minimum low-amplitude 5,000-lbf, 75- to 95-Hz plate compactor.
- Do not compact within 6 feet of the unrestrained edges of the pavers.
- The system must be thoroughly swept by a mechanical sweeper or vacuumed immediately after construction to remove any sediment or excess aggregate.
- Inspect the area for settlement. Any blocks that settle must be reset and re-inspected.
- Inspect the facility 18 to 30 hours after a significant rainfall (0.5 inch or greater) or artificial flooding to determine whether the facility is draining properly.

- Installation of Plastic Grid Reinforcement for Grass and Gravel Pavement. The installation process is determined by the manufacturer of the plastic grid reinforcement product being installed.

6.3 Construction Inspection

Inspections before, during and after construction are needed to ensure that permeable pavement is built in accordance with these specifications. For a minimum, use detailed inspection checklists that require signoffs by qualified individuals at critical stages of construction, to ensure that the contractor's interpretation of the plan is consistent with the designer's intent. Some common pitfalls can be avoided by careful construction supervision that focuses on the following key aspects of permeable pavement installation:

- Store materials in a protected area to keep them free from mud, dirt, and other foreign materials.
- The contributing drainage area shall be stabilized prior to directing water to the permeable pavement area.
- Check the aggregate material to confirm that it is clean and washed, meets specifications, and is installed to the correct depth.
- Check elevations (e.g., the invert of the underdrain, inverts for the inflow and outflow points, etc.) and the surface slope.
- Ensure that caps are placed on the upstream (but not the downstream) ends of the underdrains.
- Make sure the permeable pavement surface is even, runoff evenly spreads across it, and the storage bed drains within 48 hours.
- Inspect the pretreatment structures (if applicable) to make sure they are properly installed and working effectively.
- Once the final construction inspection has been completed, log the GPS coordinates for each facility and submit them to City of Little Rock City Engineer.

7. Maintenance

7.1 Maintenance Plan

It is recommended that a Long-Term Maintenance Plan (LTMP) be developed by the design engineer. The LTMP contains a description of the stormwater system components and information on the required inspection and maintenance activities. The LTMP for permeable pavement shall also note which conventional parking lot maintenance tasks must be avoided (e.g., sanding, re-sealing, re-surfacing, power- washing). Signs shall be posted on larger parking lots to indicate their stormwater function and special maintenance requirements.

7.2 Maintenance Tasks

It is difficult to prescribe the specific types or frequency of maintenance tasks that are needed to maintain the hydrologic function of permeable pavement systems over time. Most installations work reasonably well year after year with little or

no maintenance, whereas some have problems. One preventive maintenance task for large-scale applications involves vacuum sweeping on a frequency consistent with the use and loadings encountered in the parking lot. An annual, dry weather sweeping in the spring months shall be performed. The contract for sweeping shall specify that a vacuum sweeper be used that does not use water spray, since spraying may lead to subsurface clogging. Vacuum settings for large- scale interlocking paver applications shall be calibrated so they do not pick up the stones between pavement blocks.

7.3 Maintenance Inspections

It is highly recommended that a spring maintenance inspection and cleanup be conducted at each permeable pavement site, particularly at large-scale applications. Maintenance of permeable pavement is driven by annual inspections that evaluate the condition and performance of the practice. The following are suggested annual maintenance inspection points for permeable pavements:

- The drawdown rate shall be measured at the observation well for three (3) days following a storm event more than 0.5 inch in depth. If standing water is still observed in the well after three days, this is a clear sign that clogging in the reservoir or underdrain is a problem. If standing water is on the pavement surface, surface clogging is the issue.
- Inspect the surface of the permeable pavement for evidence of sediment deposition, organic debris, staining or ponding that may indicate surface clogging. If any signs of clogging are noted, schedule a vacuum sweeper (no brooms or water spray) to remove deposited material. Vacuum settings for large-scale interlocking paver applications shall be calibrated so they do not pick up stones between pavement blocks. Then, test sections by pouring water from a five-gallon bucket to ensure they work.
- Inspect the structural integrity of the pavement surface, looking for signs of surface deterioration, such as slumping, cracking, spalling or broken pavers. Replace or repair affected areas, as necessary.
- Check inlets, pretreatment cells and any flow diversion structures for sediment buildup and structural damage. Note if any sediment needs to be removed.
- Inspect the condition of the observation well and make sure it is still capped.
- Generally, inspect any contributing drainage area for any controllable sources of sediment or erosion.

8. Community and Environmental Concerns

Compliance with the Americans with Disabilities Act (ADA). Interlocking concrete pavers are ADA compliant, if designers ensure that surface openings between pavers do not exceed 0.5 inch. However, some forms of interlocking pavers may not be suitable for handicapped parking spaces. Interlocking concrete pavers interspersed with other hardscape features (e.g., concrete walkways) can be used in creative designs to address ADA issues.

Groundwater Protection. While well-drained soils enhance the ability of permeable pavement to reduce stormwater runoff volumes, they may also increase the risk that stormwater pollutants might migrate into groundwater aquifers. Designers shall avoid the use of infiltration-based permeable pavement in areas known to provide groundwater recharge to aquifers used for water supply. In these source water protection areas, designers shall include liners and underdrains in large-scale permeable pavement applications (i.e., when the proposed surface area exceeds 10,000 square feet).

Potential Areas with High Pollutant Loading. Designers shall also certify that the proposed permeable pavement area will not accept any runoff from a designated area with high pollutant loading. Areas with potential for high pollutant loading are operations or activities that are known to produce higher concentrations of stormwater pollutants and/or have a greater risk of spills, leaks or illicit discharges. Examples include certain industrial activities, gas stations, public works areas, petroleum storage areas (for a complete list of these areas where infiltration may be restricted or prohibited, see Section 12 of WQ-04 (Bioretention). Restricted infiltration means that a minimum of 50% of the total WQv must be treated by a filtering or bioretention practice prior to the permeable pavement system. For known severe areas with high pollutant loading the risk of groundwater contamination from spills, leaks or discharges is so great that infiltration of stormwater or snowmelt through permeable pavement is prohibited.

Underground Injection Control Permits. The Safe Drinking Water Act regulates the infiltration of stormwater in certain situations pursuant to the Underground Injection Control (UIC) Program, which is administered by Arkansas Department of Environmental Quality (ADEQ). In general, the EPA (2008) has determined that permeable pavement installations are not classified as Class V injection wells, since they are always wider than they are deep.

Air and Runoff Temperature. Permeable pavement appears to have some value in reducing summer runoff temperatures, which can be important in watersheds with sensitive cold-water fish populations. The temperature reduction effect is greatest when runoff is infiltrated into the sub-base, but some cooling may also occur in the reservoir layer, when underdrains are used. ICPI (2008) notes that the use of certain reflective colors for interlocking concrete pavers can also help moderate surface parking lot temperatures.

Vehicle Safety. Permeable pavement is generally considered to be a safer surface than conventional pavement, according to research reported by Smith (2006), Jackson (2007) and ACI (2008). Permeable pavement has less risk of hydroplaning, more rapid ice melt and better traction than conventional pavement.

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WQ-13 Infiltration Devices

1. Description

Infiltration practices use temporary surface or underground storage to allow incoming stormwater runoff to exfiltrate into underlying soils. Runoff first passes through multiple pretreatment mechanisms to trap sediment and organic matter before it reaches the practice. As the stormwater penetrates the underlying soil, chemical and physical adsorption processes remove pollutants. Infiltration practices have the greatest runoff reduction capability of any stormwater practice and are suitable for use in residential and other urban areas where measured soil infiltration exceeds 0.5 inches per hour To prevent possible groundwater contamination, infiltration shall not be used at sites with potential for high pollutant loading (see Chapter 8 for list of such sites).

2. Typical Details

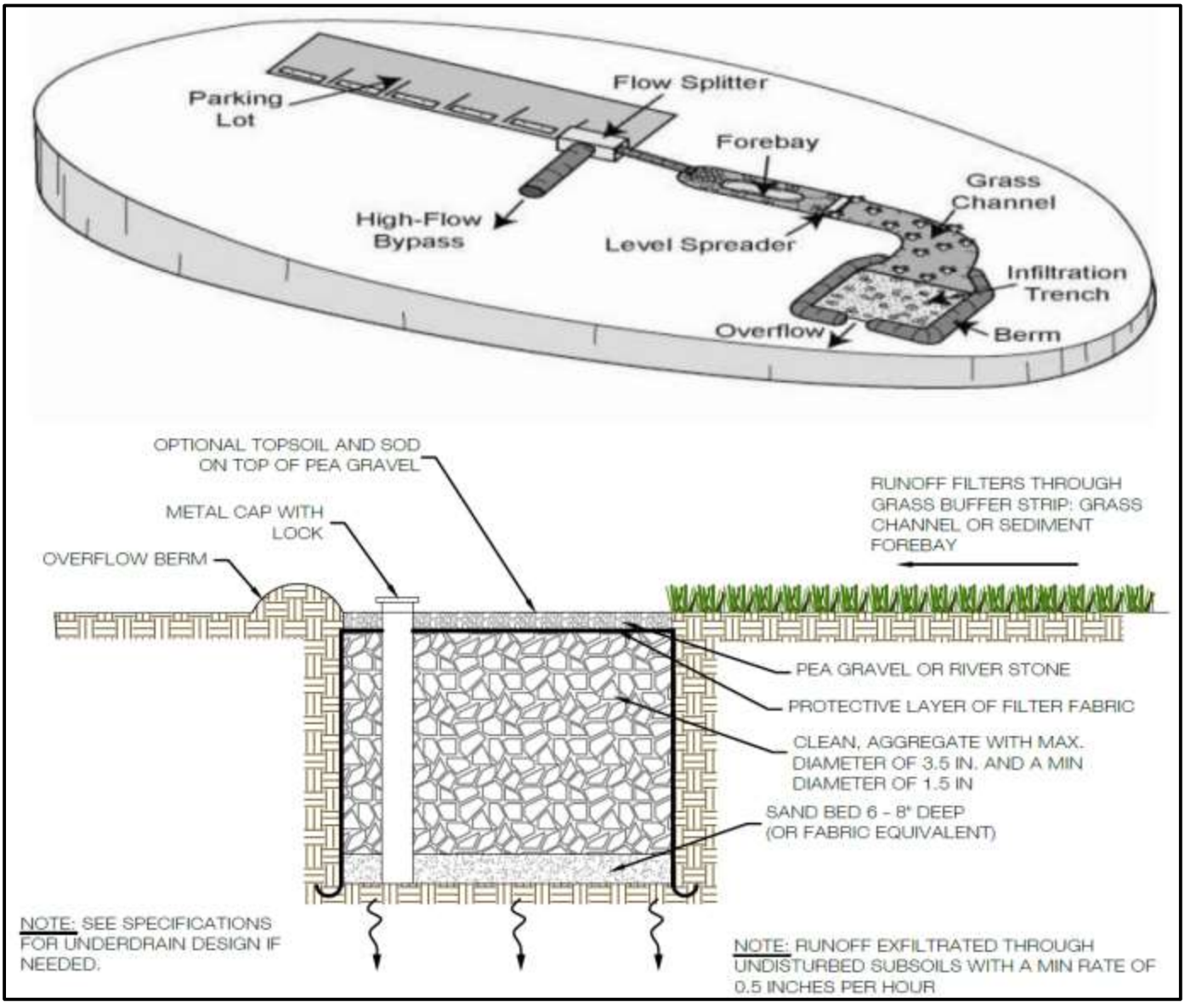


Figure C-64. Infiltration Plan and Section (Source: VADCR, 2011).

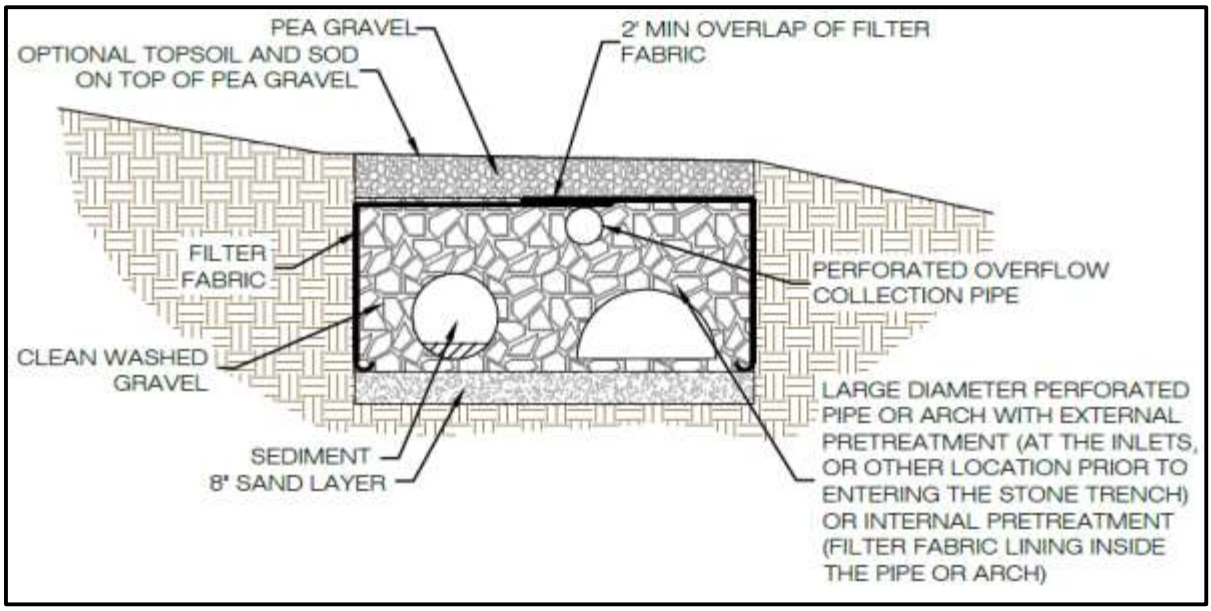


Figure C-65. Infiltration Section with Supplemental Pipe Storage (Source: VADCR, 2011).

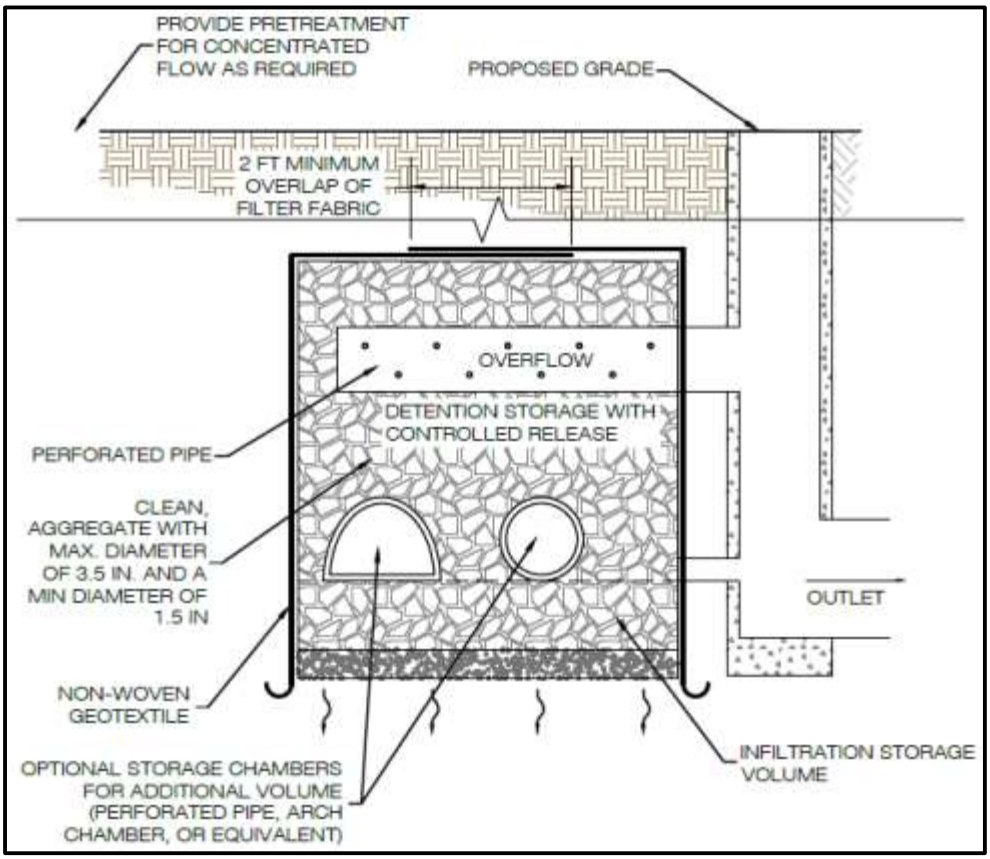


Figure C-66. Combined underground Detention (Channel and/or Flooding Protection) and Infiltration (Source: VADCR, 2011)

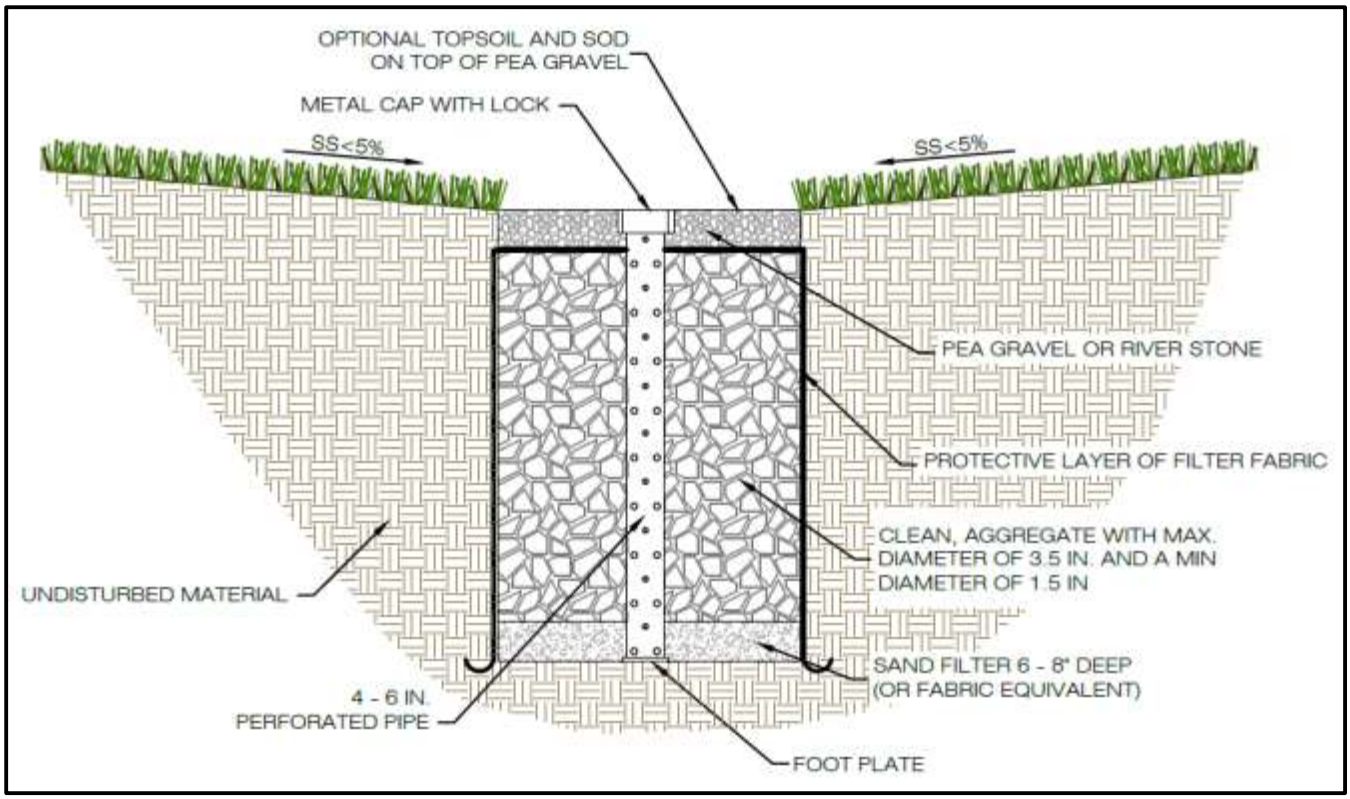


Figure C-67. Observation Well (Source: VADCR, 2011).

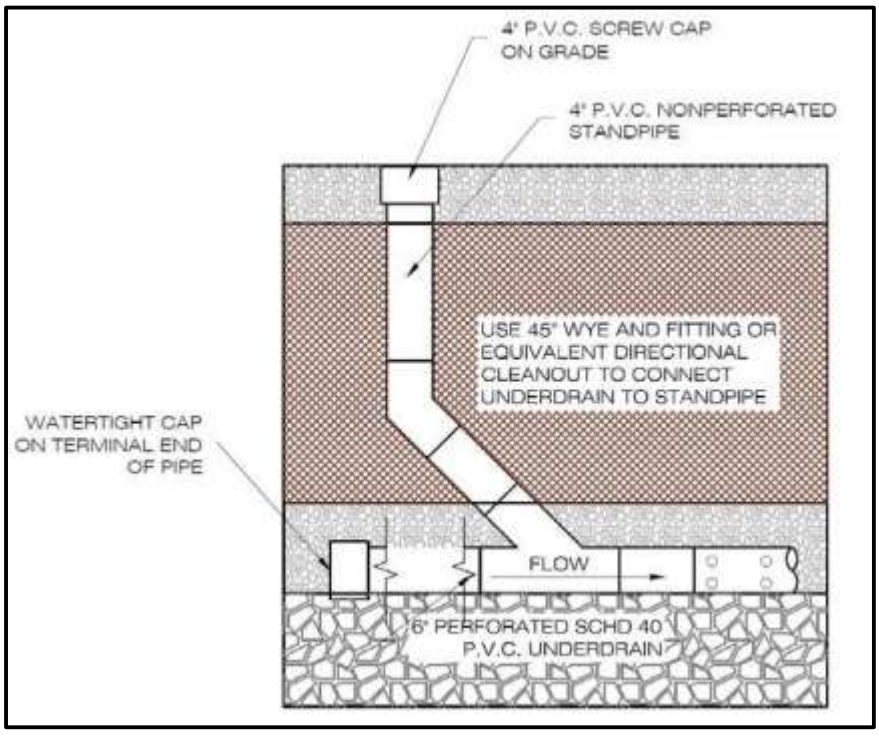


Figure C-68. Four-inch PVC Cleanout (Source: VADCR, 2011).

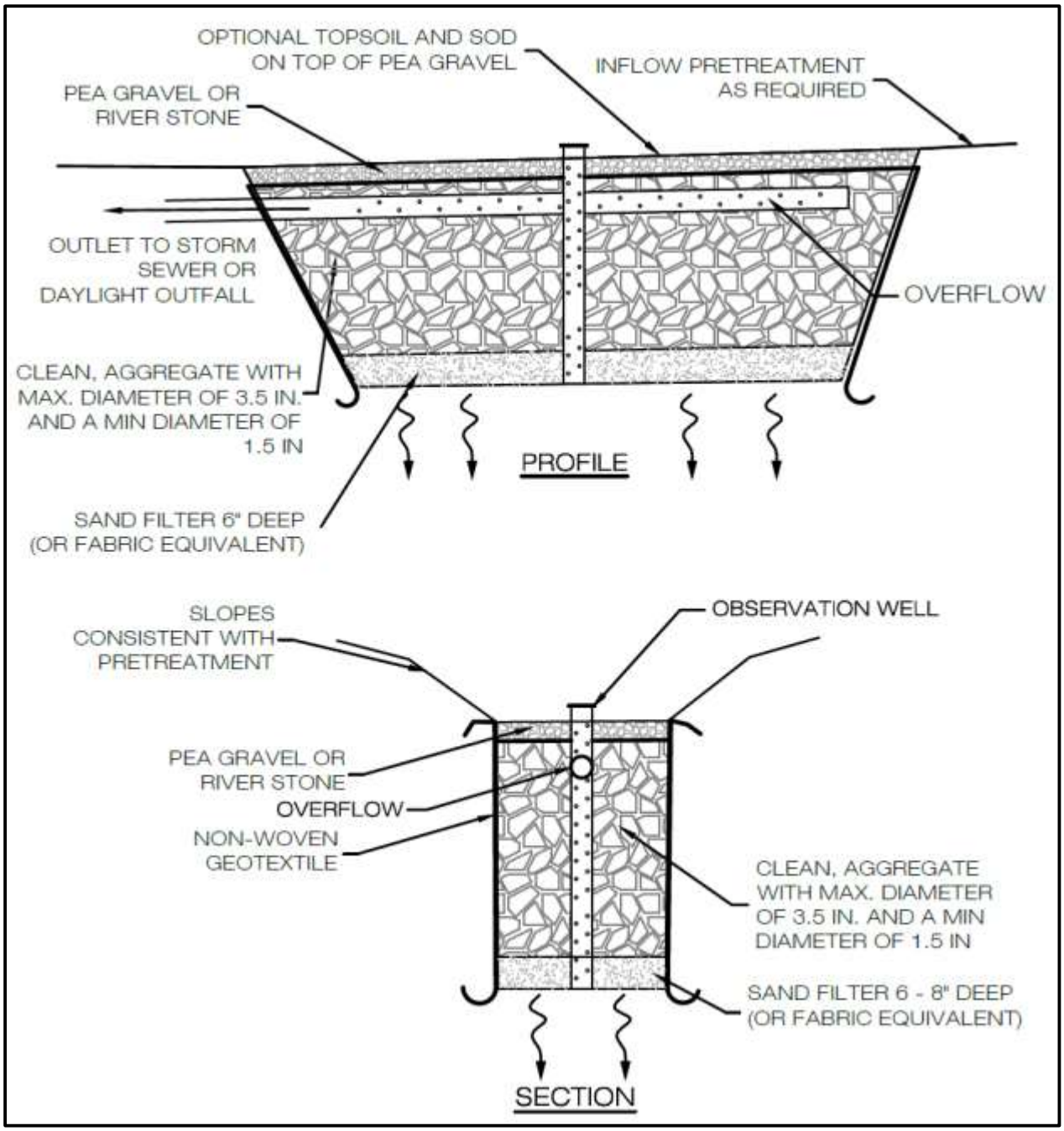


Figure C-69. Typical Infiltration Trench (Source: VADCR, 2011).

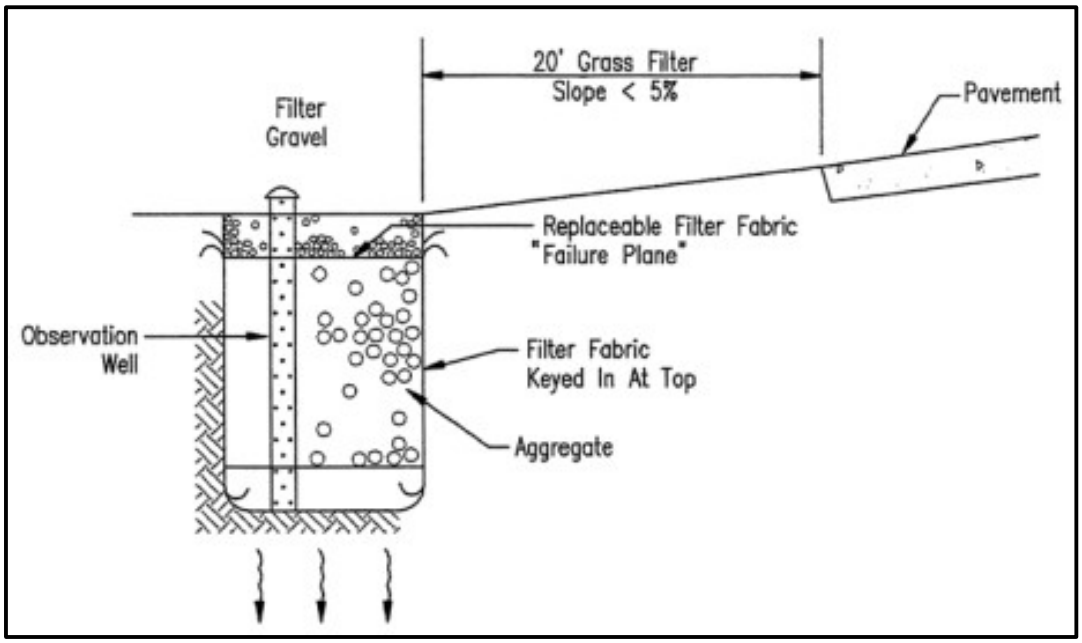


Figure C-70. Infiltration Trench Section (Source: VADCR, 2011).

3. Physical Feasibility and Design Applications

Since infiltration practices have a very high runoff reduction capability, they should be considered when initially evaluating a site. Designers should evaluate the range of soil properties during initial site layout and seek to configure the site to conserve and protect the soils with the greatest recharge and infiltration rates. Areas of Hydrologic Soil Group A or B soils shown on NRCS soil surveys shall be considered as primary locations for infiltration practices. At this point, designers should carefully identify and evaluate constraints on infiltration, as follows:

Contributing Drainage Area. The maximum contributing drainage area (CDA) to an individual infiltration practice shall be less than 2 acres and as close to 100% impervious as possible. This specification covers three scales of infiltration practices (1) Micro-infiltration (250 to 2,500 sq ft of CDA), (2) small-scale infiltration (2,500 to 20,000 sq ft of CDA) and (3) conventional infiltration (20,000 to 100,000 sq ft of CDA). The design, pretreatment and maintenance requirements differ, depending on the scale at which infiltration is applied (see Table **C-31** below for a summary).

Site Topography. Unless slope stability calculations demonstrate otherwise, infiltration practices shall be located a minimum horizontal distance of 200 ft from down-gradient slopes greater than 20%. The average slope of the contributing drainage areas shall be less than 15%.

Practice Slope. The bottom of an infiltration practice shall be flat (i.e., 0% longitudinal slope) to enable even distribution and infiltration of stormwater, although a maximum longitudinal slope of 1% is permissible if an underdrain is employed. Lateral slopes shall be 0%.

Minimum Hydraulic Head. The elevation difference needed to operate a micro-scale infiltration practice is nominal. However, 2 or more feet of head may be needed to drive small-scale and conventional infiltration practices.

Minimum Depth to Water Table or Bedrock. A minimum vertical distance of 2 ft must be provided between the bottom of the infiltration practice and the seasonal high-water table or bedrock layer.

Soils. Native soils in proposed infiltration areas must have a minimum infiltration rate of 0.5 inch per hour (typically Hydrologic Soil Group A and B soils meet this criterion). Initially, soil infiltration rates can be estimated from NRCS soil data but must be confirmed by an on-site infiltration evaluation.

Use on Urban Soils/Redevelopment Sites. Sites that have been previously graded or disturbed do not retain their original soil permeability due to compaction. Therefore, such sites are not ideal candidates for infiltration practices, however, if the measured infiltration rate of the underlying soils is 0.5 inch/hour or greater then infiltration can be used. Infiltration practices shall never be situated above earthen fill. Soils with significant clay content that is of moderate to high plasticity may lose their exfiltration properties over time due to swelling from absorbed water.

Dry Weather Flows. Infiltration practices shall not be used on sites receiving regular dry weather flows from sump pumps, irrigation nuisance water, and similar kinds of flows.

Setbacks. Infiltration practices shall not be hydraulically connected to structure foundations or pavement, in order to reduce seepage complications. Setbacks from structures and roads vary based on the scale of infiltration (see Table C-31). At a minimum, conventional and small-scale infiltration practices shall be located a minimum horizontal distance of 100 ft from any water supply well, 50 ft from septic systems, and at least 5 ft down-gradient from dry or wet utility lines.

Table C-31. Design Scales for Infiltration Practices.

Design Factor	Micro-Infiltration	Small-Scale Infiltration	Conventional Infiltration
Impervious Area Treated	250 to 2,500 sq ft	2,500 to 20,000 sq ft	20,000 to 100,000 sq ft
Typical Practices[1]	Dry Well French Drain Paving Blocks	Infiltration Trench	Infiltration Trench Infiltration Basin
Min. Infiltration Rate	0.5 in/hr measured at the depth in which the infiltration will occur		
Design Infiltration Rate	50% of measured rate		
Observation Well	No	Yes	Yes
Type of Pretreatment (See Table C-34)	External (leaf screens, grass filter strip, etc)	Vegetated filter strip or grass channel, forebay, etc.	Pretreatment Cell
Depth	Max. 3-ft depth	Max. 5-ft depth	Max. 6-ft depth
Head Required	Nominal: 1 to 3 ft.	Moderate: 1 to 5 ft.	Moderate: 2 to 6 ft.
Underdrain Requirements	An elevated underdrain only on marginal soils	None required	Backup underdrain
Required Soil Tests	One per practice	One (1) per 1,000 sq ft of surface area or max. two (2) per practice.	One per 1,000 sq ft of surface area.
Building Setbacks	5 ft down-gradient[2] 25 ft where up-gradient	10 ft down-gradient 50 ft where up-gradient	25 ft down-gradient 100 ft where up-gradient

1. Although permeable pavement is an infiltration practice, a more detailed specification is provided in WQ-12 Permeable Pavements.
2. Note that the building setback of 5 ft is intended for simple shallow foundations. The use of a dry well or French drain adjacent to an in-ground basement or finished floor area shall be carefully designed and coordinated with the design of the structure waterproofing system (foundation drains, etc.), or avoided altogether.

High Loading Situations. Infiltration practices are not intended to treat sites with high sediment or trash/debris loads, because such loads will cause the practice to clog and fail.

Groundwater Protection. WQ-04, Bioretention, presents a list of areas with potential for high pollutant loading that could pose a risk of groundwater contamination if used. Infiltration of runoff from these areas is restricted or prohibited and the list shall be referenced before selection of practices.

Site-Specific Considerations. Infiltration practices can be applied to most land uses that have measured soil infiltration rates that exceed 0.5 inches per hour. However, there is no single infiltration application that fits every development situation. The nature of the actual design application depends on four key design factors, described below:

1. The first factor is the Design Scale at which infiltration will be applied:
    - Micro-infiltration is intended for residential rooftop disconnection, rooftop rainwater harvesting systems, or other small-scale application (250 to 2,500 sq ft of impervious area treated).
    - mall-scale infiltration is intended for residential and/or small commercial applications that meet the feasibility criteria noted above.
    - Conventional infiltration can be considered for most typical development and redevelopment applications and therefore has more rigorous site selection and feasibility criteria. Table C-31above compares the different design approaches and requirements associated with each infiltration scale.
2. The second key design factor relates to the mode (or method) of temporarily storing runoff prior to infiltration – either on the surface or in an underground trench. When storing runoff on the surface (e.g., an infiltration basin), the maximum depth shall be no greater than 1 ft. However, if pretreatment cells are used, a maximum depth of 2 ft is permissible. In the underground mode, runoff is stored in the voids of the stones, and infiltrates into the underlying soil matrix. Perforated corrugated metal pipe, plastic pipe, concrete arch pipe, or comparable materials can be used in conjunction with the stone to increase the available temporary underground storage. In some instances, a combination of filtration and infiltration cells can be installed in the floor of a dry extended detention (ED) pond.
3. The third design factor relates to the degree of confidence that exfiltration can be maintained over time, given the measured infiltration rate for the subsoils at the practice location and the anticipated land uses. This determines whether an underdrain is needed, or whether an alternative practice, such as bioretention, is needed at the site (see Table C-32 below).

Table C-32. Design Choices Based on Measured Infiltration Rate.

	Measured Infiltration Rate (inches/hour)			
	Less than 0.5	**0.5 to 1.0**	**1.0 to 4.0**	**More than 4.0**
Recommended Design Solution	Use Bioretention or a Water Quality Swale with an underdrain	Use Infiltration without an underdrain, or with a 12-inch stone reservoir below the underdrain invert. Alternately, use Bioretention with an elevated underdrain.	Use Infiltration, Bioretention, or a Water Quality Swale without an underdrain.	Use Infiltration, Bioretention, or a Dry Swale without an underdrain.

4. The final factor is whether the infiltration practice will be designed as an on-line or off-line facility, as this determines the nature of conveyance and overflow mechanisms needed. Offline practices are sized to only accept some portion of the water quality volume and employ a flow splitter to safely bypass large storms. On-line infiltration practices may be connected to underground perforated pipes to detain the peak storm event or have suitable overflows to pass the storms without erosion.

4. Design Criteria

4.1 Defining the Infiltration Rate

Soil permeability is the single most important factor when evaluating infiltration practices. A field-verified minimum infiltration rate of at least 0.5 inches/hour is needed for the practice to work.

Projected Infiltration Rate. For planning purposes, the projected infiltration rate for the site can be estimated using the NRCS soil textural triangle or engineering test results for the prevailing soil types shown on the local NRCS Soil Survey. This data is used solely to locate portions of the site where infiltration may be feasible and to pinpoint where actual on-site infiltration tests will be taken to confirm feasibility.

Measured Infiltration Rate. On-site infiltration investigations shall be conducted to establish the actual infiltration capacity of underlying soils.

Design Infiltration Rate. Several studies have shown that ultimate infiltration rates decline by as much as 50% from initial rates, so designers should not attempt to use infiltration practices where infiltration rates barely meet minimum. To provide a factor of safety, the infiltration rate used in the design may be no greater than 50% of the measured rate.

4.2 Sizing of Infiltration Facilities

Several equations are needed to size infiltration practices. The first equations establish the maximum depth of the infiltration practice, depending on whether it is a surface basin (Equation C-18) or underground reservoir (Equation C-19).

$$d_{max} = {}^{1}/_{2}\, f \times t_d \qquad \textbf{Eq. C-18}$$

$$d_{max} = \frac{{}^{1}/_{2}\, f \times t_d}{V_r} \qquad \textbf{Eq. C-19}$$

Where: d_{max} = maximum depth of the infiltration practice (feet)

f = measured infiltration rate (feet/day)

t_d = maximum drawdown time (normally 1.5 to 2 days) (day)

V_r = void ratio of the stone reservoir (assume 0.35)

Designers shall compare these results to the maximum allowable depths in Table C-33, and use whichever value is less for subsequent design.

Table C-33. Maximum Depth for Infiltration Practices.

Mode of Entry	Infiltration Depth, ft		
	Micro Infiltration	Small Scale Infiltration	Conventional Infiltration
Surface Basin	1.0	1.5	2.0
Underground Reservoir	3.0	5.0	varies

Once the maximum depth is established, calculate the surface area needed for an infiltration practice using Equation C-20 or Equation C-21:

$$SA = \frac{WQv}{\left(d + {}^{1}/_{2}\, f \times t_f\right)} \qquad \textbf{Eq. C-20}$$

$$SA = \frac{WQv}{\left(V_r \times \left(d + {}^{1}/_{2}\, f\right) \times t_f\right)} \qquad \textbf{Eq. C-21}$$

Where: SA = Surface area (square feet)

WQv = Design volume (e.g., portion of the Water Quality Volume, in cubic feet)

V_r = Void Ratio (assume 0.35)

d = Infiltration depth (maximum depends on the scale of infiltration and the results of Equation C-18 or C-19 (feet)

f = Measured infiltration rate (feet/day)

t_f = Time to fill the infiltration facility (days – typically 2 hours, or 0.083 days)

If the designer chooses to infiltrate less than the full water quality volume (e.g., through the use of microinfiltration or small-scale infiltration), the runoff reduction rates shown in Table C-29 above must be directly prorated in the Runoff Reduction Method (RRM) calculations. To qualify for Level 2 runoff reduction rates, designers must provide a design based upon 110% of the site-adjusted water quality volume.

4.3 Soil Infiltration Rate

Since soil infiltration properties can vary, the different scales of infiltration shall be tested according to the following recommendations:

- **Micro-infiltration:** One test per facility
- **Small-Scale Infiltration:** One per 1,000 sq ft of surface area, or a maximum of two tests per facility
- **Conventional Infiltration:** One test per 1,000 sq ft of proposed infiltration bed

4.4 Pretreatment Features

Every infiltration practice must include multiple pretreatment techniques, although the nature of pretreatment practices depends on the scale at which infiltration is applied. The number, volume and type of acceptable pretreatment techniques needed for the three scales of infiltration are provided in Table C-34.

Table C-34. Required Pretreatment Elements for Infiltration Practices

Pretreatment[1]	Scale of Infiltration		
	Micro Infiltration	Small Scale Infiltration	Conventional Infiltration
Number and Volume of Pretreatment Techniques Employed	two external techniques; no minimum pretreatment volume required.	three techniques; 15% minimum pretreatment volume required (inclusive).	three techniques; 25% minimum pretreatment volume required (inclusive); at least one separate pre-treatment cell.
Acceptable Pretreatment Techniques	Leaf gutter screens Grass filter strip Upper sand layer Washed bank run gravel	Grass filter strip Grass channel Plunge pool Gravel diaphragm	Sediment trap cell Sand filter cell Sump pit Grass filter strip Gravel diaphragm

1. A minimum of 50% of the runoff reduction volume must be pre-treated by a filtering or bioretention practice prior to infiltration if the site is an area with potential for high pollutant loading. This application in such areas requires approval from the City engineer.

Note that conventional infiltration practices require pretreatment of at least 25% of the water quality volume, including a surface pretreatment cell capable of keeping sediment and vegetation out of the infiltration cell. All pretreatment practices shall be designed such that exit velocities are non-erosive for the two-year design storm and evenly distribute flows across the width of the practice (e.g., using a level spreader).

4.5 Internal Geometry and Drawdowns

Runoff Reduction Volume Sizing. The proper approach for designing infiltration practices is to avoid forcing a large volume to be infiltrated into a small area. Therefore, individual infiltration practices that are limited in size due to soil permeability and available space need not be sized to achieve the full water quality volume for the contributing drainage area, as long as other runoff reduction practices are applied at the site to meet the remainder of the WQv. The total runoff reduction volume must be documented using the Runoff Reduction Method spreadsheet or another locally approved methodology that achieves equivalent results. The minimum amount of runoff from a given drainage area that can be treated by individual infiltration practices is noted in Table C-31 above.

Infiltration Basin Restrictions. The maximum vertical depth to which runoff may be ponded over an infiltration area is 24 inches (i.e., infiltration basin). The side-slopes shall be no steeper than 4H:1V, and if the basin serves a CDA greater than 20,000 sq ft, a surface pre-treatment cell must be provided (this may be sand filter or dry sediment basin).

Rapid Drawdown. When possible, infiltration practices shall be sized so that the target runoff reduction volume infiltrates within 36 hours to 48 hours, to provide a factor of safety that prevents nuisance ponding conditions.

Conservative Infiltration Rates. Designers shall use the design infiltration rate, rather than the measured infiltration rate, to approximate long term infiltration rates (see Section 6.1 above).

Void Ratio. A maximum porosity value of 0.35 shall be used in the design of stone reservoirs, although a larger value may be used (based on actual storage / total volume computations) if perforated corrugated metal pipe, plastic pipe, concrete arch pipe, or comparable materials are installed within the reservoir.

4.6 Conveyance and Overflow

The nature of the conveyance and overflow to an infiltration practice depends on the scale of infiltration and whether the facility is on-line or off-line (Table C-35). Where possible, conventional infiltration practices shall be designed offline to avoid damage from the erosive velocities of larger design storms. Micro-scale and small-scale infiltration practices shall be designed to maintain non-erosive conditions for overland flows generated by the 2-year and 10-year design storms (typically 3.5 to 5.0 ft per second).

Table C-35. Conveyance and Overflow Choices Based on Infiltration Scale.

Conveyance and Overflow	Scale of Infiltration		
	Micro Infiltration	Small Scale Infiltration	Conventional Infiltration
Online Design	Discharge to a non-erosive pervious overland flow path designed to convey the 2-year design storm to the street or storm drain system.	An overflow mechanism such as an elevated drop inlet or flow splitter shall be used to redirect flows to a non-erosive down-slope overflow channel or stabilized water course designed to convey the 10-year design storm	
Off-line Design	Not Recommended	A flow splitter or overflow structure can be used for this purpose using the design guidance in Claytor and Schueler (1996) and ARC (2001).	

4.7 Landscaping and Safety

Infiltration trenches can be effectively integrated into the site plan and aesthetically designed with adjacent native landscaping or turf cover, subject to the following additional design considerations:

- Infiltration practices shall be installed only after all up-gradient construction is completed AND pervious areas are stabilized with dense and healthy vegetation.
- Vegetation associated with the infiltration practice buffers shall be regularly mowed and maintained to keep organic matter out of the infiltration device and maintain enough native vegetation to prevent soil erosion from occurring.
- Infiltration practices do not typically pose major safety hazards after construction. However, if an infiltration practice will be excavated to a depth greater than 5 ft, OSHA health and safety guidelines shall be followed for safe construction practices.
- Fencing of infiltration trenches is neither necessary nor desirable.

Designers should evaluate the nature of future operations to determine if the proposed site will be designated as a site with potential high pollutant loading and comply with the appropriate restrictions or prohibitions applicable to infiltration.

4.8 Maintenance Reduction Features

Maintenance is a crucial element that ensures the long-term performance of infiltration practices. The most frequently cited maintenance problem for infiltration practices is clogging of the stone by organic matter and sediment. The following design features can minimize the risk of clogging:

Observation Well. Small-scale and conventional infiltration practices should include an observation well, consisting of an anchored 6-inch diameter perforated PVC pipe fitted with a lockable cap installed flush with the ground surface, to facilitate periodic inspection and maintenance.

No Filter Fabric on Bottom. Avoid installing geotextile filter fabric along the bottom of infiltration practices. Experience has shown that filter fabric is prone to clogging, and a layer of coarse washed stone (choker stone) is a more effective substitute. However, permeable filter fabric must be installed on the trench sides to prevent soil piping.

Direct Maintenance Access. Access must be provided to allow personnel and heavy equipment to perform non-routine maintenance tasks, such as practice reconstruction or rehabilitation. While a turf cover is permissible for micro- and small-scale infiltration practices, the surface must never be covered by an impermeable material, such as asphalt or concrete.

4.9 Infiltration Material Specifications

The basic material specifications for infiltration practices are outlined in Table C-36 below.

Table C-36. Infiltration Material Specifications

Material	Specifications	Notes
Stone	Clean, aggregate with a maximum diameter of 3.5 inch and a minimum diameter of 1.5 inch (No. 1 size coarse aggregate, ASTM D448) or equivalent.	
Observation Well	Install a vertical 6-inch Schedule 40 PVC perforated pipe, with a lockable cap and anchor plate.	Install one per 50 ft length of the infiltration practice.
Trench Bottom	Install a 6-to-8-inch sand layer (ASTM C33) above a 2-inch minimum layer of choker stone (ASTM D 448, No. 8 stone)	
Trench Surface Cover	Install a 3-inch layer of river stone or pea gravel. Turf is acceptable when there is subsurface inflow (e.g., a roof leader).	This provides an attractive surface cover that can suppress weed growth.
Buffer Vegetation	Keep adjacent vegetation from forming an overhead canopy above infiltration practices, in order to keep leaf litter, fruits and other vegetative material from clogging the stone.	
Filter Fabric (sides only)	Use non-woven polyprene geotextile with a flow rate of > 110 gal/min/sq ft (e.g., Geotex 351 or equivalent)	
Choking Layer	Install a 2 to 4-in layer of choker stone (typically ASTM D448 No. 8 or No. 89 washed gravel) over the underdrain stone.	
Underdrain (where needed)	Use 6-in rigid schedule 40 PVC pipe, with 3/8-in perforations at 6 inches on center, with each underdrain, installed at a slope of 1% for the length of the infiltration practice.	Install non-perforated pipe with one or more caps, as needed from the downspout to a point 15 ft from the structure. Install T's as needed for the underdrain configuration.
Stone Jacket for Underdrain	The stone shall be double-washed and clean and free of all soil and fines.	Install a minimum 3-inch layer of #57 stone above the underdrain and a minimum 12-inch layer. beneath it.

5. Special Case Design Adaptations

5.1 Steep Terrain

Forcing conventional infiltration practices in steep terrain can be problematic with respect to slope stability, excessive hydraulic gradients, and sediment delivery. Unless slope stability calculations demonstrate otherwise, it is generally

recommended that infiltration practices shall be located a minimum horizontal distance of 200 ft from down-gradient slopes greater than 20%. Micro-scale and small-scale infiltration can work well, as long as up-gradient and down-gradient building setbacks are satisfied.

5.2 Linear Highway Sites

Infiltration practices can work well for linear highway projects, where soils are suitable and can be protected from heavy disturbance and compaction during road construction operations.

6. Construction

6.1 Construction Sequence

The following is a typical construction sequence to properly install infiltration practices. The sequence may need to be modified to reflect the scale of infiltration, site conditions, and whether an underdrain needs to be installed.

Infiltration practices are particularly vulnerable to failure during the construction phase for two reasons. First, if the construction sequence is not followed correctly, construction sediment can clog the practice. In addition, heavy construction equipment and disturbance can result in compaction of the soil, which can then reduce the soil's infiltration rate. For this reason, a careful construction sequence needs to be followed.

During site construction, the following steps are critical:

- Avoid excessive compaction by preventing construction equipment and vehicles from traveling over the proposed location of the infiltration practice.
- Keep the infiltration practice "off-line" until construction is complete. Prevent the entrance of sediments to the infiltration practice using silt fence, diversion berms or other means. In the erosion and sediment control plan, indicate the earliest time at which stormwater runoff may be directed to a conventional infiltration basin. This control plan must also indicate the specific methods to be used to temporarily keep runoff from the infiltration site.
- Infiltration practice sites shall never serve as the sites for temporary sediment control devices (e.g., sediment traps, etc.) during construction.
- Upland drainage areas need to be completely stabilized with a thick layer of vegetation prior to commencing excavation for an infiltration practice. Stabilization should be verified by the local erosion and sediment control inspector/program.

The actual installation of an infiltration practice is done using the following steps:

1. Excavate the infiltration practice to design dimensions using a backhoe or excavator. The floor of the pit shall be completely level, but mechanized equipment shall be kept off the base of the excavation to prevent soil compaction.
2. Scarify the bottom of the infiltration practice and spread 6 inches of sand on the bottom as a filter layer.
3. Install filter fabric on the trench sides. Large tree roots shall be trimmed flush with the sides of infiltration trenches to prevent puncturing or tearing of the filter fabric during subsequent installation procedures. The geotextile layout width shall include sufficient material to compensate for perimeter irregularities in the trench and accommodate a minimum 6- inch overlap at the top of the trench. The filter fabric itself shall be extended 4-inches beneath the sand layer on the bottom of the infiltration trench but shall not be extended across the trench. Stones or other anchoring objects shall be placed on the fabric at the trench sides, to keep the trench open during windy periods. If voids are present between the fabric and the excavated sides of a trench, they shall be backfilled with sand or pea gravel, to ensure the fabric conforms smoothly to the sides of excavation.
4. Install the underdrain if one is needed.
5. Anchor the observation well(s) and add stone to the practice in 1-ft lifts.
6. Use sod to establish a dense turf cover for at least 10 ft on each side of the infiltration practice, to reduce erosion and sloughing. If the vegetation is seeded instead, use primarily native grasses.

6.2 Construction Inspection

Inspections should be performed during construction to ensure that the infiltration practice is built in accordance with the approved design and this guidance. Qualified individuals shall use detailed inspection checklists to include signoffs at critical stages of construction, to ensure that the contractor's interpretation of the plan is consistent with the designer's intentions. A performance evaluation of the infiltration system shall be prepared and provided by the design engineer certifying that the system was installed correctly and is functioning as intended. The performance evaluation shall be submitted to the City prior to receiving the Certificate of Occupancy.

7. Maintenance

7.1 Maintenance Plans

It is recommended that a Long-Term Maintenance Plan (LTMP) be developed by the design engineer. The LTMP contains a description of the stormwater system components and information on the required inspection and maintenance activities.

When micro-scale or small-scale infiltration practices are installed on private residential lots, homeowners will need to (1) be educated about their routine maintenance needs, (2) understand the long-term maintenance plan, and (3) agree to a deed restriction, drainage easement or other enforceable mechanism to ensure that infiltrating areas are not converted or disturbed. The mechanism shall, if possible, grant authority for local agencies to access the property for inspection or corrective action. In addition, the GPS coordinates shall be logged for all infiltration practices, upon facility acceptance, and submitted to the City Engineer.

7.2 Maintenance Inspections

Annual site inspections are critical to the performance and longevity of infiltration practices, particularly for small-scale and conventional infiltration practices. Maintenance of infiltration practices is driven by annual inspections that evaluate the condition and performance of the practices, including the following:

- The drawdown rate shall be measured at the observation well for three days following a storm event in excess of 1/2 inches. If standing water is still observed in the well after three days, this is a clear sign that that clogging is a problem.
- Check inlets, pre-treatment cells, and any flow diversion structures for sediment buildup and structural damage. Note if any sediment needs to be removed.
- Inspect the condition of the observation well and make sure it is still capped.
- Check that vegetation has not formed an overhead canopy that may drop leaf litter, fruits and other vegetative materials that could clog the infiltration device.
- Evaluate the vegetative quality of the adjacent grass buffer and perform spot-reseeding if the cover density is less than 90%.
- Generally, inspect the upland CDA for any controllable sources of sediment or erosion.
- Look for weedy growth on the stone surface that might indicate sediment deposition or clogging.
- Inspect maintenance access to ensure it is free of woody vegetation, and check to see whether valves, manholes and/or locks can be opened and operated.
- Inspect internal and external infiltration side slopes for evidence of sparse vegetative cover, erosion, or slumping, and make necessary repairs immediately. Based on inspection results, specific maintenance tasks may be triggered.

7.3 Ongoing Maintenance

Effective long-term operation of infiltration practices requires a dedicated and routine maintenance inspection schedule with clear guidelines and schedules, as shown in Table C-37 below. Where possible, facility maintenance shall be integrated into routine landscaping maintenance tasks.

Table C-37. Typical Maintenance Activities for Infiltration Practices

Maintenance Activity	Schedule
• Replace pea gravel/topsoil and top surface filter fabric (when clogged). • Mow vegetated filter strips as necessary and remove the clippings.	As needed
• Ensure that the contributing drainage area, inlets, and facility surface are clear of debris. • Ensure that the contributing drainage area is stabilized. • Remove sediment and oil/grease from pre-treatment devices, as well as from overflow structures. • Repair undercut and eroded areas at inflow and outflow structures	Quarterly
• Check observation wells 3 days after a storm event more than 1/2 inch in depth. Standing water observed in the well after three days is a clear indication of clogging. • Inspect pre-treatment devices and diversion structures for sediment build up and structural damage. • Remove trees that start to grow in the vicinity of the infiltration facility.	Semi-annual inspection
• Clean out accumulated sediments from the pre-treatment cell.	Annually

8. As-Built Requirements

After the infiltration device has been constructed, the developer must have an as-built certification of the infiltration system prepared by a registered Professional Engineer and submit this to the City Engineer. The as-built certification verifies that the BMP was installed as designed and approved.

The following components must be addressed in the as-built certification:

- Infiltration rates must be verified.
- Proper dimensions for the trench must be verified.
- A mechanism for overflow for large storm events must be provided.

9. References

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WQ-14 Water Quality Swale

1. Description

Water quality swales are essentially bioretention cells that are shallower, configured as linear channels, and covered with turf or other planting material. The water quality swale is a soil filter system that temporarily stores and then filters the desired Water Quality Volume (WQv). Water quality swales rely on a pre-mixed soil media filter below the channel that is like that used for bioretention. If soils are extremely permeable, runoff infiltrates into underlying soils. Otherwise, the runoff treated by the soil media flows into an underdrain that conveys treated runoff to the conveyance system further downstream. The underdrain system consists of a perforated pipe within a gravel layer on the bottom of the swale, beneath the soil media. Water quality swales may appear as simple grass channels with the same shape and turf cover, while others may have more elaborate landscaping. Swales can be planted with turf grass, tall meadow grasses, decorative herbaceous cover, or trees.



Figure C-71. Typical Water Quality Swale in commercial/office setting (Source: VADCR, 2011).

2. Typical Details

Figures C-72 through C-76 below provide typical schematics for water quality swales.

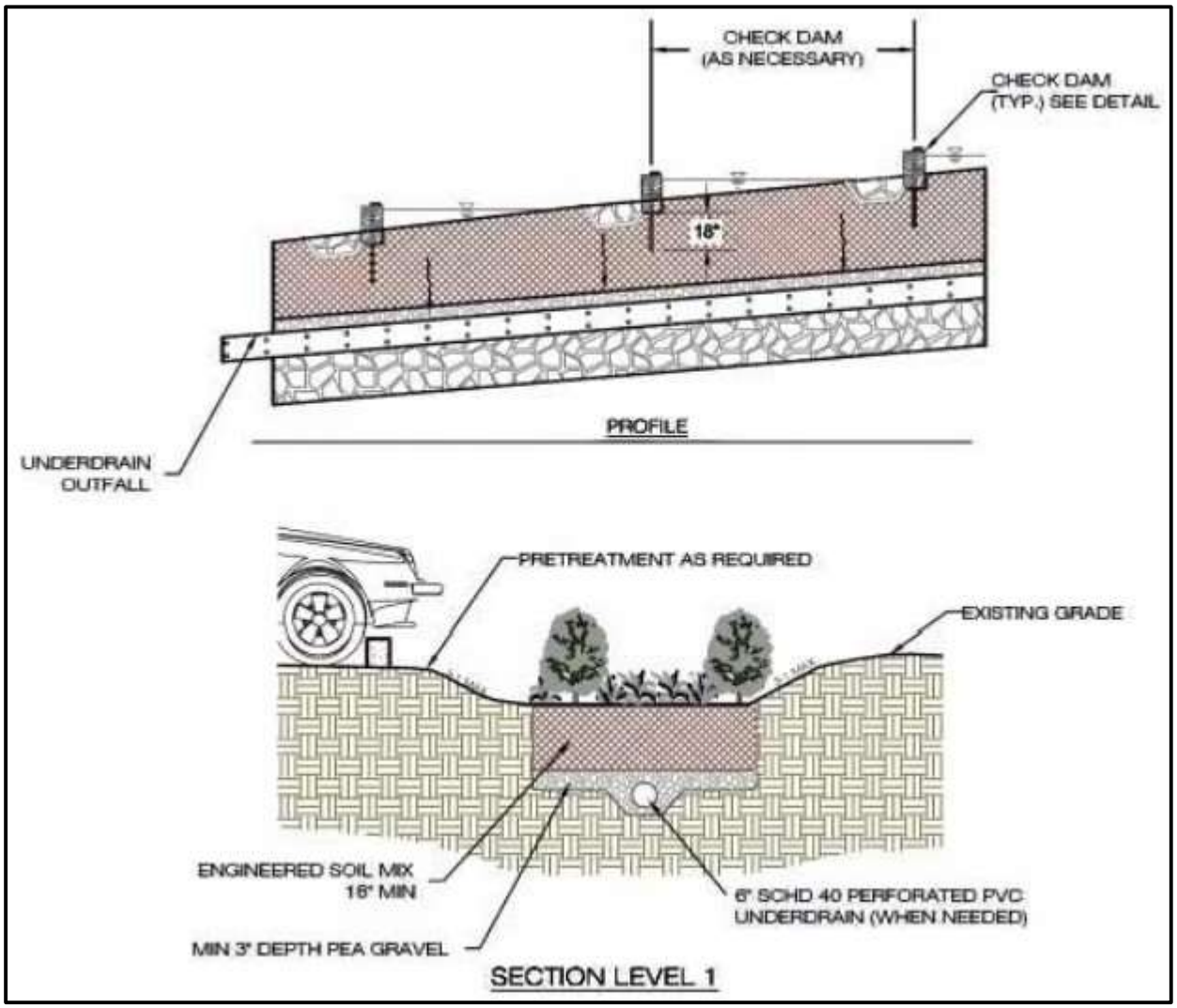


Figure C-72. Typical Detail for Level 1 Water Quality Swale (Source: VADCR, 2011).

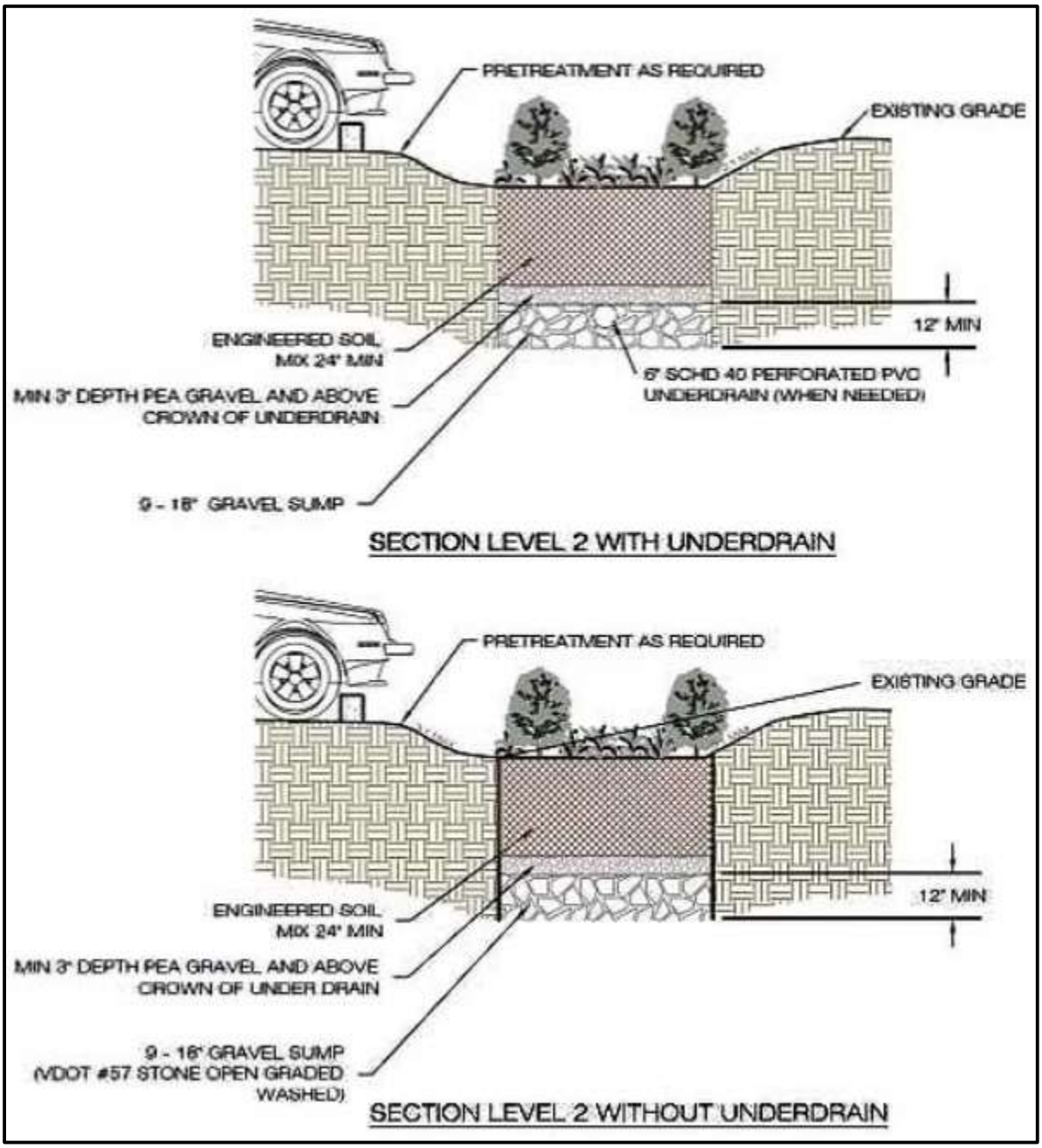


Figure C-73. Typical Detail for Level 2 Water Quality Swale (Source: VADCR, 2011).

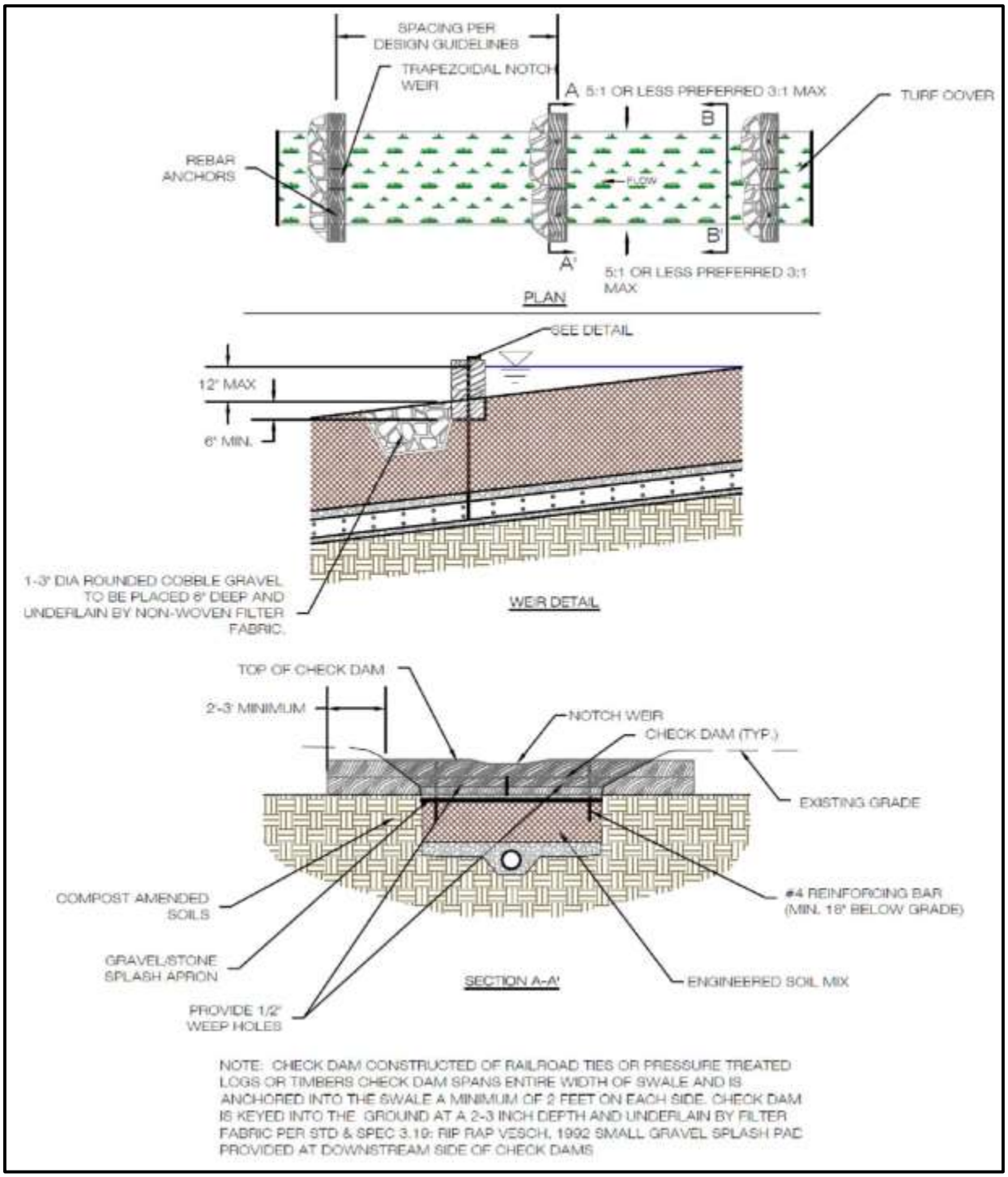


Figure C-74. Typical Detail for Water Quality Swale Check Dam (Source: VADCR, 2011).

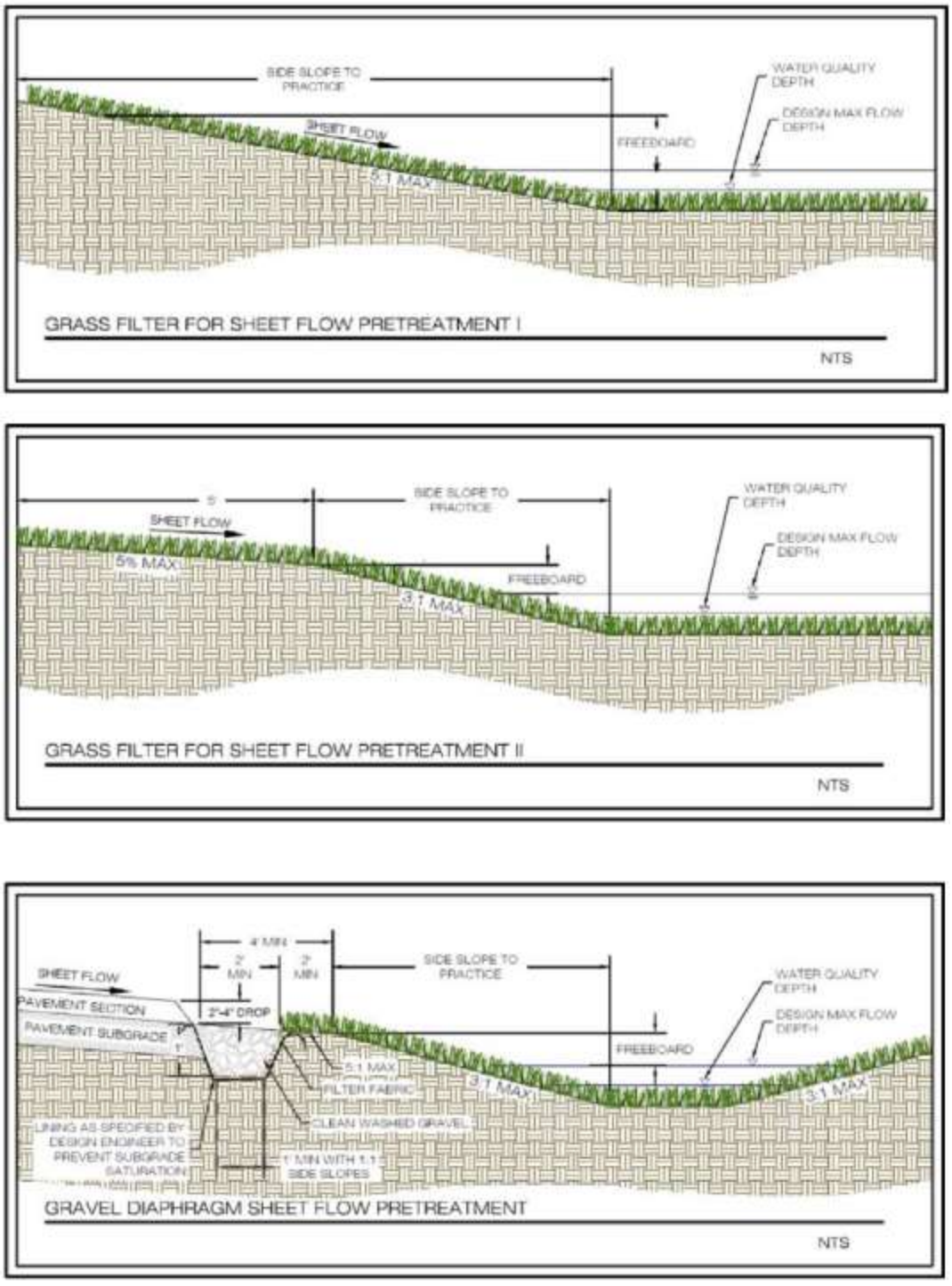


Figure C-75. Pretreatment. Gravel Diaphragm for Sheet Flow from Impervious or Pervious (Source: VADCR, 2011).

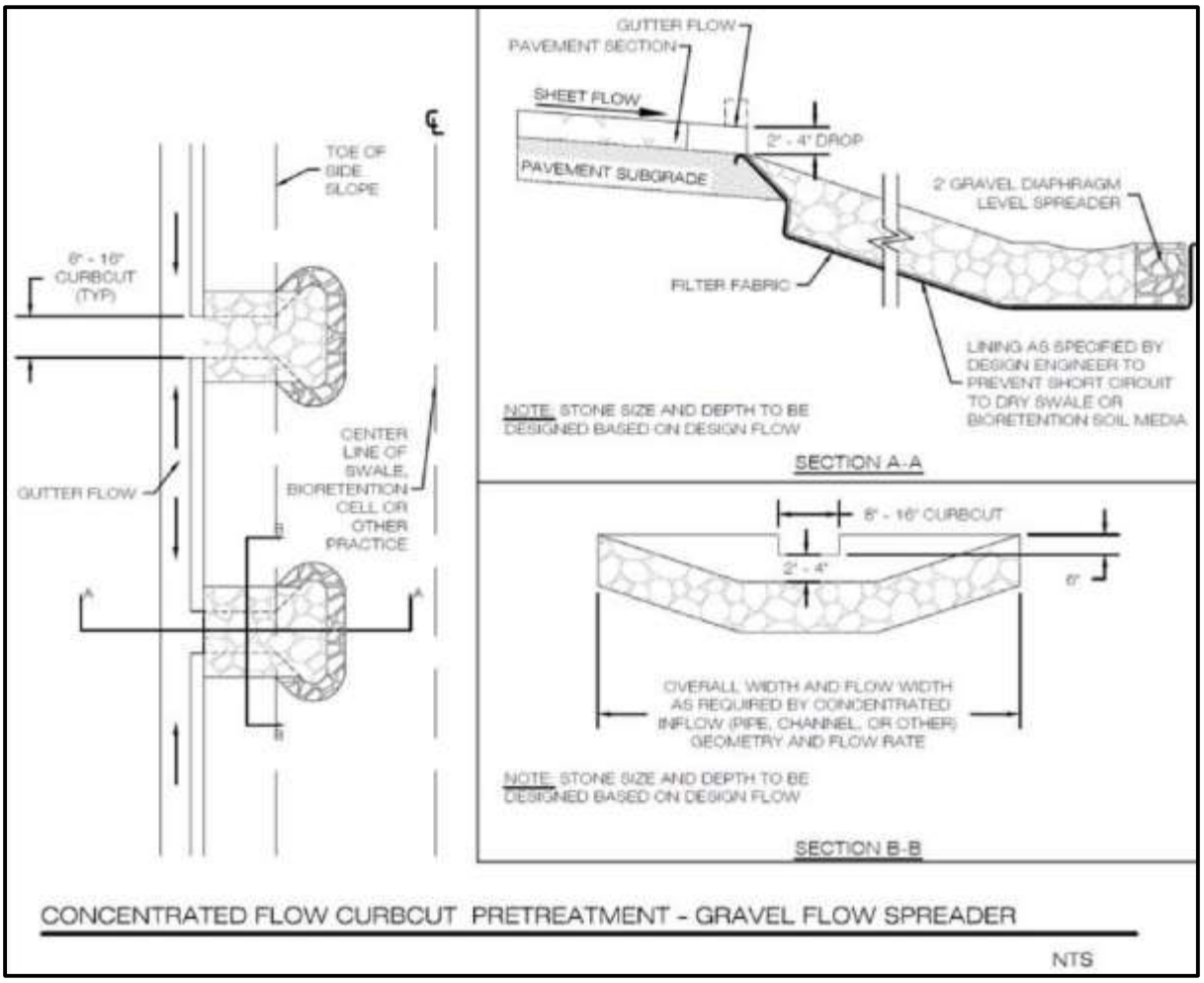


Figure C-76. Pre-Treatment – Gravel Flow Spreader for Concentrated Flow (Source: VADCR, 2011).

3. Physical Feasibility and Design Applications

Water quality swales can be implemented on a variety of development sites where density and topography permit their application. Some key feasibility issues for water quality swales include the following:

Contributing Drainage Area. The maximum contributing drainage area to a water quality swale shall be 5 acres or less. When water quality swales treat larger drainage areas, the velocity of flow through the surface channel often becomes too great to treat runoff or prevent erosion in the channel. Similarly, the longitudinal flow of runoff through the soil, stone, and underdrain may cause hydraulic overloading at the downstream sections of the water quality swale. An alternative is to provide a series of inlets or diversions that convey the treated water to an outlet location.

Available Space. Water quality swale footprints can fit into relatively narrow corridors between utilities, roads, parking areas, or other site constraints. Water quality swales should be approximately 3% to 10% of the size of the contributing drainage area, depending on the amount of impervious cover.

Site Topography. Water quality swales should be used on sites with longitudinal slopes of less than 4%, but preferably less than 2%. Check dams can be used to reduce the effective slope of the swale and lengthen the contact time to

enhance filtering and/or infiltration. Steeper slopes adjacent to the swale may generate rapid runoff velocities into the swale that may carry a high sediment loading (refer to pre-treatment criteria in Section 6.4).

Available Hydraulic Head. A minimum number of hydraulic head is needed to implement water quality swales, measured as the difference in elevation between the inflow point and the downstream storm drain invert. Water quality swales typically require 3 to 5 ft of hydraulic head since they have a filter bed and potentially an underdrain.

Hydraulic Capacity. Level 1 water quality swales are an on-line practice and must be designed with enough capacity to (1) convey a portion of the runoff from the 100-year design storms at non-erosive velocities, and (2) contain the 10-year flow within the banks of the swale. This means that the swale's surface dimensions are more often determined by the need to pass the 10-year storm events, which can be a constraint in the siting of water quality swales within existing right of way (e.g., constrained by sidewalks).

Depth to Water Table. Designers shall ensure that the bottom of the water quality swale is at least 2 ft above the seasonally high groundwater table, to ensure that groundwater does not intersect the filter bed, since this could lead to groundwater contamination or practice failure.

Soils. Soil conditions do not constrain the use of water quality swales, although they normally determine whether an underdrain is needed. Low-permeability soils with an infiltration rate of less than 0.5 inch per hour, such as many of those classified in Hydrologic Soil Groups (HSG) C and D, will require an underdrain. Designers must verify site-specific soil permeability at the proposed location to eliminate the requirements for an underdrain.

Setbacks from Utilities, Buildings and Roads. Designers shall consult local utility design guidance for the horizontal and vertical clearance between utilities and the swale configuration. Utilities can cross linear swales if they are specially protected (e.g., double-cased). Water and sewer lines may need to be placed under road pavements if water quality swales are placed adjacent to roadways. Water and sewer lines that parallel the swale shall be positioned at least 10 ft horizontally from the edge of the infiltration section of the water quality swale. Water and sewer lines that cross under the swale shall have 18 inches of separation from the bottom of the infiltration portion of the swale and shall be encased or be installed with ductile iron pipe with mechanical joints. Storm drainage pipes can cross under water quality swales if adequate cover for the pipes is provided, and it functions adequately with the stormwater management system. Given their landscape position, water quality swales are not subject to normal building setbacks. The bottom elevation of swales shall be at least 1 ft below the invert of any adjacent roadbed.

Avoidance of Irrigation or Baseflow. Water quality swales shall be located to avoid inputs from springs, irrigation systems, chlorinated wash-water, or other dry weather flows.

Areas with Potential for High Pollutant Loading. Runoff from land uses with potential for high pollutant loading should not be treated using infiltrating water quality swales. An impermeable liner with underdrain shall be used for filtration of runoff from these areas.

Community Acceptance. The main concerns of adjacent residents are perceptions that swales will create nuisance conditions or will be hard to maintain. Common concerns include the continued ability to mow grass, landscape preferences, weeds, standing water, and mosquitoes. Water quality swales are a positive stormwater management alternative, because all these concerns can be fully addressed through the design process and proper on-going operation and routine maintenance. The ponding time is less than the time required for one mosquito breeding cycle, so well-maintained water quality swales should not create mosquito problems or be difficult to mow.

4. Design Criteria

4.1 Sizing of Water Quality Conveyance and Water Quality Treatment Swales

Sizing of the surface area (SA) for water quality swales is based on the computed Water Quality Volume (WQv) of the contributing drainage area and the storage provided within the swale media and gravel layers and behind check dams. The required surface area (in square feet) is computed as the Water Quality Volume (in cubic feet) divided by the

equivalent storage depth (in feet). The equivalent storage depth is computed as the depth of the soil media, the gravel, and surface ponding (in feet) multiplied by the accepted void ratio.

The maximum Void Ratios (Vr) acceptable are shown in Table C-40 below.

Table C-40. Maximum Allowable Void Ratios.

Type	V_r
Water Quality Soil Media	0.40
Gravel	0.35
Surface Storage Behind Check Dams	1.0

For example, the equivalent storage depth for two feet of media depth (without considering surface ponding) is therefore computed as:

$$[\mathbf{2.0}\boldsymbol{ft} \times \mathbf{0.40} + (\mathbf{1.0}\boldsymbol{ft} \times \mathbf{0.35}\ )] = \mathbf{1.15}\boldsymbol{ft}$$

The effective storage depths will vary according to the actual void ratios, design depths of the soil media and gravel layer.

Note: When using Equations C-22 below to calculate the required surface area of a water quality swale that includes surface ponding (with check dams), the storage depth calculation (shown above) shall be adjusted accordingly.

The Water Quality Swale Surface Area (SA) is computed as:

$$\boldsymbol{SA}\ (\boldsymbol{sq\ ft}) = (\boldsymbol{WQv - the\ volume\ reduced\ by\ an\ upstream\ GSP}) / \boldsymbol{ESD\ Level\ 1} \qquad \textbf{Eq. C-22}$$

NOTE: The volume reduced by upstream pretreatment practices is supplemented with the anticipated volume of storage created by check dams along the swale length.

Where: SA = Minimum surface area of Water Quality Swale (sq ft)

WQv = Water Quality Volume (cu. Ft.) = [(1.05 inch) (Rv)(A)] *3630

A = Area in acres.

The final water quality swale design geometry will be determined by dividing the SA by the swale length to compute the required width; or by dividing the SA by the desired width to compute the required length.

4.2 Soil Infiltration Rate Testing

The second key sizing decision is to measure the infiltration rate of subsoils below the water quality swale area to determine if an underdrain will be needed. The infiltration rate of the subsoil must exceed 0.5 inch per hour to avoid installation of an underdrain. A soil test shall be conducted for every 200 linear feet of water quality swale.

4.3 Water Quality Swale Geometry

Design guidance regarding the geometry and layout of water quality swales is provided below.

Shape. A parabolic shape at the surface is preferred for water quality swales for aesthetic, maintenance, and hydraulic reasons. However, the design may be simplified with a trapezoidal cross-section at the surface, as long as the soil filter bed boundaries are “keyed in” with vertical sides and lay in the flat bottom areas.

Side Slopes. The side slopes of water quality swales shall be no steeper than 3H:1V for maintenance considerations (i.e., mowing). Flatter slopes are encouraged where adequate space is available, to enhance pre-treatment of sheet flows entering the swale. Swales should have a bottom width of from 2 to 8 ft to ensure that an adequate surface area

exists along the bottom of the swale for filtering. If a swale will be wider than 8 ft, the designer should incorporate berms, check dams, level spreaders or multi-level cross-sections to prevent braiding and erosion of the swale bottom.

Swale Longitudinal Slope. The longitudinal slope of the swale should be moderately flat to permit the temporary ponding of the Water Quality Volume within the channel. The recommended swale slope is less than or equal to 2% for a Level 1 design and less than or equal to 1% for a Level 2 design, though slopes up to 4% are acceptable if check dams are used. The minimum recommended slope for an on-line water quality swale is 0.5%. Refer to Table C-41 for check dam spacing based on the swale longitudinal slope.

Table C-41. Typical Check Dam (CD) Spacing to Achieve Effective Swale Slope.

Swale Longitudinal Slope	Level 1	Level 2
	Spacing[1] of 12-inch High (max.) Check Dams [2, 3] to Create an Effective Slope of 2%	Spacing[1] of 12-inch High (max.) Check Dams [2, 3] to Create an Effective Slope of 0 to 1%
0.5%	–	200 ft to –
1.0%	–	100 ft to –
1.5%	–	67 ft to 200 ft
2.0%	–	50 ft to 100 ft
2.5%	200 ft	40 ft to 67 ft
3.0%	100 ft	33 ft to 50 ft
3.5%	67 ft	30 ft to 40 ft
4.0%	50 ft	25 ft to 33 ft

1. The spacing dimension is half of the above distances if a 6-inch check dam is used.
2. Check dams require a stone energy dissipater at the downstream toe.
3. Check dams require weep holes at the channel invert. Swales with slopes less than 2% will require multiple weep holes at least three in each check dam.

Check dams. Check dams must be firmly anchored into the side-slopes to prevent outflanking and be stable during the 10-year storm design event. The height of the check dam relative to the normal channel elevation shall not exceed 12 inches. Each check dam shall have a minimum of one weep hole or a similar drainage feature so it can dewater after storms. Armoring may be needed behind the check dam to prevent erosion. The check dam must be designed to spread runoff evenly over the water quality swale's filter bed surface, through a centrally located depression with a length equal to the filter bed width. In the center of the check dam, the depressed weir length shall be checked for the depth of flow and sized for the appropriate design storm (Figure C-74). Check dams shall be constructed of wood or stone.

Soil Plugs. Soil plugs are small areas of compacted soil placed in front of and below the check dam to prevent water from flowing under the check dams. Soil plugs serve to help minimize the potential for blow-out of the soil media underneath the check dams, due to hydrostatic pressure from the upstream ponding. Soil plugs are appropriate for water quality swales with 12-inch-high check dams.

Ponding Depth. Drop structures or check dams can be used to create ponding cells along the length of the swale. The maximum ponding depth in a swale shall not exceed 12 inches at the most downstream point.

Drawdown. Water quality swales shall be designed so the desired water quality volume is completely filtered within 24 hours or less. This drawdown time can be achieved by using the soil media mix specified in Section 6.6 and an underdrain along the bottom of the swale, or native soils with adequate permeability, as verified through testing (Section 6.2).

Underdrain. Underdrains are provided in water quality swales to ensure that they drain properly after storms. The underdrain shall have a minimum diameter of 4 inches and be encased in a 6-inch-deep gravel bed. Two layers of stone

shall be used. A choker stone layer, consisting of #8 or #78 stone at least 3 inches deep, shall be installed immediately below the filter media. Below the choker stone layer, the main underdrain layer shall be at least 12 inches deep and composed of clean washed #57 stone. The underdrain pipe shall be set at least 4 inches above the bottom of the stone layer.

4.4 Pre-treatment

Several pre-treatment measures are feasible, depending on whether the specific location in the water quality swale system will be receiving sheet flow, shallow concentrated flow, or fully concentrated flow:

Initial Sediment Forebay (channel flow). This grass cell is located at the upper end of the water quality swale segment with a 2:1 length to width ratio and a storage volume equivalent to at least 15% of the total water quality volume.

- Check dams (channel flow). These energy dissipation devices are acceptable as pre-treatment on small swales with drainage areas of less than 1 acre.
- Tree Check dams (channel flow). These are street tree mounds that are placed within the bottom of a water quality swale up to an elevation of 9 to 12 inches above the channel invert. One side has a gravel or river stone bypass to allow storm runoff to percolate through.
- Grass Filter Strip (sheet flow). Grass filter strips extend from the edge of the pavement to the bottom of the water quality swale at a 5:1 slope or flatter. Alternatively, provide a combined 5 feet of grass filter strip at a maximum 5% (20:1) slope and 3:1 or flatter side slopes on the water quality swale. (Figure C-74)
- Gravel Diaphragm (sheet flow). A gravel diaphragm located at the edge of the pavement shall be oriented perpendicular to the flow path to pre-treat lateral runoff, with a 2-to-4-inch drop. The stone must be sized according to the expected rate of discharge. (Figure C-75)
- Pea Gravel Flow Spreader (concentrated flow). The gravel flow spreader is located at curb cuts, downspouts, or other concentrated inflow points, and shall have a 2-to-4-inch elevation drop from a hard-edged surface into a gravel or stone diaphragm. The gravel shall extend the entire width of the opening and create a level stone weir at the bottom or treatment elevation of the swale. (Figure C-76)

4.5 Conveyance and Overflow

The bottom width and slope of a water quality swale shall be designed such that the velocity of flow from a 1-inch rainfall will not exceed 3 feet per second. Check dams may be used to achieve the needed runoff reduction volume, as well as to reduce the flow. Check dams shall be spaced based on channel slope and ponding requirements, consistent with the criteria in Table C-41.

The swale shall also convey the 2- and 10-year storms at non-erosive velocities with at least 6 inches of freeboard. The analysis shall evaluate the flow profile through the channel at normal depth, as well as the flow depth over top of the check dams.

Water quality swales may be designed as off-line systems, with a flow splitter or diversion to divert runoff in excess of the design capacity to an adjacent conveyance system, or strategically placed overflow inlets may be placed along the length of the swale to periodically pick up water and reduce the hydraulic loading at the downstream limits.

4.6 Water quality Swale Material Specifications

Table C-42 outlines the standard material specifications for constructing water quality swales.

Table C-42. Water Quality Swale Material Specifications.

Material	Specification	Notes
Filter Media Composition	Engineered Soil Mix A provided in Appendix C	The volume of filter media is based on 110% of the product of the surface area and the media depth, to account for settling.
Filter Media Testing	Mix on-site or procure from an approved media vendor (refer to Appendix C for additional soil media information.	
Filter Fabric	A non-woven polyprene geotextile with a flow rate of > 110 gal/min/sq ft (e.g., Geotex 351 or equivalent); Apply immediately above the underdrain only.	
Choking Layer (Initial Filtering Layer)	A 3- inch layer of choker stone (typically ASTM D448 No. 8 or No. 89 or equivalent washed gravel) laid above the underdrain stone.	
Stone and/or Storage Layer	A 9-to-18-inch layer (depending on the desired depth of the storage layer) of ASTM D448 No. 57 stone shall be double-washed and clean and free of all soil and fines.	
Underdrains, Cleanouts, and Observation Wells	6-inch rigid schedule 40 PVC pipe, with 3/8-inch perforations. For observation well, perforated sections shall only extend through stone/gravel layer.	Install perforated pipe for the full length of the water quality swale. Use non-perforated pipe, as needed, to connect with the storm drain system.
Vegetation	Plant species as specified on the landscaping plan	
Check Dams	Use non-erosive material such as wood, gabions, riprap, or concrete. All check dams shall be underlain with filter fabric and include weep holes. Wood used for check dams shall consist of pressure-treated logs or timbers, or water-resistant tree species such as cedar, hemlock, swamp oak or locust.	
Erosion Control Fabric	Where flow velocities dictate, use woven biodegradable erosion control fabric or mats that are durable enough to last at least 2 growing seasons.	

4.7 Filter Media

Water quality swales require replacement of native soils with a prepared soil media. The soil media provides adequate drainage, supports plant growth, and facilitates pollutant removal within the water quality swale. At least 18 inches of soil media should be added above the choker stone layer to create an acceptable Level 1 filter. The mixture for the soil media is identical to that used for bioretention and details are provided in Appendix C under Engineered Soil Mix A.

4.8 Underdrain and Underground Storage Layer

Some Level 2 water quality swale designs will not use an underdrain (where soil infiltration rates meet minimum standards (see Section 6.2 and the design table in Section 3). For Level 2 designs with an underdrain, an underground storage layer, consisting of a minimum 12 inches of stone, shall be incorporated below the invert of the underdrain. The depth of the storage layer will depend on the target treatment and storage volumes needed to meet water quality, channel protection, and/or flood protection criteria. However, the bottom of the storage layer must be at least 2 feet above the seasonally high groundwater table. The storage layer shall consist of clean, washed #57 stone or an approved infiltration module. A water quality swale should include observation wells with cleanout pipes along the length of the swale if the contributing drainage area exceeds 1 acre. The wells shall be tied into any T's or Y's in the underdrain system, and shall extend upwards to be flush with surface, with a vented cap.

4.9 Landscaping and Planting Plan

Designers should choose grasses, herbaceous plants or trees that can withstand both wet and dry periods and relatively high velocity flows for planting within the channel. Salt tolerant grass species should be chosen for water quality swales located along roads. Taller and denser grasses are preferable, although the species is less important than good stabilization and dense vegetative cover. Grass species shall have the following characteristics: a deep root system to resist scouring; a high stem density with well-branched top growth; water-tolerance; resistance to being flattened by runoff; and an ability to recover growth following inundation.

5. Special Case Design Adaptations

5.1 Steep Terrain

In areas of steep terrain, water quality swales can be implemented with contributing slopes of up to 20% gradient if a multiple cell design is used to dissipate erosive energy prior to filtering. This can be accomplished by terracing a series of water quality swale cells to manage runoff across or down a slope. The drop in elevation between cells shall be limited to 1 foot and the splash apron area armored with river stone or a suitable equivalent. A greater emphasis on properly engineered energy dissipaters and/or drop structures is warranted.

6. Construction

6.1 Construction Sequence

Construction Stage Erosion Protection and Sediment Controls. Water quality swales shall be fully protected by silt fence or construction fencing, particularly if they will provide an infiltration function (i.e., have no underdrains). Ideally, water quality swale areas should remain outside the limits of disturbance during construction to prevent soil compaction by heavy equipment.

Water quality swale locations may be used for small sediment traps or basins during construction. However, these must be accompanied by notes and graphic details on the EPSC plan specifying that the maximum excavation depth of the sediment trap/basin at the construction stage must (1) be at least 1 foot above the depth of the post-construction water quality swale installation, (2) contain an underdrain, and (3) specify the use of proper procedures for conversion from a temporary practice to a permanent one, including dewatering, cleanout and stabilization.

6.2 Construction Sequence

The following is a typical construction sequence to properly install a water quality swale, although the steps may be modified to adapt to different site conditions.

Step 1. Protection during Site Construction. As noted above, water quality swales should remain outside the limit of disturbance during construction to prevent soil compaction by heavy equipment. However, this is seldom practical given that swales are a key part of the drainage system at most sites. In these cases, temporary sediment controls such as dikes, silt fences and other similar measures should be integrated into the swale design throughout the construction sequence. Specifically, barriers shall be installed at key check dam locations, erosion control fabric shall be used to protect the channel, and excavation shall be no deeper than 2 feet above the proposed invert of the bottom of the planned underdrain. Water quality swales that lack underdrains (and rely on filtration) must be fully protected by silt fence or construction fencing to prevent compaction by heavy equipment during construction.

Step 2. Installation may only begin after the entire contributing drainage area has been stabilized by vegetation. The designer should check the boundaries of the contributing drainage area to ensure it conforms to original design. Additional EPSC may be needed during swale construction, particularly to divert stormwater from the water quality swale until the filter bed and side slopes are fully stabilized. Pre-treatment cells shall be excavated first to trap sediments before they reach the planned filter beds.

Step 3. Excavators or backhoes shall work from the sides to excavate the water quality swale area to the appropriate design depth and dimensions. Excavating equipment shall be adequate reach so it does not sit inside the footprint of the water quality swale area.

Step 4. The bottom of the water quality swale should be ripped, roto-tilled or otherwise scarified to promote greater infiltration.

Step 5. Place an acceptable filter fabric on the underground (excavated) sides, but not the base, of the water quality swale with a minimum 6-inch overlap. Place filter bed and then place the stone needed for storage layer over the filter bed. Place the perforated underdrain pipe and check its slope. Add the remaining stone jacket, pack ASTM D 448 No. 57 stone to 3 inches above the top of the underdrain, then add 3 inches of pea gravel as a filter layer.

Step 6. Add the soil media in 12-inch lifts without compaction until the desired top elevation of the water quality swale is achieved. Wait a few days to check for settlement, and add additional media as needed.

Step 7. Install check dams, driveway culverts and internal pre-treatment features, as specified in the plan.

Step 8. Prepare planting holes for specified trees and shrubs, install erosion control fabric where needed, spread seed, or lay sod, and install any temporary irrigation.

Step 9. Plant landscaping materials as shown in the landscaping plan, and water them weekly during the first 2 months. The construction contract should include a care and replacement warranty to ensure that vegetation is properly established and survives during the first growing season following construction.

Step 10. Conduct a final construction inspection and develop a punch list for facility acceptance.

6.3 Construction Inspection

Inspections are needed during construction to ensure that the water quality swale is built in accordance with this guidance and project-specific specifications, and certified as-built plans will be required post construction. Detailed inspection checklists should be used that include signoffs by qualified individuals at critical stages of construction, to ensure that the contractor's interpretation of the plan is consistent with the designer's intent. Some common pitfalls can be avoided by careful construction supervision that focuses on the following key aspects of water quality swale installation.

- Check the filter media to confirm that it meets specifications and is installed to the correct depth.
- Check elevations such as the invert of the underdrain, inverts for the inflow and outflow points, and the ponding depth provided between the surface of the filter bed and the overflow structure.
- Ensure that caps are placed on the upstream (but not the downstream) ends of the underdrains.
- Make sure the desired coverage of turf or erosion control fabric has been achieved following construction, both on the filter beds and their contributing side-slopes.
- Inspect check dams and pre-treatment structures to make sure they are properly installed and working effectively.
- Check that outfall protection/energy dissipation measures at concentrated inflow and outflow points are stable.

The real test of a water quality swale occurs after its first big storm. The post-storm inspection should focus on whether the desired sheet flow, shallow concentrated flows or fully concentrated flows assumed in the plan occur in the field. Also, inspectors should check that the water quality swale drains completely within 24-hour drawdown period. Minor adjustments are normally needed because of this post-storm inspection (e.g., spot reseeding, gully repair, add armoring at inlets or outfalls, and check dam realignment).

7. Maintenance

7.1 Maintenance Plans

It is recommended that a Long-Term Maintenance Plan (LTMP) be developed by the design engineer. The LTMP contains a description of the stormwater system components and information on the required inspection and maintenance activities. The BMP owner must maintain and update the BMP operations and maintenance plan.

7.2 Maintenance Inspections

Annual inspections are used to trigger maintenance operations such as sediment removal, spot revegetation and inlet stabilization. The following is a list of several key maintenance inspection points:

- Add reinforcement planting to maintain 95% turf cover or vegetation density. Reseed or replant any dead vegetation.
- Remove any accumulated sand or sediment deposits on the filter bed surface or in pre-treatment cells.
- Inspect upstream and downstream of check dams for evidence of undercutting or erosion and remove trash or blockages at weepholes.
- Examine filter beds for evidence of braiding, erosion, excessive ponding, or dead grass.
- Check inflow points for clogging and remove any sediment.
- Inspect side slopes and grass filter strips for evidence of any rill or gully erosion, and repair as needed.
- Look for any bare soil or sediment sources in the contributing drainage area and stabilize immediately.

Ideally, inspections should be conducted in the spring of each year.

7.3 Routine Maintenance and Operation

Once established, water quality swales have minimal maintenance needs outside of the spring clean-up, regular mowing, and pruning and management of trees and shrubs. The surface of the filter bed can become clogged with fine sediment over time, but this can be alleviated through core aeration or deep tilling of the filter bed. Rake around existing vegetation and replant any vegetation that has not survived. Additional effort may be needed to repair check dams, stabilize inlet points, and remove deposited sediment from pretreatment cells.

8. As-Built Requirements

After the water quality swale has been constructed, the developer must have an as-built certification of the swale prepared by a registered Professional Engineer and provide it to the City Engineer. The as-built certification verifies that the BMP was installed as designed and approved.

The following components must be addressed in the as-built certification:

- Appropriate underdrain system for water quality swales.
- Correctly sized Water Quality Volume.
- Appropriate filter media and stone installed.
- Adequate vegetation in place.
- Overflow system in place for high flows.

9. References

Center for Watershed Protection (CWP). 2007. National Pollutant Removal Performance Database Version 3.0. Center for Watershed Protection, Ellicott City, MD.

City of Fayetteville, 2014. *Drainage Criteria Manual.* Fayetteville, AR. Drainage-Criteria-Manual-Appendix-B-PDF (fayetteville-ar.gov)

Claytor, R. and T. Schueler. 1996. Design of Stormwater Filtering Systems. Center for Watershed Protection. Ellicott City, MD.

Hirschman, D. and J. Kosco. 2008. Managing Stormwater in Your Community: A Guide for Building an Effective Post-Construction Program. EPA Publication 833-R-08-001, Tetra-Tech, Inc. and the Center for Watershed Protection. Ellicott City, MD.

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VADCR (Virginia Department of Conservation and Recreation). 1999. *Virginia Stormwater Management Handbook.* Volumes 1 and 2. Division of Soil and Water Conservation. Richmond, VA.

VADCR. 2011. Stormwater Design Specification No. 10: Dry Swales. Version 1.9. Division of Soil and Water Conservation. Richmond, VA.

WQ-15 Downspout Disconnection

1. Description

This strategy involves managing runoff close to its source by intercepting, infiltrating, filtering, treating, or reusing it as it moves from the impervious surface to the drainage system. Two kinds of disconnection are allowed: (1) simple disconnection, whereby rooftops and/or on-lot residential impervious surfaces are directed to pervious areas, and (2) disconnection leading to an alternative runoff reduction practice(s) adjacent to the roof (Figure C-77). Alternative practices that take up less space can be used where space in not available for the disconnection practices described above.

2. Typical Details

Figures 77 to 79 portray various rooftop disconnect and alternative runoff reduction options.

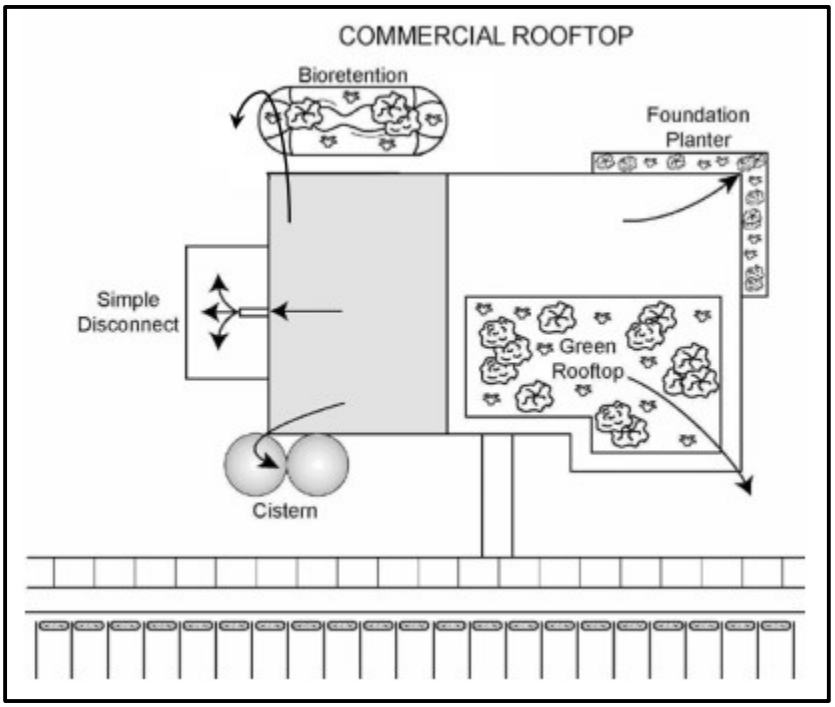


Figure C-77. Roof Disconnection with Alternative Runoff Reduction Practices (Source: VADCR, 2011).

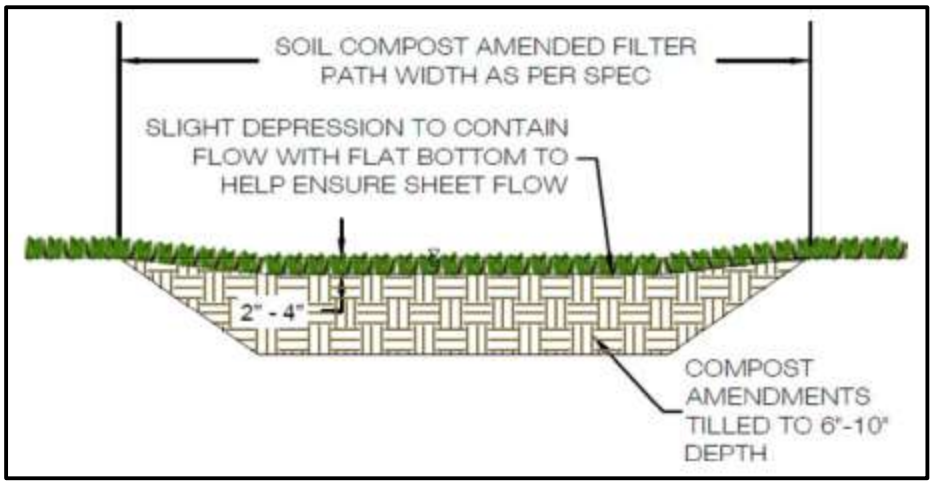


Figure C-78. Disconnection: Soil Compost Amended Filter Path (Source: VADCR, 2011).

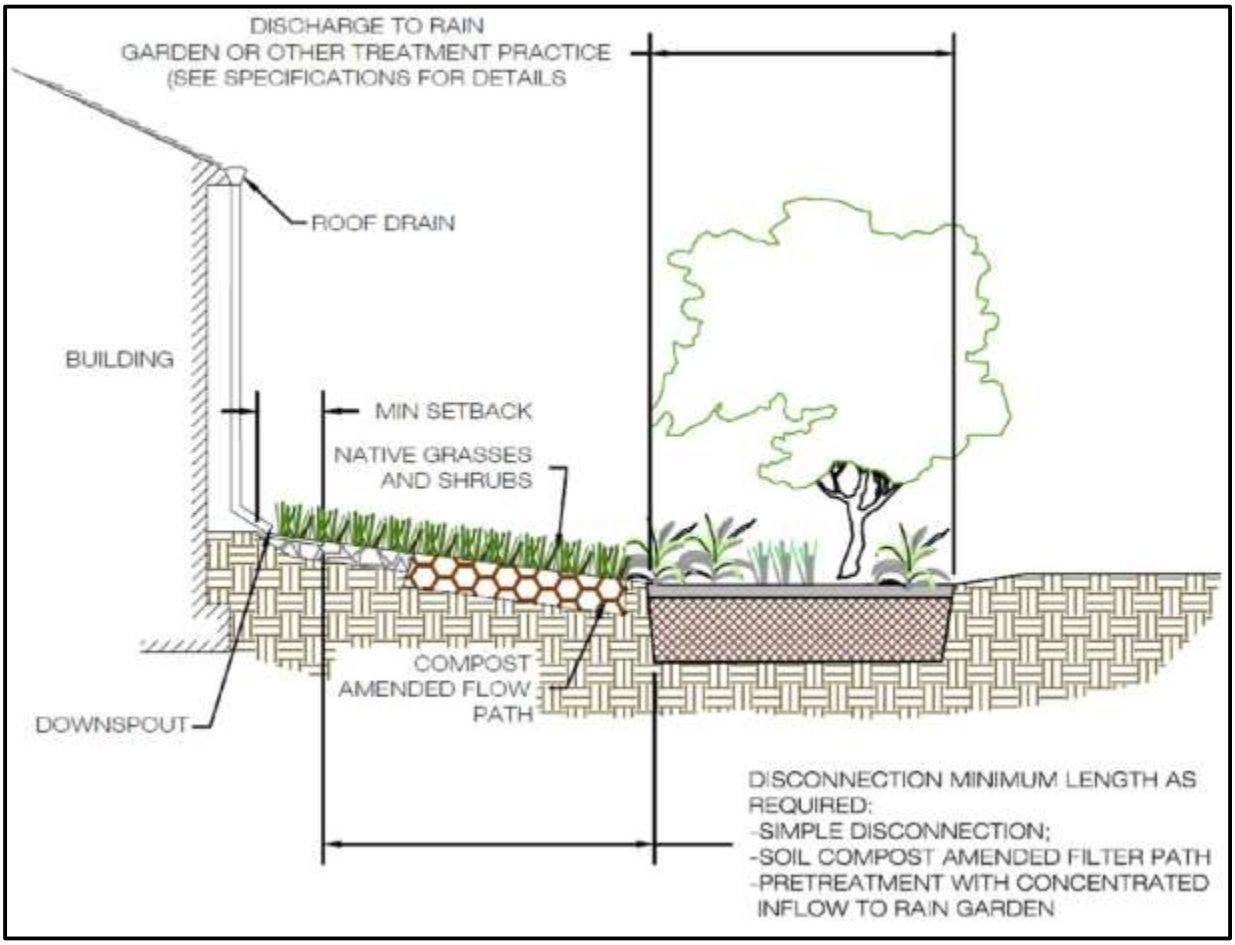


Figure C-79. Rooftop Disconnection – Section View

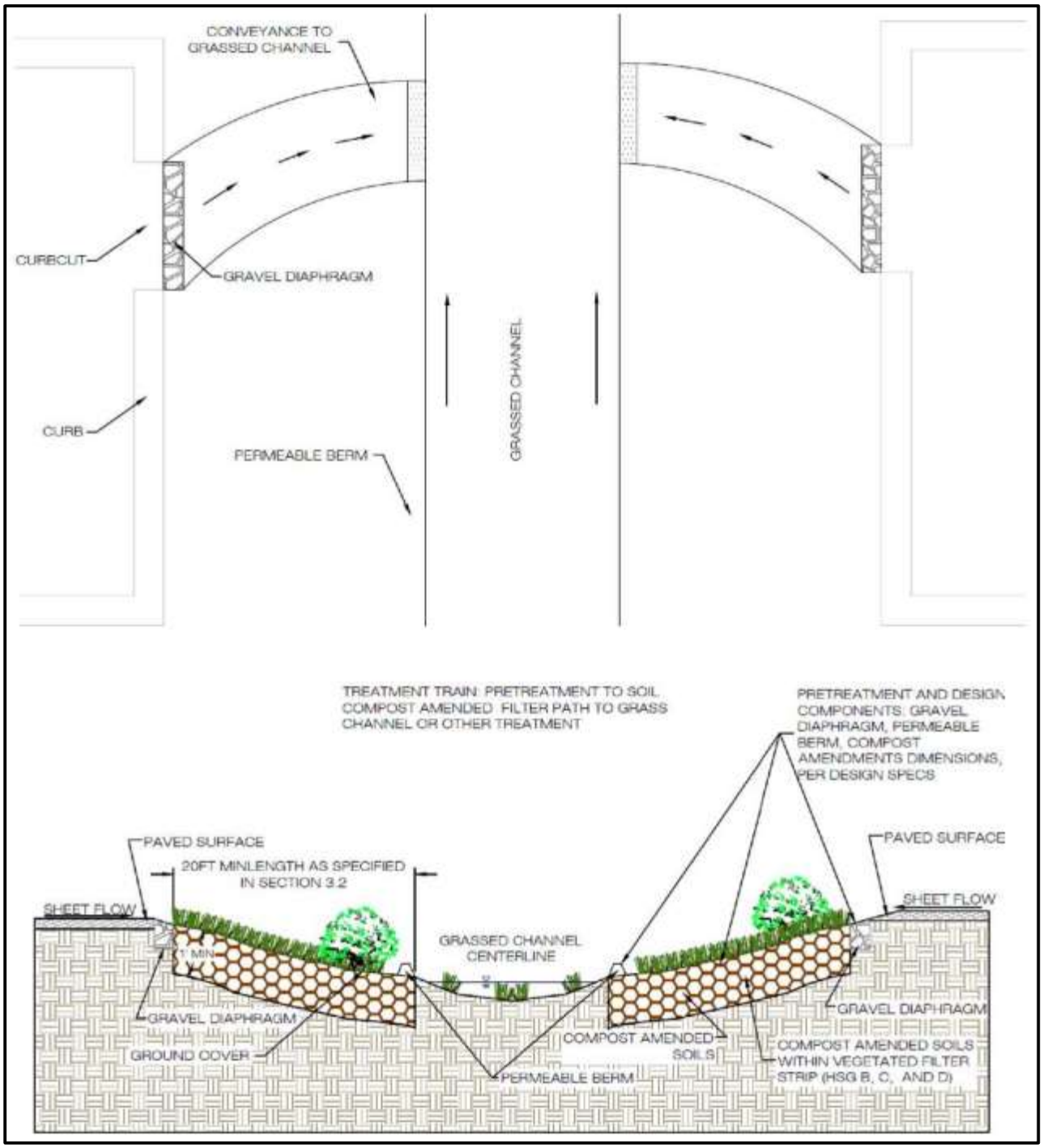


Figure C-80. Amended Filter Path to Downstream Grass Channel (or other treatment) (Source: VADCR, 2011).

3. Design Criteria

3.1 Simple Rooftop Disconnection

Table C-45 provides the primary design criteria for simple rooftop disconnection.

In general:

- Simple disconnection is not credited for residential lots, except in the case of common areas within a development where the longevity and proper maintenance of the downspout disconnect has been ensure as described in page 1 of this GSP.
- No more than half of a roof area (1,000 sq. ft. maximum) shall be discharged to any one point.
- Care shall be taken to locate downspout disconnections in areas that slope away from structures. Downspouts may not be disconnected on slopes over 5%, or within 500 ft of steep slopes or landslide-prone areas. To reduce the possibility of erosion, flow shall also not be directed onto lawns or beds that are sloped more than 15%.
- Simple disconnection can be used on any post-construction Hydrologic Soil Group (HSG). However, for HSG C or D, alternative runoff reduction practices (e.g., compost-amended filter path, bioretention, rainwater harvesting) can boost the runoff reduction rate. Also, erodibility of soils must be considered and, if erosive soils are present, addressed when designing simple disconnection.
- Maintenance of disconnected downspouts usually involves regular lawn or landscaping maintenance in the filter path from the roof to the street. In some cases, runoff from a simple disconnection may be directed to a more natural, undisturbed setting (i.e., where lot grading and clearing is "fingerprinted" and the proposed filter path is protected).

Table C-45. Simple Rooftop Disconnection Design Criteria[1]

Design Factor	Simple Disconnection
Maximum impervious (Rooftop) area treated	1,000 sq ft per disconnection
Longest flow path (roof/gutter)	75 ft
Disconnection length	Equal to longest flow path, but no less than 40 ft[2]
Disconnection slope	< 2%, or < 5% with turf reinforcement[3]
Distance from buildings or foundations	Extend downspouts 5 ft[4] away from building if grade away from building is less than 1%.
Type of pretreatment	External (leaf screens, etc)

1. For alternative runoff reduction practices, see the applicable specification for design criteria. See Table 7.1 herein for eligible practices and associated GSP specification numbers.
2. An alternative runoff reduction practice must be used when the disconnection length is less than 40 ft.
3. Turf reinforcement may include appropriate reinforcing materials that are confirmed by the designer to be non-erosive for the specific characteristics and flow rates anticipated at each individual application and are acceptable to the city.
4. Note that the downspout extension of 5 ft is intended for simple shallow foundations. The use of a dry well or french drain adjacent to an in-ground basement or finished floor area should be coordinated with the design of the structure water-proofing system (foundation drains, etc.), or avoided altogether.

3.2 Soil Compost-Amended Filter Path

The incorporation of compost amendments shall conform to the Soil Amendment portion of Appendix C, and shall include the following design elements:

- Flow from the downspout shall be spread over a 10-ft wide strip extending down-gradient along the flow path from the building to the street or conveyance system.
- The filter path shall be at least 20 ft in length.

- A pea gravel or river stone diaphragm, or other accepted flow spreading device shall be installed at the downspout outlet to distribute flows evenly across the filter path.
- The strip shall have adequate “freeboard” so that flow remains within the strip and is not diverted away from the strip. In general, this means that the strip should be lower than the surrounding land area in order to keep flow in the filter path. Similarly, the flow area of the filter strip shall be level to discourage concentrating of flow in the middle of the filter path.
- Use 2 to 4 inches of compost and till to a depth of 6 to 10 inches within the filter path.

3.3 Infiltration

Infiltration trenches are excavations typically filled with stone to create an underground reservoir for stormwater runoff, allowing the runoff volume to gradually exfiltrate through the bottom and sides of the trench into the subsoil over a 2-day period. By diverting runoff into the soil, infiltration trenches serve to both treat the water quality volume and to preserve the natural water balance on a site. Infiltration systems are limited to areas with highly porous soils where the water table and/or bedrock are located well below the bottom of the trench. The major design goal for Infiltration is to maximize runoff volume reduction. To this end, designers may choose to meet the requirements of a Level 1 baseline design or choose an enhanced design (Level 2) that maximizes runoff reduction, as described in WQ-13 (Infiltration) and summarized in Table C-46.

Table C-46. Level 1 and Level 2 Infiltration Design Guidelines.

Level 1 Design (RR:50)	Level 2 Design (RR:90)
Sizing[1]: WQv = [(Rv)(A)/12] – the volume reduced by an upstream BMP	Sizing: WQv = [1.1(Rv)(A)/12] – the volume reduced by an upstream BMP
At least two forms of pre-treatment (see WQ-13 Table C-34 Pretreatment Elements)	At least three forms of pre-treatment (see **WQ-13** Table **C-34** Pretreatment Elements)
Soil infiltration rate 0.5 to 1 in./hr., number of tests depends on the scale (Appendix C)	Soil infiltration rates of 1.0 to 4.0 in/hr, number of tests depends on the scale (Appendix C)
Minimum of 2 ft between the bottom of the infiltration practice and the seasonal high-water table or bedrock	
WQv infiltrates within 24 to 48 hours	
Setbacks – see suggested minimum setbacks (Section 5.1 of WQ-04, see WQ-05 for urban applications)	
All designs are subject to potential area of high pollutant loading restrictions/prohibitions	

1. A= Area, sq. ft.

3.4 Bioretention

Providing a place for the water to soak in – such as with a compost-amended landscape area (see above), bioretention areas, or rock-filled trench – increases infiltration. Depending on soil properties, roof runoff may be filtered through a shallow bioretention area. The design for this option shall meet the requirements of Bioretention as described in WQ-04 and summarized in Table C-47.

The bioretention media is 24 to 36 inches deep, and for Level 2 is located over a 12 inch deep (or greater) stone reservoir (as required by WQ-04). A perforated underdrain is located above the stone reservoir, to promote storage and recharge in poorly draining soils. In urban settings, the underdrain is directly connected into the storm drainpipe running underneath the street or in the street right-of-way. A trench needs to be excavated during construction to connect the underdrain to the street storm drain system. Appropriate approvals are required for making any connections to a common (or public) drainage system.

Construction of the remainder of the front yard bioretention system is deferred until after the lot has been stabilized. The front yard design should reduce the risk of homeowner conversion because it allows the owners to choose whether they want turf or landscaping. Front yard bioretention requires regular mowing and/or landscape maintenance to perform effectively. It is recommended that the practice be located in an expanded right-of-way or stormwater easement so that it can be accessed in the event that it fails to drain properly.

Table C-47. Bioretention Design Criteria[1].

Design Factor	Bioretention
Impervious Area Treated	1,000 ft^2
Type of Inflow	Sheet flow or roof leader
Minimum Soil Infiltration Rate	0.5 inches/hour (or use underdrain)
Observation Well/ Cleanout Pipes	No
Type of Pretreatment	External (leaf screens, etc)
Underdrain and gravel layer	Level 1: Yes; Level 2: Optional per soils1
Minimum Filter Media Depth	24 inches (Level 1); 36 inches (Level 2)
Media Source	Mixed on-site
Hydraulic Head Required	1 to 3 ft
Required Soil Borings	One test pit/standard soil boring for every 1,000 sq ft only when an underdrain is not used
Building Setbacks	10 ft down-gradient, 50 ft up-gradient

1. Refer to WQ-10 for Level 1 and Level 2 Design Criteria, and sizing criteria for individual and multiple downspout applications.

3.5 Rain Tanks and Cisterns

The runoff reduction rates for rain tanks and cisterns depend on their storage capacity and ability to draw down water in between storms for reuse as grey-water or irrigation use. All devices shall have a suitable overflow area to route extreme flows into the next treatment practice or the stormwater conveyance system.

3.6 Stormwater Planter (Urban Bioretention)

This form of disconnection must conform to the design requirements for stormwater planters, as outlined in WQ-05, Urban Bioretention. Foundation planters are a useful option to disconnect and treat rooftop runoff, particularly in ultra-urban areas. They consist of confined planters that store and/or infiltrate runoff in a soil bed to reduce runoff volumes and pollutant loads. Stormwater planters combine an aesthetic landscaping feature with a functional form of stormwater treatment. Stormwater planters generally receive runoff from adjacent rooftop downspouts and are landscaped with plants that are tolerant to periods of both drought and inundation. The two basic design variations for stormwater planters are the infiltration planter and the filter planter.

An ***infiltration planter*** filters rooftop runoff through soil in the planter followed by infiltration into soils below the planter. The recommended minimum depth is 30 inches, with the shape and length determined by architectural considerations. The planter shall be sized to temporarily store at least 0.5 inch of runoff from the contributing rooftop area in a reservoir above the planter bed. Infiltration planters shall be placed at least 10 ft away from a building to prevent possible flooding or basement seepage damage.

A ***filter planter*** has an impervious liner on the bottom. The minimum planter depth is 30 inches, with the shape and length determined by architectural considerations. Runoff is temporarily stored in a reservoir located above the planter bed. Overflow pipes are installed to discharge runoff when maximum ponding depths are exceeded, to avoid water spilling over the side of the planter. In addition, an underdrain is used to carry runoff to the storm sewer system. Since a filter planter is self-contained and does not infiltrate into the ground, it can be installed with no setback, or right next to a building.

All planters shall be placed at grade level or above ground. They shall be sized to allow captured runoff to drain out within four hours after a storm event. Plant materials shall be capable of withstanding seasonally moist and dry conditions. Planting media shall have an infiltration rate of at least 2 inches per hour. The sand and gravel on the bottom

of the planter shall have a minimum infiltration rate of 5 inches per hour. The planter can be constructed of stone, concrete, brick, wood, or other durable material.

4. Maintenance

The rooftop disconnection and supplementary treatment device must be covered by a drainage easement to allow inspection and maintenance and must also be included in the Long-Term Maintenance Plan (LTMP) for the site. The LTMP shall include an inspection checklist for Downspout Disconnection.

5. As-Built Requirements

During and after the downspout disconnection construction, the developer must have adequate inspection and an as-built certification prepared by a registered Professional Engineer and provided to the city. The as-built certification verifies that the BMP was installed as designed and approved.

The following components must be addressed in the as-built certification:

1. Ensure disconnect is treating appropriate area size from either sheet flow or roof leader.
2. Ensure filter media depth is properly sized.
3. Ensure building setbacks are 10 ft down-gradient, 50 ft up-gradient.
4. Ensure underdrain and gravel layer (if required) have been properly installed.
5. If alternative practices have been utilized, insure that as-built requirements for those BMPs are also certified using the as-built section of the utilized BMP.

6. References

City of Fayetteville, 2014. *Drainage Criteria Manual.* Fayetteville, AR. Drainage-Criteria-Manual-Appendix-B-PDF (fayetteville-ar.gov)

City of Philadelphia, 2011 revision. *Philadelphia Stormwater Management Guidance Manual*.

City of Portland, Environmental Services, 2004*. Portland Stormwater Management Manual*. Portland, OR.

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Northern Virginia Regional Commission. 2007. Low Impact Development Supplement to the Northern Virginia BMP Handbook. Fairfax, Virginia.

Schueler, T., D. Hirschman, M. Novotney and J. Zielinski. 2007. *Urban Stormwater Retrofit Practices. Manual 3 in the Urban Subwatershed Restoration Manual Series*. Center for Watershed Protection, Ellicott City, MD.

Schueler, T. 2008. Technical Support for the Baywide Runoff Reduction Method. Chesapeake Stormwater Network. Baltimore, MD. www.chesapeakestormwater.net

VADCR (Virginia Department of Conservation and Recreation) 2011. Stormwater Design Specification No. 1: Rooftop (Impervious Surface) Disconnection, Version 1.9.

WQ-16 Grass Channel

1. Description

Grass channels are conveyance channels that are designed to provide some treatment of runoff, as well as to slow runoff velocities for treatment in other structural controls. Grass channels are appropriate for a number of applications including treating runoff from paved roads and from other impervious areas.

Grass channels can provide a modest amount of runoff filtering and volume attenuation within the stormwater conveyance system resulting in the delivery of less runoff and pollutants than a traditional system of curb and gutter, storm drain inlets and pipes. The performance of grass channels will vary depending on the underlying soil permeability. Grass channels, however, are not capable of providing the same stormwater functions as water quality swales as they lack the storage volume associated with the engineered soil media. Their runoff reduction performance can be increased when compost amendments are added to the bottom of the swale. Grass channels are a preferable alternative to both curb and gutter and storm drains as a stormwater conveyance system, where development density, topography, and soils permit.

2. Design Table

Grass channels must meet the minimum criteria outlined in Table C-49.

Table C-49. Grass Channel Design Guidance.

Design Criteria
• The bottom width of the channel shall be between 4 to 8 ft wide. • The channel side slopes shall be 3H:1V or flatter. • The maximum total contributing drainage area to any individual grass channel is 5 acres. • The longitudinal slope of the channel shall be no greater than 4 %. • Check dams may be used to increase residence time. • The maximum flow velocity of the channel must be less than 1 ft per second during a 1-inch storm event. • The channel shall be designed such that flow velocity is non-erosive during the 2-year and 10-year design storm events and the 10-year design flow is contained within the channel (minimum of 6 inches of freeboard). • The vegetation used must withstand relatively high velocities as well as a range of moisture conditions from very wet to dry.

3. Typical Details

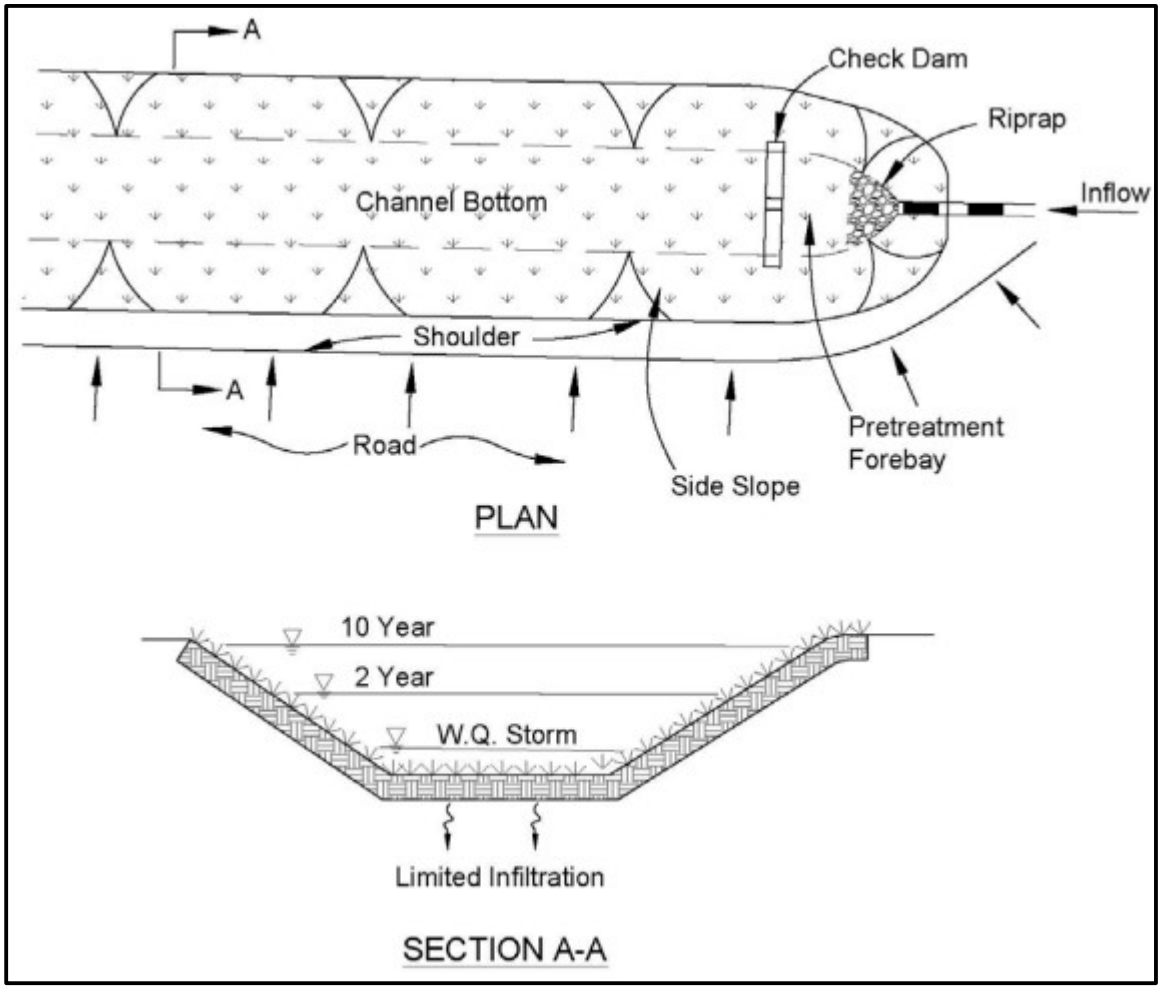


Figure C-81. Grass Channel – Typical Plan and Section.

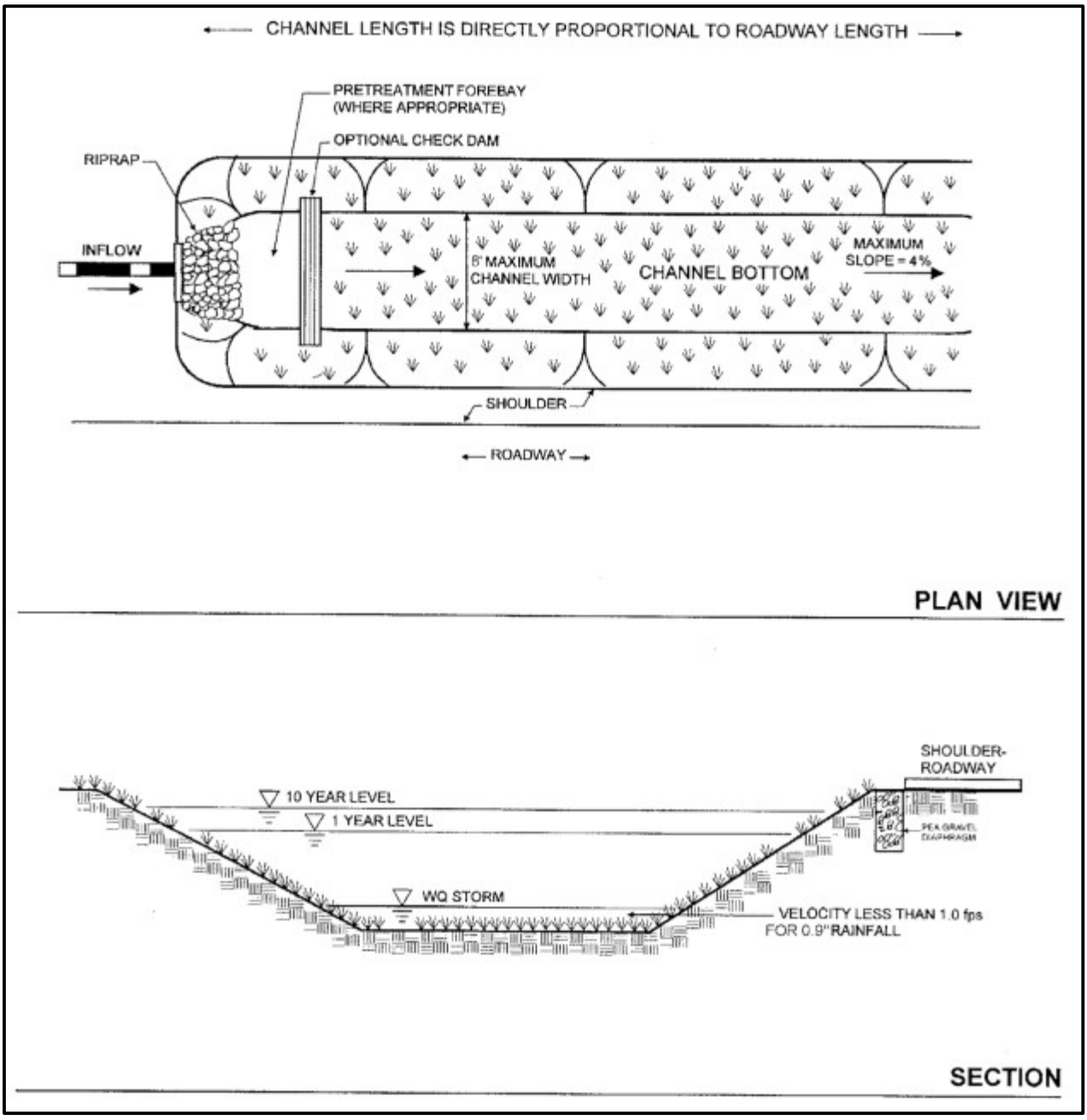


Figure C-82. Grass Channel along Roadway – Typical Plan and Section (Source: VADCR, 2011).

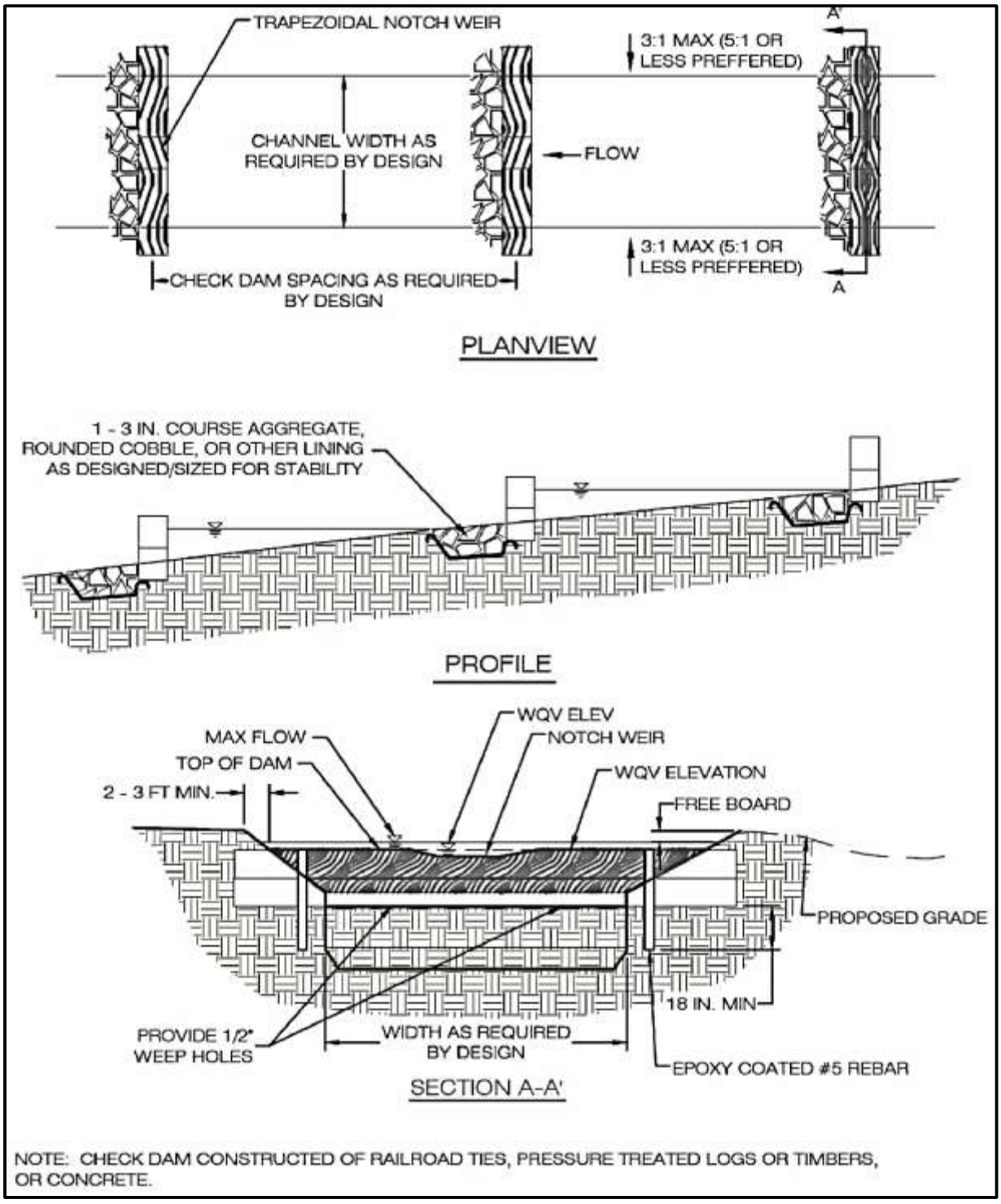


Figure C-83. Grass Channel with Check Dams – Typical Plan, Profile, and Section. (Source: VADCR, 2011).

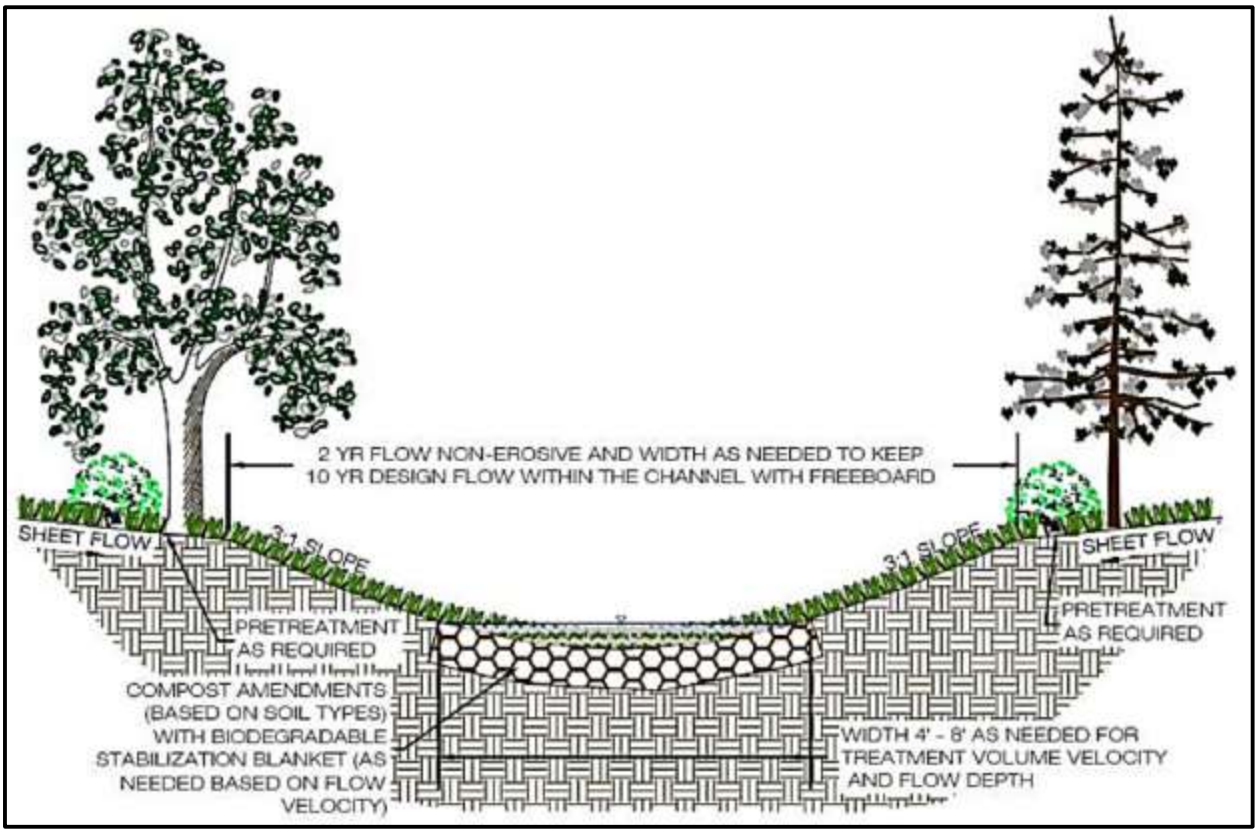


Figure C-84. Grass Channel with Compost Amendments – Section. (Source: VADCR, 2011).

4. Physical Feasibility and Design Applications

Grass channels can be implemented on development sites where development density, topography and soils are suitable. The linear nature of grass channels makes them well-suited to treat highway runoff, low and medium density residential road runoff and small commercial parking areas or driveways. However, a Water Quality Swale (WQ-14) will provide much greater runoff reduction and pollutant removal performance.

Key constraints for grass channels include:

Land Use. Grass channels can be used in residential, commercial, or institutional development settings. However, when grass channels are used for both conveyance and water quality treatment, they shall be applied only in linear configurations parallel to the contributing impervious cover, such as roads and small parking areas.

- Large commercial site applications may require multiple channels to effectively break up the drainage areas and meet the design criteria.
- The linear nature of grass channels makes them well suited to treat highway or low- and medium density residential road runoff if there is adequate right-of-way width and distance between driveways.

Contributing Drainage Area. The drainage area (contributing or effective) to a grass channel must be 5 acres or less. When grass channels treat and convey runoff from drainage areas greater than 5 acres, the velocity and flow depth through the channel becomes too great to treat runoff or prevent channel erosion.

Available Space. Grass channels can be incorporated into linear development applications (e.g., roadways) by utilizing the footprint typically required for an open section drainage feature. The footprint required will likely be greater than that

of a typical conveyance channel. However, the benefit of the runoff reduction may reduce the footprint requirements for stormwater management elsewhere on the development site.

Longitudinal Slope. Grass channels shall be designed with slopes of less than 4%. Slopes steeper than 4% create rapid runoff velocities that can cause erosion and do not allow enough contact time for infiltration or filtering, unless check dams are used. Slopes of 1-2% are recommended, and slopes of less than 2% may eliminate the need for check dams. Channels designed with longitudinal slopes of less than 1% shall be monitored carefully during construction to ensure a continuous grade, to avoid flat areas with pockets of standing water.

Soils. Grass channels can be used on most soils with some restrictions on the most impermeable soils. Grass channels situated on Hydrologic Soil Group C and D soils will require compost amendments in order to improve performance. Grass channels shall not be used on soils with infiltration rates less than 0.5 inches per hour if infiltration of small runoff flow is intended.

Hydraulic Capacity. Grass channels are an on-line practice and must be designed with enough capacity to convey runoff from the 10-year design storm event within the channel banks and be non-erosive during both the 2-year and 10-year design storm events. Larger flows shall be accommodated by the channel if dictated by the surrounding conditions. Refer to the applicable chapter(s) within the Drainage Criteria Manual for site hydraulic design requirements.

Depth to Water Table. Designers shall ensure that the bottom of the grass channel is at least 2 ft above the known seasonally high-water table to prevent a moist swale bottom and ensure that groundwater does not intersect the filter bed and possibly lead to groundwater contamination or practice failure.

Utilities and Building Setbacks. Designers shall consult local utility design guidance for the horizontal and vertical clearance between utilities and the channels. Typically, utilities can cross grass channels if they are specially protected (e.g., double casing) or are located below the channel invert. Arkansas One Call (811) shall be contacted before digging onsite begins. Grass channels shall be set back at least 10 ft down-gradient from building foundations, 50 ft from septic system fields and 100 ft from private wells.

Forebays. A forebay is recommended in order to minimize the volume of sediment in the channel. Refer to Chapter 7 of the Drainage Criteria Manual for additional information.

5. Design Criteria

5.1 Sizing of Grass Channels

Unlike other stormwater practices, grass channels are designed based on a peak rate of flow. Designers must demonstrate channel conveyance and treatment capacity in accordance with the following guidelines:

- The longitudinal slope of the channel should ideally be between 1% and 2% in order to avoid scour and short-circuiting within the channel. Longitudinal slopes up to 4% are acceptable; however, check dams will likely be required in order to meet the allowable maximum flow velocities.
- A minimum residence time is of five minutes is required.
- Hydraulic capacity shall be verified using Manning's Equation or an accepted equivalent method, such as erodibility factors and vegetal retardance (NOVA 2007).
    - The Flow Depth for the peak water quality volume shall be maintained at 3 inches or less.
    - Manning's "n" value shall be selected based on depth of flow, for overland flow or channel flow using the appropriate reference tables in Chapter 3.
    - Velocities and Peak Flow Rates for the 2-year and 10-year frequency storms must be non-erosive, and the 10-year peak flow must be contained within the channel banks (with a minimum of 6 inches of freeboard).
- Larger flows shall be accommodated by the channel if dictated by the surrounding conditions. Refer to the applicable chapter(s) within the Drainage Criteria Manual for site hydraulic design requirements.

- Calculations for peak flow depth and velocity shall reflect any increase in flow along the length of the channel, as appropriate. If a single flow is used, the flow at the outlet shall be used.
- The minimum length may be achieved with multiple swale segments connected by culverts with energy dissipaters.

Table C-50. Maximum Permissible Velocities for Grass Channels[1]

Cover Type	Erosion Resistant Soils (ft/sec)	Highly Erodible Soils (ft/sec)
Bermuda grass	6	4.5
Buffalo grass Kentucky bluegrass Reed canary grass Tall fescue	5	3.8
Grass-legume mixture	4	3
Red Fescue	2.5	1.9

1. Source: VADCR, 2011

5.2 Geometry and Site Layout

- Grass channels shall generally be aligned adjacent to and the same length (minimum) as the contributing drainage area identified for treatment.
- Grass channels shall be designed with a trapezoidal or parabolic cross section with relatively flat side slopes. A parabolic shape is preferred for aesthetic, maintenance, and hydraulic reasons.
- The bottom width of the channel shall be between 2 to 6 ft wide. If a channel will be wider than 8 ft, the designer shall incorporate benches, check dams, level spreaders or multi-level cross sections to prevent braiding and erosion along the channel bottom. The bottom width is a dependent variable in the calculation of velocity based on Manning's equation. If a larger channel is needed, the use of a compound cross section is recommended.
- Grass channel side slopes shall be no steeper than 4 H:1 V for ease of mowing and routine maintenance. Flatter slopes are encouraged, where adequate space is available, to aid in pretreatment of sheet flows entering the channel. Under no circumstances are side slopes to exceed 3 H:1 V.

5.3 Check Dams

Check dams may be used for pre-treatment, to break up slopes, and to increase the hydraulic residence time in the channel. Design requirements for check dams are as follows:

- Check dams shall be spaced based on the channel slope, as needed to increase residence time, provide WQv storage volume, or any additional volume attenuation requirements. The ponded water at a downhill check dam should not touch the toe of the upstream check dam.
- The maximum desired check dam height is 12 inches (for maintenance purposes). The average ponding depth throughout the channel shall be 12 inches.
- Armoring may be needed at the downstream toe of the check dam to prevent erosion.
- Check dams must be firmly anchored into the side-slopes to prevent outflanking; check dams must also be anchored into the channel bottom to prevent hydrostatic head from pushing out the underlying soils.
- Check dams must be designed with a center weir sized to pass the channel design storm peak flow (10-year storm event for man-made channels).
- The check dam shall be designed so that it facilitates easy mowing.
- Each check dam shall have a weep hole or similar drainage feature so it can dewater after storms.
- Check dams shall be composed of wood, concrete, stone, or other non-erodible material, or shall be configured with elevated driveway culverts.
- Individual channel segments formed by check dams or driveways shall generally be 25 to 40 ft in length.

5.4 Grass Channel Material Specifications

The basic material specifications for grass channels are outlined in Table C-51 below.

Table C-51. Grass Channel Material Specifications

Component	Specification
Grass	A dense cover of water-tolerant, erosion-resistant grass. The selection of an appropriate species or mixture of species is based on several factors including climate, soil type, topography and sun or shade tolerance. Grass species should have the following characteristics: a deep root system to resist scouring; a high stem density with well-branched top growth; water-tolerance; resistance to being flattened by runoff; an ability to recover growth following inundation; and, if receiving runoff from roadways, salt-tolerance.
Check Dams	• All check dams shall be constructed of non-erosive material such as wood, gabions, riprap or concrete, and underlain with filter fabric conforming to local design standards. • Wood used for check dams shall consist of pressure treated logs or timbers, or water-resistant tree species such as cedar, hemlock, swamp oak or locust. • Computation of check dam material is necessary, based on the surface area and depth used in the design computations.
Diaphragm	Pea gravel used to construct pre-treatment diaphragms shall consist of washed, open-graded, coarse aggregate between 3 and 10 mm in diameter and must conform to local design standards.
Erosion Control Fabric	Where flow velocities dictate, biodegradable erosion control netting or mats that are durable enough to last at least two growing seasons must be used.
Filter Fabric (check dams)	Needled, non-woven, polypropylene geotextile meeting the following specifications: Grab Tensile Strength (ASTM D4632): > 120 lbs Mullen Burst Strength (ASTM D3786): > 225 lbs/sq in Flow Rate (ASTM D4491): > 125 gpm/sq ft Apparent Opening Size (ASTM D4751): US #70 or #80 sieve

5.5 Grass Channel as Pretreatment

Several structural controls such as bioretention areas and infiltration trenches may be supplemented by a grass channel that serves as pretreatment for runoff flowing to the device. The lengths of grass channels vary based on the drainage area imperviousness and slope. Channels must be no less than 20 ft long. Table C-52 below gives the minimum lengths for grass channels based on slope and percent imperviousness.

Table C-52. Grass Channel Length Guidance[1]

Parameter	<=33% Impervious		Between 33% and 67% Impervious		>=67% Impervious	
Slope (max = 4%)	< 2%	> 2%	< 2%	> 2%	< 2%	> 2%
Grass channel minimum length (ft) [2]	25	40	30	45	35	50

1. Source: Georgia Stormwater Management Manual.
2. Assumes 2-ft bottom width.

5.6 Compost Soil Amendments

Soil compost amendments serve to increase the runoff reduction capability of a grass channel. The following design criteria apply when compost amendments are used:

- The compost-amended strip shall extend over the length and width of the channel bottom.
- The amended area shall be stabilized with grass.

- Depending on the slope of the channel, it may be necessary to install a protective biodegradable geotextile fabric to protect the compost-amended soils. Care must be taken to consider the erosive characteristics of the amended soils when selecting an appropriate geotextile.
- For redevelopment or retrofit applications, the final elevation of the grass channel (following compost amendment) must be verified as meeting the original design hydraulic capacity.

5.7 Planting Grass Channels

Designers shall specify grass species that can withstand both wet and dry periods as well as relatively high velocity flows within the channel. For applications along roads and parking lots, salt tolerant species should be chosen. Taller and denser grasses are preferable, though the species of grass is less important than good stabilization. Grass channels shall be seeded at such a density to achieve a 90 % turf cover after the second growing season. Grass channels shall be seeded and not sodded. Seeding establishes deeper roots and sod may have muck soil that is not conducive to infiltration (Wisconsin DNR, 2007). Grass channels shall be protected by a biodegradable erosion control fabric to provide immediate stabilization of the channel bed and banks.

6. Special Case Design Adaptations

Steep Terrain

Grass swales are not practical in areas of steep terrain, although terracing a series of grass swale cells may work on slopes from 5% to 10%. The drop in elevation between check dams shall be limited to 18 inches in these cases, and the check dams shall be armored on the down-slope side with suitably sized stone to prevent erosion.

7. Construction

7.3 Construction Sequence

The following is a typical construction sequence to properly install a grass channel, although steps may be modified to reflect different site conditions. Grass channels shall be installed at a time of year that is best to establish turf cover without irrigation.

Step 1. Protection during Site Construction. Ideally, grass channels shall remain outside the limit of disturbance during construction to prevent soil compaction by heavy equipment. However, this is seldom practical, given that the channels are a key part of the drainage system at most sites. In these cases, temporary EPSC such as dikes, silt fences and other erosion control measures shall be integrated into the swale design throughout the construction sequence. Specifically, barriers shall be installed at key check dam locations, and erosion control fabric shall be used to protect the channel.

Step 2. Grass channel installation may only begin after the entire contributing drainage area has been stabilized with vegetation. Any accumulation of sediments that does occur within the channel must be removed during the final stages of grading to achieve the design cross-section. Erosion and sediment control measures shall be installed as specified in the erosion and sediment control plan. Stormwater flows must not be permitted into the grass channel until the bottom and side slopes are fully stabilized.

Step 3. Grade the grass channel to the final dimensions shown on the plan.

Step 4. Install check dams, driveway culverts and internal pre-treatment features as shown on the plan. Fill material used to construct check dams shall be placed in 8- to 12-inch lifts and compacted to prevent settlement. The top of each check dam shall be constructed level at the design elevation.

Step 5. (Optional). Till the bottom of the channel to a depth of 1 ft and incorporate compost amendments.

Step 6. Add soil amendments as needed, hydro-seed the bottom and banks of the grass channel and peg in erosion control fabric or blanket where needed. After initial planting, a biodegradable erosion control fabric shall be used.

Step 7. Prepare planting holes for any trees and shrubs, then plant materials as shown in the landscaping plan and water them weekly in the first two months. The construction contract shall include a Care and Replacement Warranty to ensure vegetation is properly established and survives during the first growing season following construction.

Step 8. Conduct the final construction inspection and develop a punch list for facility acceptance.

7.4 Construction Inspection

Inspections during construction are needed to ensure that the grass channel is built in accordance with these specifications. Some common pitfalls can be avoided by careful post-storm inspection of the grass channel:

- Make sure the desired coverage of turf or erosion control fabric has been achieved following construction, both on the channel beds and their contributing side-slopes.
- Inspect check dams and pre-treatment structures to make sure they are at correct elevations, are properly installed, and are working effectively.
- Make sure outfall protection/energy dissipation at concentrated inflows are stable.

The real test of a grass channel occurs after its first big storm. Minor adjustments are normally needed as part of this post-storm inspection (e.g., spot reseeding, gully repair, added armoring at inlets or realignment of outfalls and check dams).

8. Maintenance

8.1 Ongoing Maintenance

Once established, grass channels have minimal maintenance needs outside of the spring cleanup, regular mowing, repair of check dams and other measures to maintain the hydraulic efficiency of the channel and a dense, healthy grass cover.

Table C-53. Suggested Spring Maintenance Inspections/Cleanups for Grass Channels[1]

Activity
Add reinforcement planting to maintain 90% turf cover. Reseed any dead vegetation.
Remove any accumulated sand or sediment deposits behind check dams.
Inspect upstream and downstream of check dams for evidence of undercutting or erosion and remove any trash or blockages at weepholes.
Examine channel bottom for evidence of erosion, braiding, excessive ponding or dead grass.
Check inflow points for clogging and remove any sediment.
Inspect side slopes and grass filter strips for evidence of any rill or gully erosion and repair.
Look for any bare soil or sediment sources in the contributing drainage area and stabilize immediately.

1. Source: VADCR, 2011.

8.2 Maintenance

It is recommended that a Long-Term Maintenance Plan (LTMP) be developed by the design engineer. The LTMP contains a description of the stormwater system components and information on the required inspection and maintenance activities.

Basic maintenance requirements for grass channels include the following:

- Maintain grass height of 3 to 4 inches.
- Remove sediment build up in channel bottom when it accumulates to 25% of original total channel volume.
- Ensure that rills and gullies have not formed on side slopes. Correct if necessary.
- Remove trash and debris build up.
- Replant areas where vegetation has not been successfully established.

All grass channels must be covered by a drainage easement to allow inspection and maintenance. If a grass channel is located in a residential private lot, the existence and purpose of the grass channel shall be noted on the deed of record.

9. As-Built Requirements

After the grass channel has been constructed, an as-built certification of the grass channel must be prepared by a registered Professional Engineer and submitted to the city. The as-built certification verifies that the BMP was installed as designed and approved.

The following components must be addressed in the as-built certification:

1. The channel must be adequately vegetated.
2. The channel flow velocities must not exceed allowable velocities for design storms.
3. A mechanism for overflow for large storm events must be provided.

10. Roadway Application

Grass-lined channels have been widely used in roadway drainage systems for many years. They are easily constructed and maintained and work well in a variety of climates and soil conditions. Grass channels are applicable to:

- Major Thoroughfares (Interstates and Other Freeways).
- Major Urban Streets (Principal Arterials, Minor Arterials and Collectors).
- Local Roads.



Figure C-85. Typical Grass Channel



Figure C-86. Roadside Channel in Spokane, WA (Source: VADCR, 2011).

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WQ-17 Vegetated Filter Strips

1. Description

Filter strips are vegetated areas that treat sheet flow delivered from adjacent impervious areas by slowing runoff velocities and allowing sediment and attached pollutants to settle and/or be filtered by the vegetation. The two design variants of filter strips are (1) Conserved Open Space and (2) designed Vegetated Filter Strips. The design, installation, and management of these design variants are quite different, as outlined in this specification.

In both instances, stormwater must enter the filter strip or conserved open space as sheet flow. If the inflow is from a pipe or channel, an engineered level spreader must be designed in accordance with the criteria contained herein to convert the concentrated flow to sheet flow.

2. Design Table

Conserved Open Space and Vegetated Filter Strips each must meet the appropriate minimum criteria outlined in Tables C-54 and C-55 and Section 5 to qualify for the indicated level of runoff reduction. In addition, designers must conduct a site reconnaissance prior to design to confirm topography and soil conditions.

Table C-55. Sheet Flow Design Guidance.

Design Issue	Conserved Open Space	Vegetated Filter Strip
Soil and Vegetative Cover (Sections 6.1 and 6.2)	Undisturbed soils and native vegetation	Amended soils and dense turf cover or landscaped with herbaceous cover, shrubs, and trees
Overall Slope1 and Width (perpendicular to the flow) (Section 5)	Maximum flow length[2] of 150 ft. from adjacent pervious areas; Maximum flow length of 75 ft. from adjacent impervious areas	
Concentrated Flow (Section 6.3)	Length of ELS[6] Lip = 13 lin. ft. per each 1 cfs of inflow if area has 90% Vegatative Cover[3] Length = 40 lin. ft. per 1 cfs where discharge to conserved open space (forest/re-forested)[4]	Length of ELS6 Lip = 13 lin.ft. per each 1 cfs of inflow (13 lin.ft. min; 130 lin.ft. max.)
Construction Stage (Section 7)	Located outside the limits of disturbance and protected by erosion and sediment control	Prevent soil compaction by heavy equipment
Typical Applications (Section 5)	Adjacent to stream or wetland buffer or forest conservation area	Treat small areas of impervious cover (e.g., 5,000 sq. ft.) close to source
Compost Amendments (Section 6.1)	No	Yes (B, C, and D Soils)[5]
Boundary Spreader (Section 6.3)	GD [6] at top of filter	GD[6] at top of filter PB[6] at toe of filter

1. A minimum of 1 % is recommended to ensure positive drainage.
2. For Conservation Areas with a varying slope, a pro-rated length may be computed only if the first 10 ft. is 2% or less.
3. Vegetative Cover is described in Section 6.2.
4. Where the Conserved Open Space is a mixture of native grasses, herbaceous cover, and forest (or re-forested area), the length of the Engineered Level Spreader lip can be established by computing a weighted average of the lengths required for each vegetation type. Refer to Section 6.3 for design criteria.
5. The city may waive the requirement for compost amended soils for filter strips on B soils if adequate infiltration characteristics are met post-construction (see Section 6.1).
6. ELS = Engineered level spreader; GD = Gravel Diaphragm; PB = Permeable Berm.

3. Typical Details

Figure C-87 shows a typical approach for sheet flow to a Conserved Open Space (Cappiella et al., 2006). Figures C-88 and C-89 provide standard details for an engineered level spreader developed by North Carolina State University (Hathaway and Hunt, 2006).

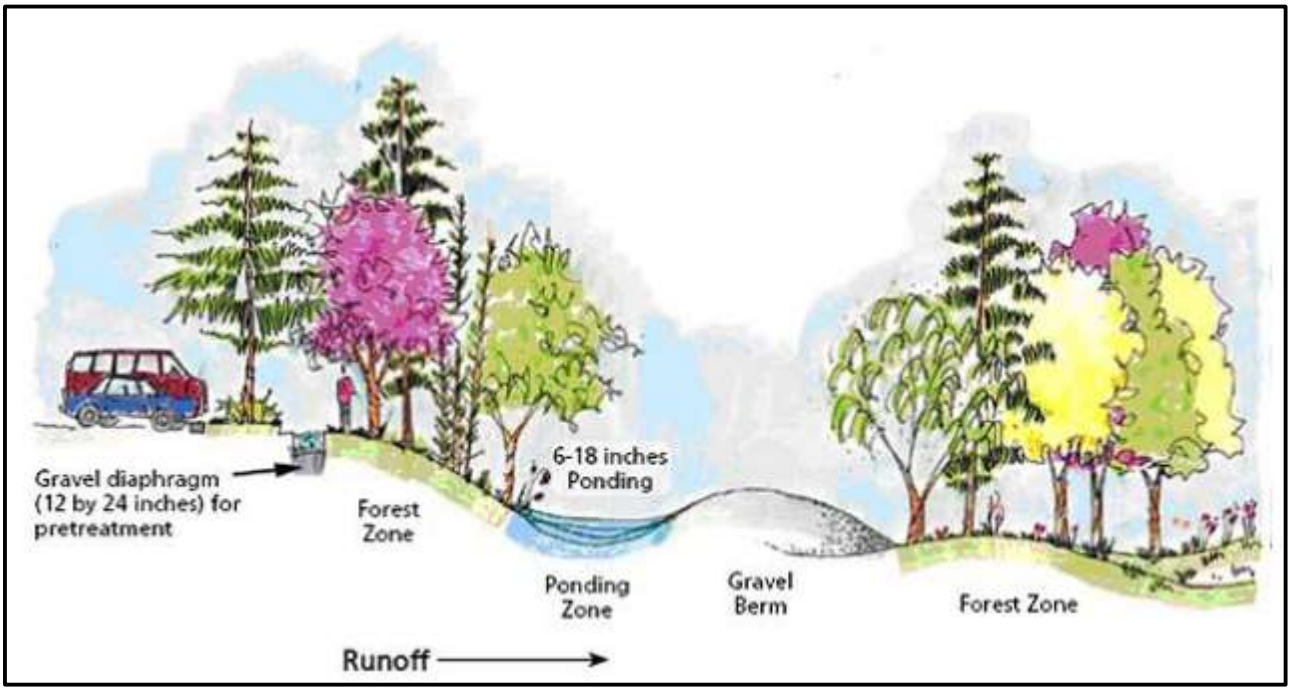


Figure C-87. Typical Sheet flow to Conserved Open Space (Source: Hathaway and Hunt 2006).

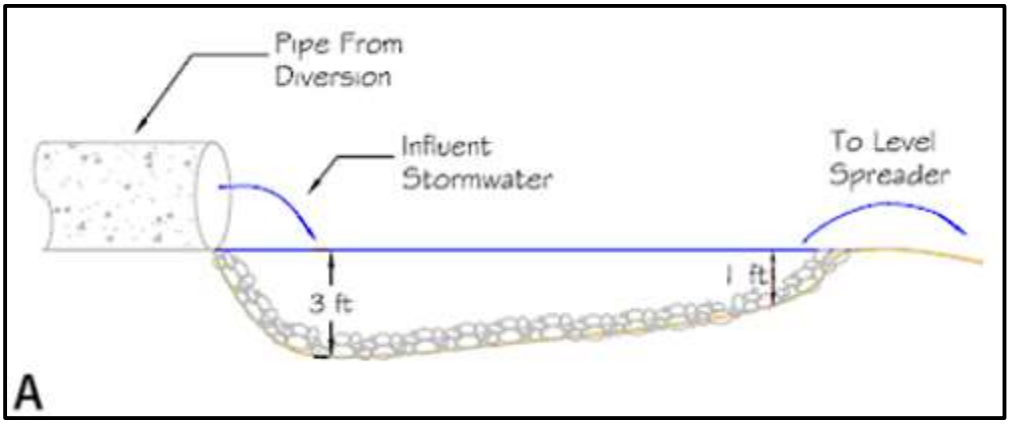


Figure C-88. Level Spreader Forebay (Source: Hathaway and Hunt 2006).

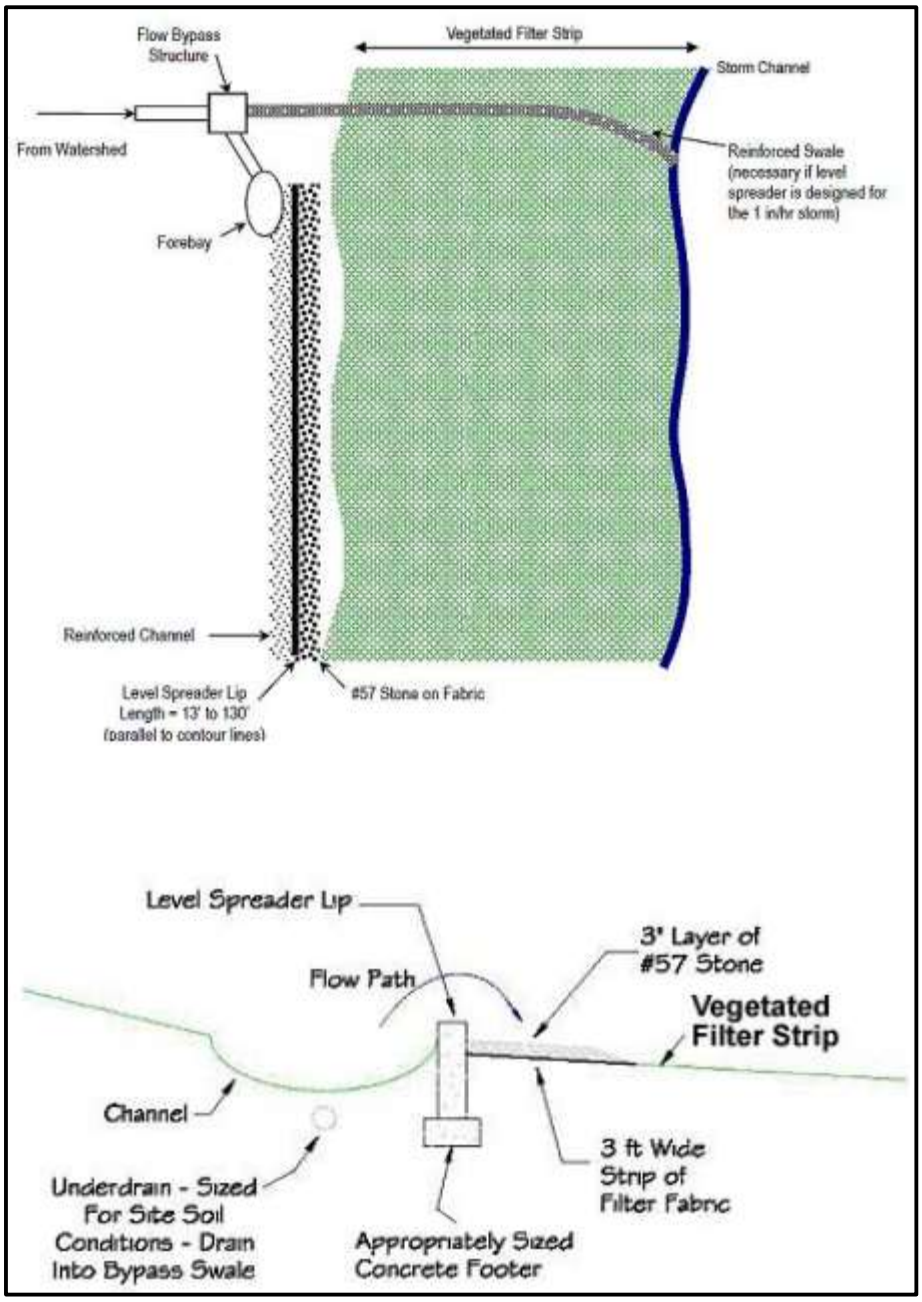


Figure C-89. Plan and Cross Section of Engineered Level Spreader (ELS) (Source: Hathaway 2006).

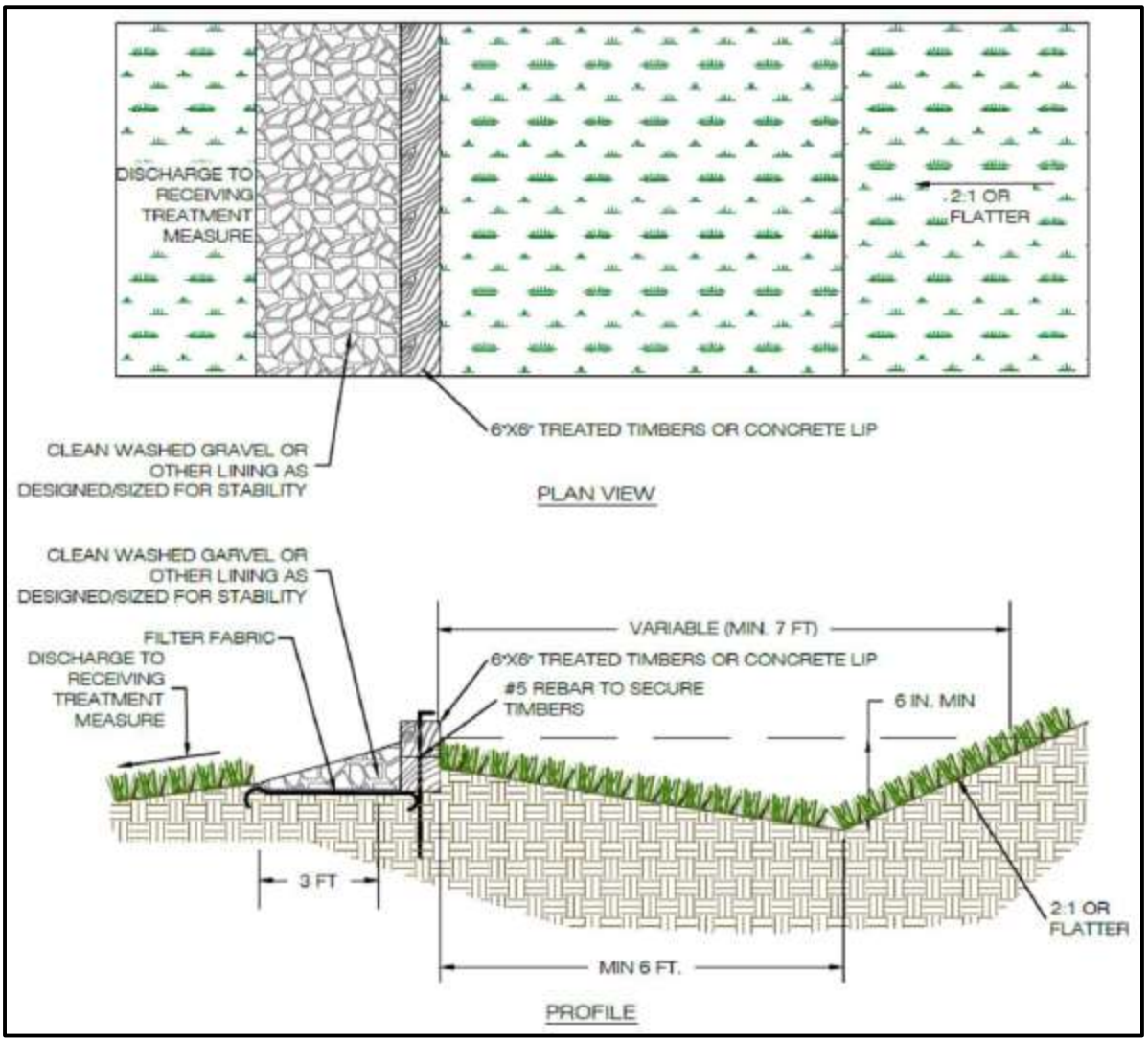


Figure C-90. Section - Level Spreader with Rigid Lip (Source: VADCR, 2011).

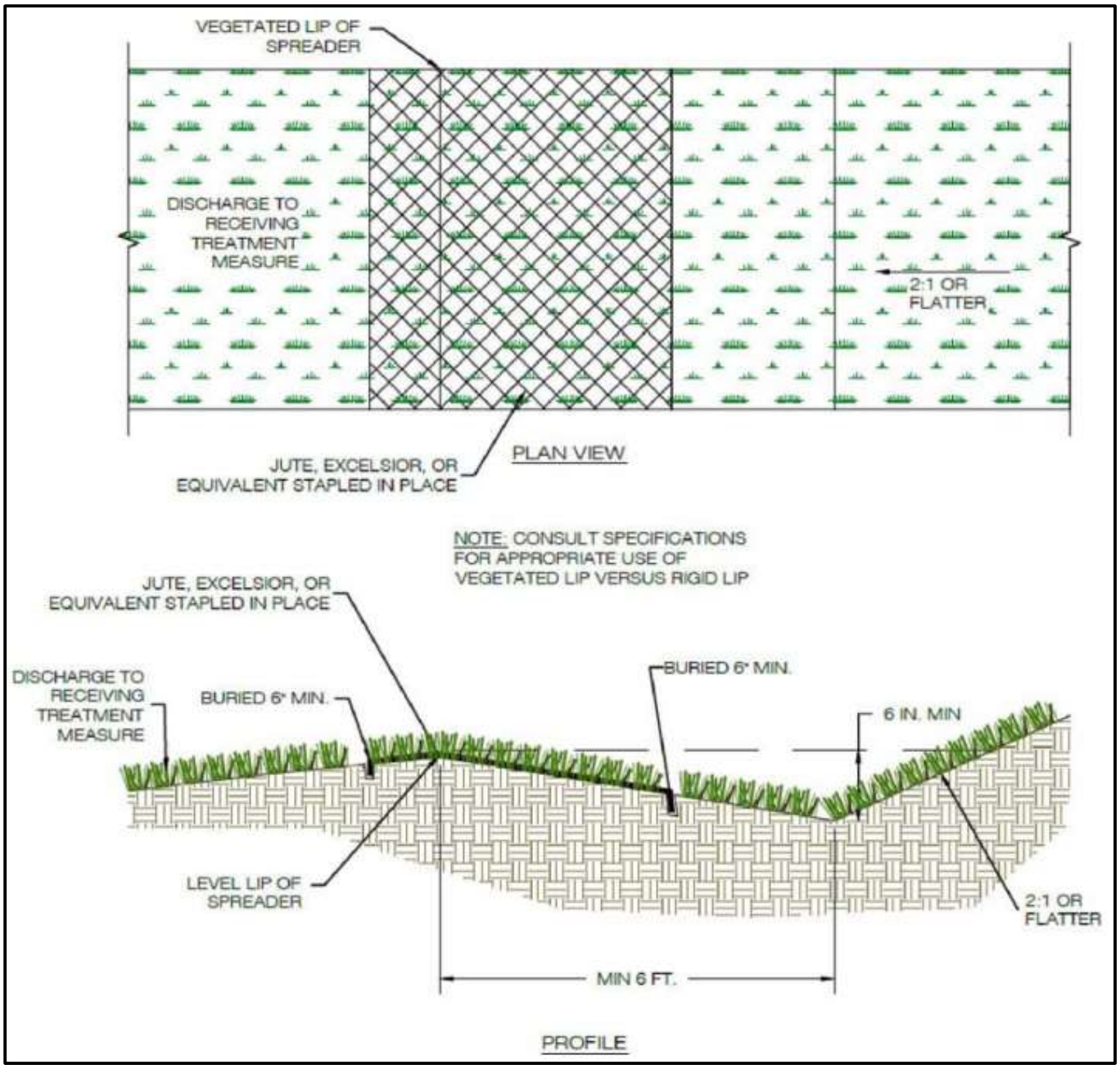


Figure C-91. Section - Alternative Level Spreader with Vegetated Lip (Source: VADCR, 2011).

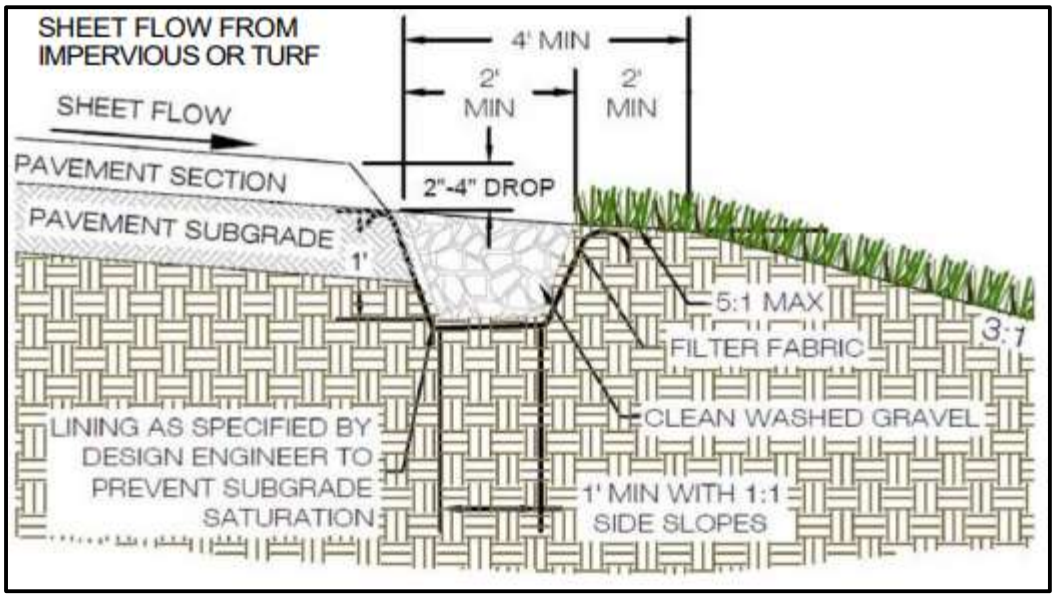


Figure C-92. Gravel Diaphragm – Sheet Flow Pre-treatment (Source: VADCR, 2011).

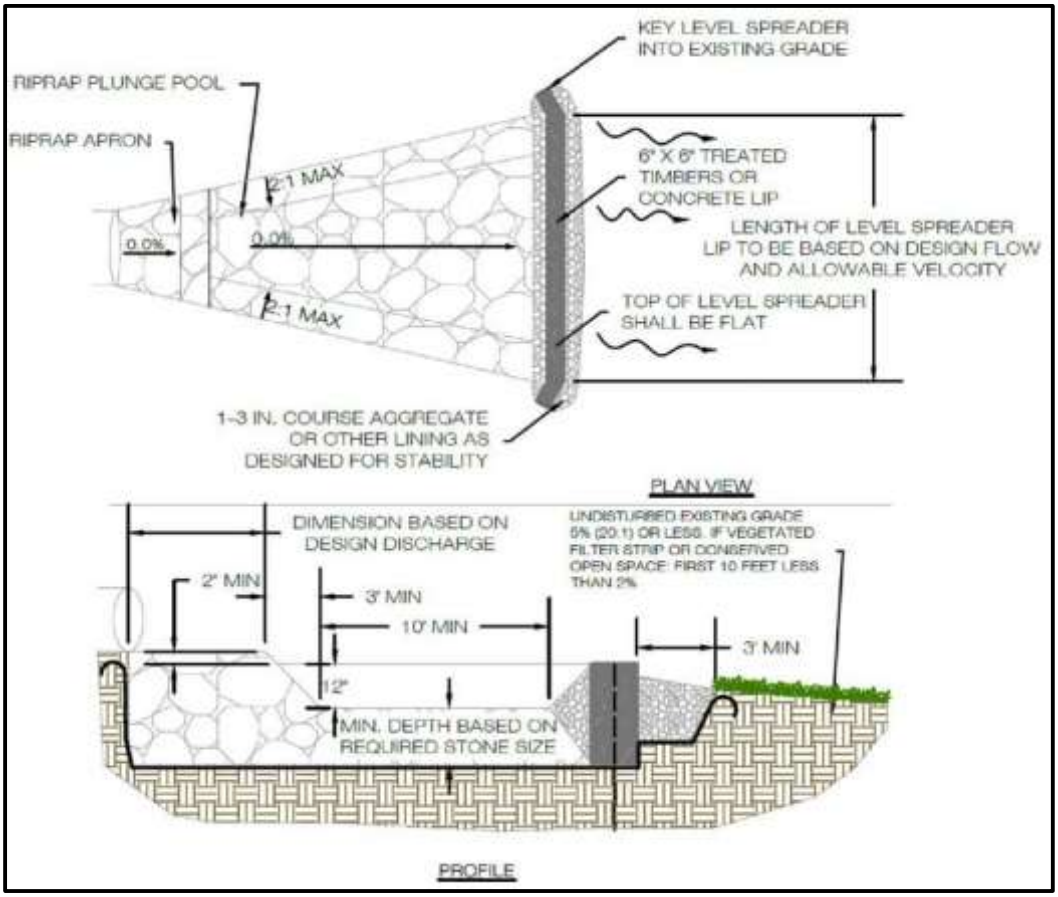


Figure C-93. Level Spreader: Pipe or Channel Flow to Filter Strip or Preserved Open Space (Source: VADCR, 2011).

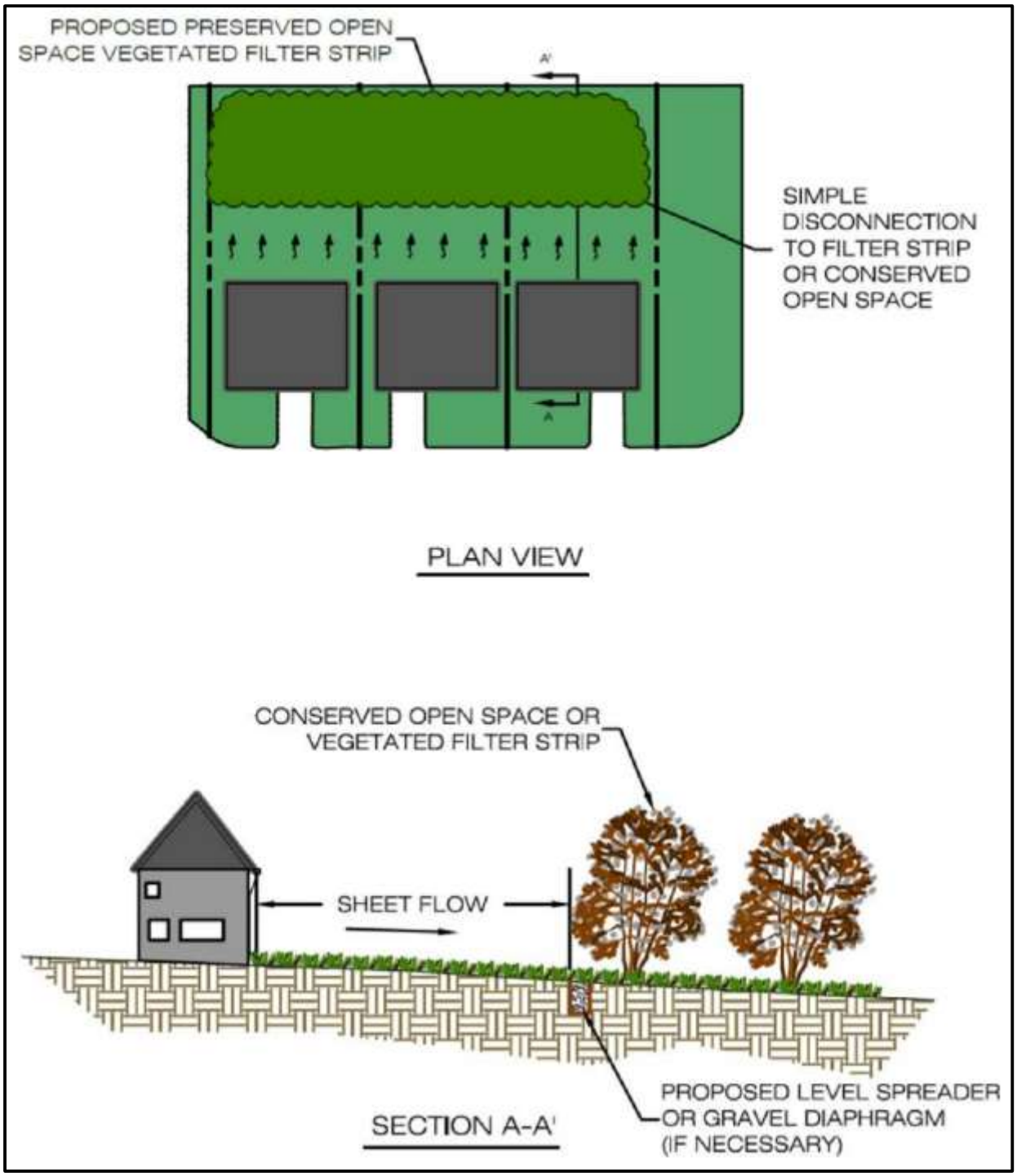


Figure C-94. Simple Disconnection to downstream Preserved Open Space or Vegetated Filter Strip (Source: VADCR, 2011).

4. Physical Feasibility and Design Applications

4.1 Conserved Open Space

Designers may apply a runoff reduction credit to any impervious area that is hydrologically connected and effectively treated by a protected Conserved Open Space that meets the following eligibility criteria:

- No major disturbance may occur within the conserved open space during or after construction. No clearing or grading shall occur except temporary disturbances associated with incidental utility construction of less than 10% total conserved open space area, restoration operations, or management of nuisance vegetation. The Conserved Open Space area shall not be stripped of topsoil. If required, some light grading may be performed at the boundary using tracked vehicles to prevent compaction.
- The limits of disturbance shall be clearly shown on all construction drawings and protected by appropriate signage, fencing, and erosion control measures.
- A long-term vegetation management plan must be prepared to maintain the Conserved Open Space in a natural vegetative condition. Generally, Conserved Open Space management plans do not allow any active management. However, a specific plan shall be developed to manage the unintended consequences of passive recreation, control invasive species, provide for tree and understory maintenance, etc.
- The Conserved Open Space must be protected by a perpetual easement or deed restriction that assigns the responsible party to ensure that no future development, disturbance, or clearing may occur within the area.
- The practice does not apply to jurisdictional wetlands that are sensitive to increased stormwater inflows.

4.2 Vegetated Filter Strips

Vegetated Filter Strips are best suited to treat runoff from small segments of impervious cover (usually less than 5,000 sq ft) adjacent to road shoulders, small parking lots and rooftops. Vegetated Filter Strips may also be used as pretreatment for another stormwater practice such as a dry swale, bioretention, or infiltration areas. If sufficient pervious area is available at the site, larger areas of impervious cover can be treated by vegetated filter strips, using an engineered level spreader to recreate sheet flow.

Conserved Open Space and Vegetated Filter Strips can be used in a variety of situations; however, there are several constraints to their use:

- ***Filter Slopes and Widths.*** Maximum slope for both Conserved Open Space and Vegetated Filter Strips is 6%, in order to maintain sheet flow through the practice. In addition, the overall contributing drainage area must likewise be relatively flat to ensure sheet flow draining into the filter. Where this is not possible, alternative measures, such as an engineered level spreader, can be used. Minimum widths (flow path) for Conserved Open Space and Vegetated Filter Strips are dependent on slope, as specified in Table C-55.
- ***Soils.*** Vegetated Filter Strips are appropriate for all soil types, except fill soils without amendment.
- ***Contributing Flow Path to Filter.*** Vegetated Filter Strips are used to treat very small drainage areas of a few acres or less. The limiting design factor is the length of flow directed to the filter. As a rule, flow tends to concentrate after traveling 75 ft for impervious surfaces, and 150 ft for pervious surfaces (Claytor, 1996). When flow concentrates, it moves too rapidly to be effectively treated by a Vegetated Filter Strip, unless an engineered level spreader is used. When the existing flow at a site is concentrated, a water quality swale shall be used instead of a Vegetated Filter Strip (Lantin and Barrett, 2005).
- ***Potential Areas of High Pollutant Loading.*** Vegetated Filter Strips shall not receive untreated runoff from these areas since the infiltrated runoff could cause groundwater contamination.
- ***Proximity of Underground Utilities.*** Underground pipes and conduits that cross the Vegetated Filter Strip are acceptable. Care shall be taken in backfilling of utilities to assure that the surface drainage pattern is not disturbed.

5. Design Criteria

5.1 Compost Soil Amendments

Compost soil amendment enhance the runoff reduction capability of a vegetated filter strip when they are located on hydrologic soil groups (HSG) B, C, and D, subject to the following design requirements:

- The compost amendments shall extend over the full length and width of the filter strip.
- The amended area will be raked to achieve the most level slope possible without using heavy construction equipment, and it will be stabilized rapidly with perennial grass and/or herbaceous species.
- If slopes exceed 3%, a protective biodegradable fabric or matting shall be installed to stabilize the site prior to runoff discharge.
- Compost amendments shall not be incorporated until the gravel diaphragm and/or engineered level spreader are installed (see Section 6.3).
- The City may waive the requirement for compost amendments on HSG-B soils in order to receive credit as a filter strip if (1) the designer can verify adequate soil type, texture, and infiltration characteristics exista filter strip, and (2) the area designated for the filter strip is not disturbed during construction.

5.2 Planting and Vegetation Management

Conserved Open Space. No grading or clearing of native vegetation is allowed within the Conserved Open Space other than for vegetation management in accordance with Section 5.1 of this WQ.

Reforested Conserved Open Space. At some sites, the Conserved Open Space may be in turf or meadow cover or overrun with invasive plants and vines. In these situations, a landscape architect shall prepare a reforestation plan for the Conserved Open Space with any credits and associated plans receiving approval by the City.

Vegetated Filter Strips. Vegetated Filter Strips shall be planted at such density as to achieve a 90% grass/herbaceous cover after the second growing season. Filter strips shall be seeded, not sodded, whenever possible. Seeding establishes deeper roots, and sod may have muck soil that is not conducive to infiltration (Wisconsin DNR, 2007). The filter strip vegetation may consist of turf grasses, meadow grasses, other herbaceous plants, shrubs, and trees, as long as the primary goal of at least 90% coverage with grasses and/or other herbaceous plants is achieved. Designers shall choose vegetation that stabilizes the soil and is salt tolerant. Vegetation at the toe of the filter, where temporary ponding may occur behind the permeable berm, shall be specified that is able to withstand both wet and dry periods. The planting areas can be divided into zones to account for differences in inundation and slope. Planting plans shall be prepared by a licensed Landscape Architect.

5.3 Diaphragms, Berms and Level Spreaders

Gravel Diaphragms: A diaphragm consisting of pea gravel at the top of the slope is required for both Conserved Open Space and Vegetated Filter Strips that receive sheet flow. The pea gravel diaphragm is created by excavating a 2-ft wide and 1-ft deep trench of constant elevation at the top of the filter strip. The diaphragm serves two purposes, acting as a pretreatment device to settle out sediment particles before they reach the practice, and as a level spreader, maintaining sheet flow as runoff flows over the Filter Strip. Refer to Figure C-92.

- The flow shall be directed over the impervious area and to the practice as sheet flow and then drop 2 to 4 inches onto the gravel diaphragm. The drop helps to prevent runoff from running laterally along the pavement edge, where grit and debris tend to build up (thus allowing by-pass of the Filter Strip).
- A layer of filter fabric shall be placed between the gravel and the underlying soil trench.
- If the contributing drainage area is steep (6% slope or greater), then larger stone (B-Stone) shall be used in the diaphragm.

Permeable Berm: Vegetated Filter Strips shall be designed with a permeable berm at the toe of the Filter Strip to create a shallow ponding area for runoff. Runoff then will gradually flow through outlet pipes in the berm or through a permeable lens or filter that contains a perforated pipe within the berm. During larger storms, runoff may overtop the berm (Cappiella

et al., 2006) and shall be designed to accommodate erosive velocities up to and including a 50-yr design storm. The permeable berm shall have the following properties:

- A wide and shallow trench, 6 to 12 inches deep, shall be excavated at the upstream toe of the berm, parallel with the contours.
- Media for the berm may consist of 40% excavated soil (if non-friable), 40% sand, and 20% pea gravel.
- The 6- to 12-inch-tall berm shall be located down gradient of the excavated depression and shall have gentle side slopes to promote easy mowing (Cappiella et al., 2006).
- Stone may be needed to armor the top of berm to handle extreme storm events (up to and including the 50-yr design storm).
- A permeable berm is not needed when vegetated filter strips are used as pretreatment to another stormwater practice treating the same runoff.

Engineered Level Spreaders. The design of engineered level spreaders shall conform to the following design criteria based on recommendations of Hathaway and Hunt (2006) in order to ensure non-erosive sheet flow into the vegetated area. Figure C-89 represents a configuration that includes a bypass structure that diverts the design storm to the level spreader and bypasses the larger storm events around the Conserved Open Space or Vegetated Filter Strip through an improved channel.

An alternative approach involves pipe or channels discharging at the landward edge of a floodplain. The entire flow is directed through a stilling basin energy dissipater and then a level spreader such that the entire design storm for the conveyance system (typically a 10-year frequency storm) is discharged as sheet flow through the floodplain.

Key design elements of the engineered level spreader, as provided in Figures C-88 and C-89, include the following:

- High Flow Bypass provides safe passage for larger design storms through the filter strip. The bypass channel design shall accommodate all peak flows greater than the water quality design flow.
- The forebay shall have a maximum depth of 3 ft and gradually transition to a depth of 1 ft at the level spreader lip (Figure C-88). The forebay is sized such that the surface area is 0.2% of the contributing impervious area. (A forebay is not necessary if the concentrated flow is from the outlet of an extended detention basin or similar practice).
    - The length of the level spreader shall be determined by the type of filter area and the design flow:
    - 13 ft of level spreader length per every 1 cubic ft per second (cfs) of inflow for discharges to a Vegetated Filter Strip or Conserved Open Space consisting of native grasses or thick ground cover.
    - 40 ft of level spreader length per every 1 cfs of inflow when the spreader discharges to a Conserved Open Space consisting of forested or reforested area (Hathaway and Hunt, 2006).
    - Where the Conserved Open Space is a mix of native grasses / thick ground cover and forest (or re-forested), establish the level spreader length based on a weighted average of the lengths required for each vegetation type.
    - The minimum level spreader length is 13 ft, and the maximum is 130 ft.
    - For the purposes of determining the Level Spreader length, the peak discharge shall be determined for the entire contributing area, using the Rational Equation with an intensity of 1- inch/hour, or a more detailed method in conformance with relevant portions of the Drainage Criteria Manual.
- The level spreader lip shall be concrete, wood, or prefabricated metal, with a well-anchored footer, or other accepted rigid, non-erodible material.
- The ends of the level spreader section shall be tied back into the slope to avoid scouring around the ends of the level spreader; otherwise, short-circuiting of the facility could occur.
- The width of the level spreader channel on the up-stream side of the level lip shall be three times the diameter of the inflow pipe, and the depth shall be 9 inches or one-half the culvert diameter, whichever is greater.

- The level spreader shall be placed 3 to 6 inches above the downstream natural grade elevation to avoid turf buildup. To prevent grade drops that re-concentrate the flows, a 3-ft long section of coarse aggregate, underlain by filter fabric, shall be installed just below the spreader to transition from the level spreader to natural grade.

Vegetated receiving areas down-gradient from the level spreader must be able to withstand the force of the flow coming over the lip of the device. It may be necessary to stabilize this area with temporary or permanent materials in accordance with the calculated velocity (on-line system peak, or diverted off-line peak) and material performance data, along with seeding and stabilization in conformance with the requirements of the Fayetteville Drainage Criteria manual.

5.4 Filter Design Material Specifications

Table C-56 describes materials specifications for the primary treatment within filter strips.

Table C-56. Vegetated Filter Strip Material Specifications.

Material	Specification	Quantity
Gravel Diaphragm	Pea Gravel (#8 or ASTM equivalent); where steeper (6% +) use B-stone or ASTM equivalent (1-inch maximum).	Diaphragm shall be 2 ft wide, 1 ft deep, and 2 to 4 inches below the edge of pavement.
Permeable Berm	40% excavated soil (non-friable), 40% sand, and 20% pea gravel to serve as the media for the berm.	
Geotextile	Needled, non-woven, polypropylene geotextile meeting the following specifications: Grab Tensile Strength (ASTM D4-632): > 120 lbs. Mullen Burst Strength (ASTM D-3786): > 225 lbs./sq. in. Flow Rate (ASTM D-4491): > 125 gpm/sq. ft. Apparent Opening Size (ASTM D-4751): US #70 or #80 sieve	
Engineered Level spreader	Level spreader lip shall be concrete, metal, timber, or other rigid material, Reinforced channel on upstream of lip. See Hathaway and Hunt (2006)	
Erosion Control Fabric/ Matting	Where flow velocities dictate, use woven biodegradable erosion control fabric or mats that are durable enough to last at least 2 growing seasons.	
Topsoil	If existing topsoil is inadequate to support dense turf growth, imported topsoil (loamy sand or sandy loam texture), with 5% or lower clay content, corrected pH at 6 to 7, a soluble salt content not exceeding 500 ppm, and an organic matter content of at least 2% shall be used. Topsoil shall be uniformly distributed and lightly compacted to a minimum depth of 6 to 8 inches.	
Compost	Compost shall be derived from plant material and consistent with other requirements	

6. Construction

6.1 Construction Sequence for Conserved Open Space Areas

The Conserved Open Space must be fully protected during the construction stage of development and kept outside the limits of disturbance on the Erosion and Sediment Control Plan.

- No clearing, grading or heavy equipment access is allowed except temporary disturbances associated with incidental utility construction, restoration operations or management of nuisance vegetation.
- The perimeter of the Conserved Open Space shall be protected by a silt fence, chain link fence, orange safety fence, signage, and other measures as needed to meet stormwater pollution prevention sediment discharge requirements.
- The limits of disturbance and extent of Conserved Areas shall be clearly shown on site development plans, Grading Permit applications and/or concept plans and identified and clearly marked in the field.

- Construction of the gravel diaphragm or engineered level spreader shall not commence until the contributing drainage area has been stabilized and perimeter erosion and sedimentation controls have been removed and cleaned.
- Some light grading may be needed at the area boundary; this shall be done with tracked vehicles to minimize compaction.
- Stormwater shall not be diverted into the area until the gravel diaphragm and/or level spreader are installed and stabilized.

6.2 Construction Sequence for Vegetated Filter Strips

Vegetated Filter Strips can be within the limits of disturbance during construction. Soil amendments may be required in accordance with previous sections. The following procedures shall be followed during construction:

- Before site work begins, Vegetated Filter Strip boundaries shall be clearly marked.
- Only vehicular traffic used for Filter Strip construction shall be allowed within 10 ft of the Filter Strip boundary (City of Portland, 2004).
- Existing topsoil shall not be stripped during grading; if it is, it shall be stockpiled for later use.
- Construction runoff shall be directed away from the proposed Filter Strip site, using perimeter silt fence, or, preferably, a diversion dike.
- Construction of the gravel diaphragm or engineered level spreader shall not commence until the contributing drainage area has been stabilized and perimeter erosion and sedimentation controls have been removed and cleaned out.
- Vegetated Filter Strips may require light grading to achieve desired elevations and slopes. This shall be done with tracked vehicles to prevent compaction. Topsoil and or compost amendments shall be incorporated evenly across the filter strip area, stabilized with seed, and protected by biodegradable erosion control matting or blankets.
- Stormwater shall not be diverted into the Filter Strip until the turf cover is dense and well established.

6.3 Construction Inspection

Construction inspection is critical to ensure construction is consistent with design, to ensure the gravel diaphragm or engineered level spreader is constructed level and constructed to the correct design elevation. As-built certification is required to ensure compliance with design standards. Inspectors shall evaluate the performance of the Filter Strip after the first big storm to look for evidence of gullies, short-circuiting, undercutting or sparse vegetative cover. Repairs shall be made as needed.

7. Maintenance

7.1 Maintenance Requirements

A Long-Term Maintenance Plan (LTMP) for the Vegetated Filter Strip WQ BMP shall be prepared by the design engineer to address inspection and maintenance. If the filter area is a natural Conserved Open Space, it must be protected by a perpetual easement or deed restriction that assigns a responsible party to ensure that no future development, disturbance or clearing may occur within the area, except as stipulated in the long-term vegetation management plan.

7.2 Maintenance Inspections

Annual inspections are used to trigger maintenance operations such as sediment removal, re-vegetation, and level spreader repair. Ideally, inspections shall be conducted in the non-growing season when it easier to see the flow path. Inspectors shall check that:

- Flows through the Filter Strip do not short-circuit the overflow control section.
- Debris and sediment do not build up at the top of the Filter Strip.
- The gravel diaphragm performance is not compromised.
- Erosion does not occur within the Filter Strip.

- Trash and sediments are adequately removed from Level Spreader forebays and flow splitters.
- Vegetative density exceeds a 90% cover in the boundary zone or grass filter.

7.3 Ongoing Maintenance

Once established, Vegetated Filter Strips have minimal maintenance needs outside of the spring cleanup, regular mowing, repair of check dams and other measures to maintain the hydraulic efficiency of the strip and a dense, healthy grass cover. Vegetated Filter Strips that consist of grass/turf cover shall be mowed at least twice a year to prevent woody growth.

8. As-Built Requirements

After the filter strip has been constructed, the developer must provide the city an as-built certification of the filter strip prepared by a registered Professional Engineer. The as-built certification verifies that the BMP was installed as designed and approved or documents exceptions and verifies functionality. The following components must be addressed in the as-built certification:

1. Ensure level spreader is properly installed to create sheet flow.
2. Ensure vegetated filter strip or open space that receives sheet flow has minimal slope.
3. Ensure appropriate drainage.
4. Ensure the proper vegetation has been established or protected – this may be established by a qualified Landscape Architect.

9. References

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WQ-18 Green Roof

1. Description

Vegetated roofs (also known as green roofs, living roofs or Eco roofs) are alternative roof surfaces that typically consist of waterproofing and drainage materials and an engineered growing media that is designed to support plant growth. Vegetated roofs capture and temporarily store stormwater runoff in the growing media before it is conveyed into the storm drain system. A portion of the captured stormwater evaporates or is taken up by plants, which helps reduce runoff volumes, peak runoff rates and pollutant loads on development sites.

There are two different types of vegetated roof systems: intensive vegetated roofs and extensive vegetated roofs. Intensive systems have a deeper growing media layer that ranges from 6 inches to 4 feet thick, which is planted with a wider variety of plants, including trees. By contrast, extensive systems typically have much shallower growing media (under 6 inches), which is planted with carefully selected drought tolerant vegetation. Extensive vegetated roofs are much lighter and less expensive than intensive vegetated roofs and are recommended for use on most development and redevelopment sites.

NOTE: This specification is intended for situations where the primary design objective of the vegetated roof is stormwater management and, unless specified otherwise, addresses extensive roof systems. Hence tree planting recommendations are not provided in this document.

Designers may wish to pursue other design objectives for vegetated roofs, such as energy efficiency, green building or LEED points, architectural considerations, visual amenities, and landscaping features, which are often maximized with intensive vegetated roof systems. However, these design objectives are beyond the scope of this specification.

Vegetated roofs typically contain a layered system of roofing, which is designed to support plant growth and retain water for plant uptake while preventing ponding on the roof surface. The roofs are designed so that water drains vertically through the media and then horizontally along a waterproofing layer towards the outlet. Extensive vegetated roofs are designed to have minimal maintenance requirements. Plant species are selected so that the roof does not need supplemental irrigation or fertilization after vegetation is initially established. Tray systems are also available with removable dividers allowing the media to meld together creating a seamless appearance but with less difficulty in construction.



Figure C-95. Photos of Vegetated Roof Cross-Sections (Source: B. Hunt, NCSU).

2. Typical Details

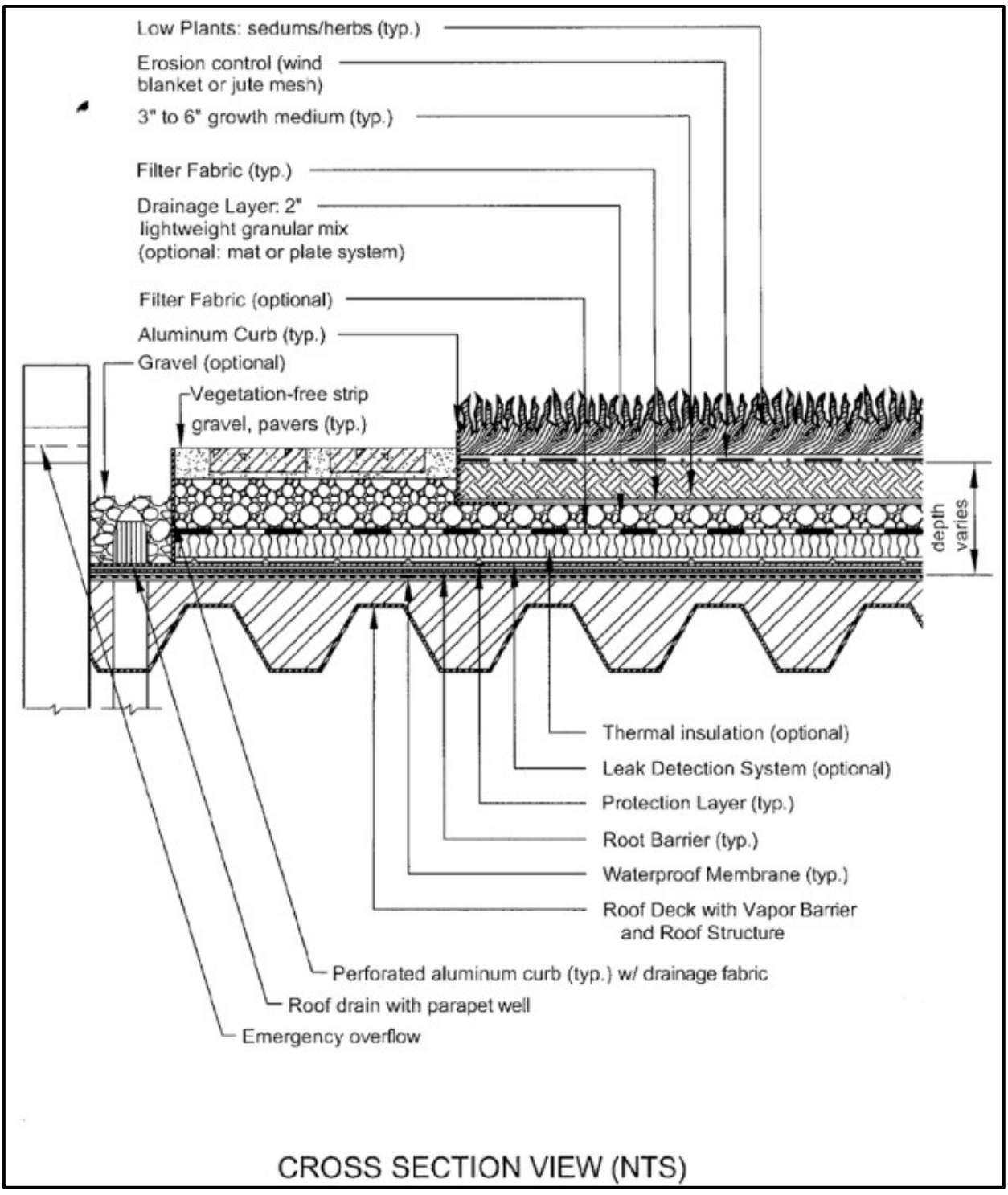


Figure C-96. Typical Section – Extensive Vegetated Roof (Source: Northern VA Regional Commission).

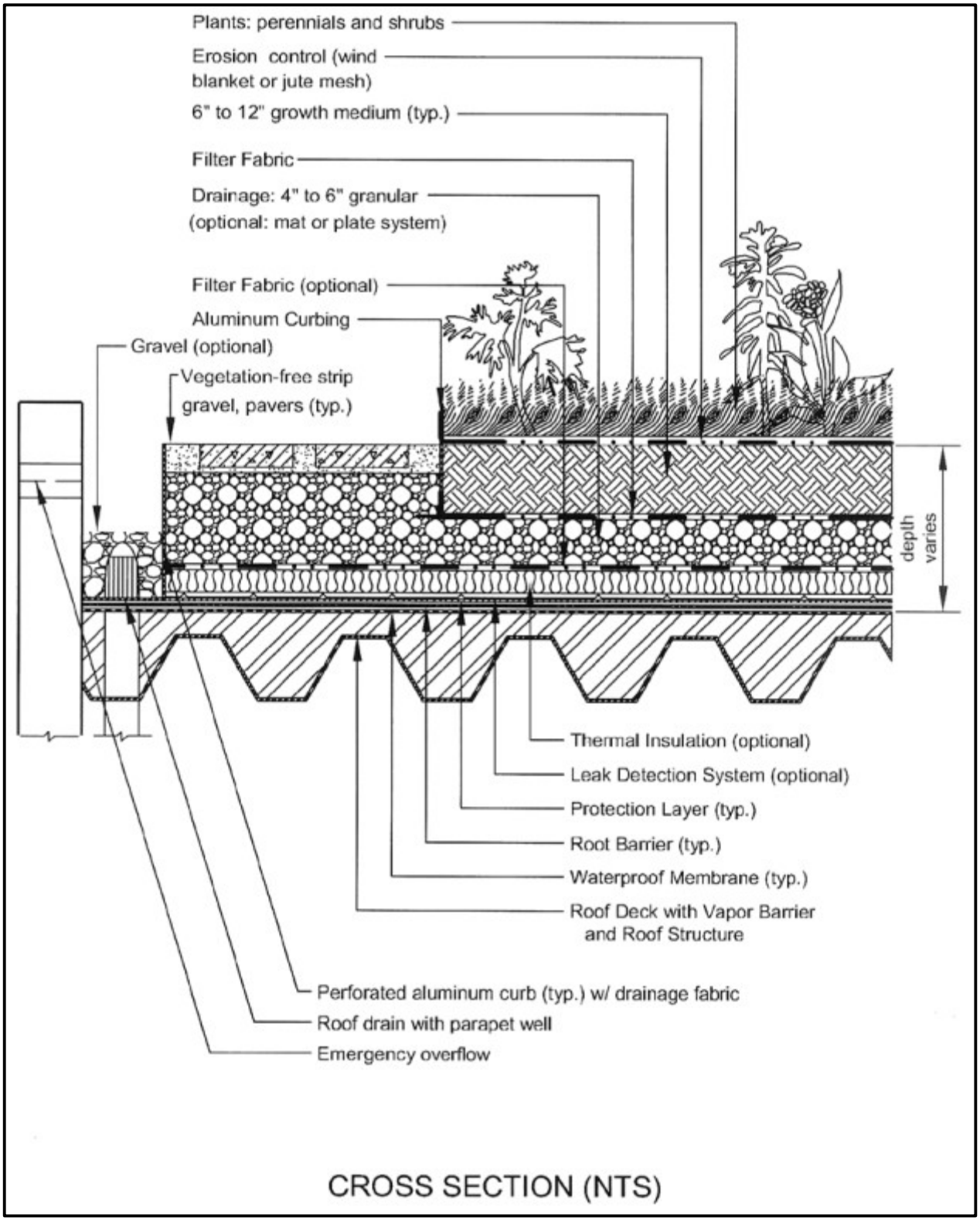


Figure C-97. Typical Section – Intensive Vegetated Roof (Source: Northern VA Regional Commission).

3. Physical Feasibility and Design Applications

3.1 Typical applications

Vegetated roofs are ideal for use on commercial, institutional, municipal, and multi-family residential buildings. They are particularly well suited for use on ultra-urban development and redevelopment sites. Vegetated roofs can be used on a variety of rooftops, including the following:

- Non-residential buildings (e.g., commercial, industrial, institutional and transportation uses)
- Multi-family residential buildings (e.g., condominiums or apartments)
- Mixed-use buildings

3.2 Common Site Constraints

Structural Capacity of the Roof. When designing a vegetated roof, designers must not only consider the stormwater storage capacity of the vegetated roof, but also its structural capacity to support the weight of the additional water. A conventional rooftop typically must be designed to support an additional 15 to 30 pounds per square foot (psf) for an extensive vegetated roof. As a result, a licensed structural engineer shall be involved with all vegetated roof designs to ensure that the building has enough structural capacity to support a vegetated roof.

Roof Pitch. Water Quality Volume (WQv) is maximized on relatively flat roofs (a pitch of 1 to 2%). Some pitch is needed to promote positive drainage and prevent ponding and/or saturation of the growing media. Vegetated roofs can be installed on rooftops with slopes up to 25% if baffles, grids, or strips are used to prevent slippage of the media. The effective Water Quality Volume (WQv), however, diminishes on rooftops with steep pitches (Van Woert et al., 2005).

Roof Access. Adequate access to the roof must be available to deliver construction materials and perform routine maintenance. Roof access can be achieved either by an interior stairway through a penthouse or by an alternating tread device with a roof hatch or trap door not less than 16 square feet in area and with a minimum dimension of 24 inches (NVRC, 2007). Designers should also consider how they will get construction materials up to the roof (e.g., by elevator or crane), and how and where construction materials will be stockpiled in the confined space.

Non-Vegetated Areas. Roof access paths, mechanical equipment, photovoltaic panels, and skylights are counted as part of the green roof for calculation purposes. These areas shall not exceed 20% of the roof area counted as green roof.

Roof Type. Vegetated roofs can be applied to most roof surfaces, although concrete roof decks are preferred. Certain roof materials, such as exposed treated wood and uncoated galvanized metal, may not be appropriate for vegetated rooftops due to pollutant leaching through the media (Clark et al.., 2008).

Retrofitting Green Roofs. Key feasibility factors to consider when evaluating a retrofit include the area, age and accessibility of the existing roof, and the capability of the building's owners to maintain it. Options for green roof retrofits are described in Profile Sheet RR-3 of Schueler et al. (2007). The structural capacity of the existing rooftop can be a major constraint to a green roof retrofit.

Building Codes. The vegetated roof design shall comply with all applicable building codes for Little Rock with respect to roof drains and emergency overflow devices. If the green roof is designed to be accessible, the access must not only be convenient for installation and maintenance purposes but also must adhere to applicable building codes and other regulations for access and safety.

Construction Cost. When viewed strictly as stormwater treatment systems, vegetated roofs can cost between $12 and $25 per square foot (Moran et al., 2005, Schueler et al. 2007). These cost analyses, however, do not include life cycle cost savings relating to increased energy efficiency, higher rents due to green building scores and increased roof longevity. These benefits over the life cycle of a vegetated roof may make it a more attractive investment.

Risks of Leaky Roofs. Although well designed and installed green roofs purportedly have less problems with roof leaks than traditional roofs, there is a perception among property managers, insurers, and product fabricators that this emerging technology could have a greater risk of problems. For an excellent discussion on how to properly manage risk in vegetated roof installations, see Chapter 9 in Weiler and Scholz-Barth (2009).

4. Design Criteria

4.1 Overall Sizing

Vegetated roof areas shall be sized to capture a portion of the Water Quality Volume (WQv). The required size of a vegetated roof will depend on several factors, including the porosity and hydraulic conductivity of the growing media and the underlying drainage materials. Site designers and planners shall consult with vegetated roof manufacturers and material suppliers for specific sizing guidelines. As a general sizing rule, the following equation can be used to determine the water quality treatment storage volume retained by a vegetated roof:

$$\boldsymbol{WQv = (RA * D * P)/12} \qquad \textbf{Eq. C-24}$$

Where: WQv = storage volume (cu. ft.)

RA = vegetated roof area (sq. ft.)

D = media depth (in.)

P = media porosity (usually 0.3, but consult manufacturer specifications)

The resulting WQv can then be compared to the required WQv for the entire rooftop area (including all non-vegetated areas) to determine if it meets or exceeds the required WQv.

4.2 Structural Capacity of the Roof

Vegetated roofs must be designed and constructed for the additional weight of the fully saturated growing medium and mature plants, in terms of the physical capacity of the roof to bear structural loads. The designer shall consult with a licensed structural engineer or architect to ensure that the building will be able to support the additional live and dead structural load and determine the maximum depth of the vegetated roof system and any needed structural reinforcement.

In most cases, fully saturated extensive vegetated roofs have a maximum load of about 30 lbs./sq. ft., which is fairly similar to traditional new rooftops (12 to 15 lbs./sq. ft.) that have a waterproofing layer anchored with stone ballast. For an excellent discussion of vegetated roof structural design issues, consult Chapter 9 in Weiler and Scholz-Barth (2009) and ASTM E-2397, Standard Practice for Determination of Dead Loads and Live Loads Associated with Green (Vegetated) Roof Systems.

4.3 Functional Elements of a Vegetated Roof System

A vegetated roof is composed of up to eight different systems or layers, from bottom to top, that are combined to protect the roof and maintain a vigorous cover. Designers can employ a wide range of materials for each layer, which can differ in cost, performance, and structural load. The entire system as a whole must be assessed to meet design requirements. Some manufacturers offer proprietary vegetated roofing systems, whereas in other cases, the designer or architect must assemble their own system, in which case they are advised to consult Weiler and Scholz-Barth (2009), Snodgrass and Snodgrass (2006) and Dunnett and Kingsbury (2004).

1. **Deck Layer.** The roof deck layer is the foundation of a vegetated roof. It and may be composed of concrete, wood, metal, plastic, gypsum, or a composite material. The type of deck material determines the strength, load bearing capacity, longevity, and potential need for insulation in the vegetated roof system. In general, concrete decks are preferred for vegetated

roofs, although other materials can be used as long as the appropriate system components are matched to them.

2. **Waterproofing Layer.** All vegetated roof systems must include an effective and reliable waterproofing layer to prevent water damage through the deck layer. A wide range of waterproofing materials can be used, including built up roofs, modified bitumen, single-ply, and liquid-applied methods (see Weiler and Scholz-Barth, 2009 and Snodgrass and Snodgrass, 2006). The waterproofing layer must be 100% waterproof and have an expected life span as long as any other element of the vegetated roof system.
3. **Insulation Layer.** Many vegetated rooftops contain an insulation layer, usually located above, but sometimes below, the waterproofing layer. The insulation increases the energy efficiency of the building and/or protects the roof deck (particularly for metal roofs). According to Snodgrass and Snodgrass (2006), the trend is to install insulation on the outside of the building, in part to avoid mildew problems.
4. **Root Barrier (Optional).** The next layer of a vegetated roof system is an optional root barrier that protects the waterproofing membrane from root penetration. A wide range of root barrier options are described in Weiler and Scholz-Barth (2009). Chemical root barriers or physical root barriers that have been impregnated with pesticides, metals or other chemicals that could leach into stormwater runoff shall be avoided.
5. **Drainage Layer and Drainage System.** A drainage layer is then placed between the optional root barrier and the growing media to quickly remove excess water from the vegetation root zone. The drainage layer shall consist of synthetic or inorganic materials (e.g., gravel, recycled polyethylene, etc.) that are capable of retaining water and providing efficient drainage. A wide range of prefabricated water cups or plastic modules can be used, as well as a traditional system of protected roof drains, conductors and roof leader. The required depth of the drainage layer is governed by both the required stormwater storage capacity and the structural capacity of the rooftop. ASTM E2396 and E2398 can be used to evaluate alternative material specifications.
6. **Root-Permeable Filter Fabric.** A semi-permeable polypropylene filter fabric is normally placed between the drainage layer and the growing media to prevent the media from migrating into the drainage layer and clogging it.
7. **Growing Media.** The next layer in an extensive vegetated roof is the growing media, which is typically 4-to-6 inches deep for extensive roofs and 6 inches or more for intensive roofs. The depth and composition of the media is described in Section 6.5.
8. **Plant Cover.** The top layer of a vegetated roof consists of slow-growing, shallow-rooted, perennial, succulent plants that can withstand harsh conditions at the roof surface. Guidance on selecting the appropriate vegetated roof plants for hardiness zones in Little Rock can be found in Snodgrass and Snodgrass (2006). A mix of base ground covers (usually Sedum species) and accent plants can be used to enhance the visual amenity value of a green roof.

4.4 Pretreatment

Pretreatment is not needed for green roofs.

4.5 Material Specifications

Standards specifications for North American vegetated roofs continue to evolve, and no universal material specifications exist that cover the wide range of roof types and system components currently available. The American Society for Testing and Materials (ASTM) has recently issued several overarching vegetated roof standards, which are described and referenced in Table C-59 below.

Designers and reviewers shall also fully understand manufacturer specifications for each system component listed in Section 6.3, particularly if they choose to install proprietary “complete” vegetated roof systems or modules.

Table C-59. Green Roof Design Guidance

Material	Specification
Roof	Structural Capacity shall conform to ASTM E-2397-05, Practice for Determination of Live Loads and Dead Loads Associated with Green (Vegetated) Roof Systems. In addition, use standard test methods ASTM E2398-05 for Water Capture and Media Retention of Geocomposite Drain Layers for Green (Vegetated) Roof Systems, and ASTME 2399-05 for Maximum Media Density for Dead Load Analysis.
Waterproof Membrane	See Chapter 6 of Weiler and Scholz-Barth (2009) for waterproofing options that are designed to convey water horizontally across the roof surface to drains or gutter. This layer may sometimes act as a root barrier.
Root Barrier (Optional)	Impermeable liner that impedes root penetration of the membrane.
Drainage Layer	1 to 2 in. layer of clean, washed granular material, such as ASTM D 448 size No. 8 stone. Roof drains and emergency overflow shall be designed in accordance with Fayetteville Building Safety Codes.
Filter Fabric	Needled, non-woven, polypropylene geotextile. Density (ASTM D3776) > 16 oz./sq. yd., or approved equivalent. Puncture resistance (ASTM D4833) > 220 lbs., or approved equivalent.
Growth Media	85% lightweight inorganic materials and 15% organic matter (e.g., well-aged compost). Media shall have a maximum water retention capacity of around 30%. Media shall provide sufficient nutrients and water holding capacity to support the proposed plant materials. Determine acceptable saturated water permeability using ASTM E2396-05.
Plant Materials	Low plants such as sedum, herbaceous plants, and perennial grasses that are shallow-rooted, self-sustaining, and tolerant of direct sunlight, drought, wind, and frost. See ASTM E2400-06, Guide for Selection, Installation and Maintenance of Plants for Green (Vegetated) Roof Systems.

4.6 Filter Media Composition

The recommended growing media for extensive vegetated roofs is composed of approximately 80% to 90% lightweight inorganic materials, such as expanded slates, shales or clays, pumice, scoria, or other similar materials. The remaining media shall contain no more than 15% organic matter, normally well-aged compost. The percentage of organic matter shall be limited, since it can leach nutrients into the runoff from the roof and clog the permeable filter fabric. The growing media shall have a maximum water retention capacity of around 30%. It is advisable to mix the media in a batch facility prior to delivery to the roof. More information on growing media can be found in Weiler and Scholz-Barth (2009) and Snodgrass and Snodgrass (2006).

The composition of growing media for intensive vegetated roofs may be different, and it is often much greater in depth (e.g., 6 inches to 4 feet). If trees are included in the vegetated roof planting plan, the growing media must provide enough volume for the root structure of mature trees.

4.7 Conveyance and Overflow

The drainage layer below the growth media shall be designed to convey the 10-year storm without backing water up to into the growing media. The drainage layer shall convey flow to an outlet or overflow system such as a traditional rooftop drainage system with inlets set slightly above the elevation of the vegetated roof surface. Roof drains immediately adjacent to the growing media shall be boxed and protected by flashing extending at least 3 inches above the growing media to prevent clogging.

4.8 Vegetation and Surface Cover

A planting plan must be prepared for a vegetated roof by a licensed Landscape Architect, botanist or other professional experienced with vegetated roofs, and it must be reviewed and approved by the City

of Little Rock. Plant selection for vegetated rooftops is an integral design consideration, which is governed by local climate and design objectives. The primary ground cover for most vegetated roof installations is a hardy, low-growing succulent, such as Sedum, Delosperma, Talinum, Semperivum or Hieracium that is matched to the local climate conditions and can tolerate the difficult growing conditions found on building rooftops (Snodgrass and Snodgrass, 2006). Little Rock lies in the USDA Plant Hardiness Zone 8a (AHS, 2003). A detailed list of options for local conditions is provided in the Green Roof Plant List, Table C-60.

- Plant choices can be much more diverse for deeper intensive vegetated roof systems. Herbs, forbs, grasses, shrubs and even trees can be used, but designers should understand they have higher watering, weeding and landscape maintenance requirements.
- The species and layout of the planting plan shall be based on considerations including height, exposure to wind, snow loading, heat stress, orientation to the sun, and shading by surrounding buildings. In addition, plants should be selected that are fire resistant and able to withstand heat, cold and high winds.
- Designers shall also designate species suited to the expected rooting depth of the growing media, which can also provide enough lateral growth to stabilize the growing media surface. The planting plan should usually include several accent plants to provide diversity and seasonal color. For a comprehensive resource on vegetated roof plant selection, consult Snodgrass and Snodgrass (2006).
- Given the limited number of vegetated roof plant nurseries in the region, designers should order plants 6 to 12 months prior to the expected planting date. It is also advisable to have plant materials contract-grown.
- When appropriate species are selected, most vegetated roofs will not require supplemental irrigation, except during the first year that the vegetated roof is being established or during periods of drought. Irrigation shall thus be provided as needed for full establishment and during drought periods. The planting window extends from the spring to early fall, although it is important to allow plants to root thoroughly before the first killing frost.
- Plants can be established using cuttings, plugs, mats, and, more rarely, seeding or containers. Several vendors also sell mats, rolls, or proprietary vegetated roof planting modules. For the pros and cons of each method, see Snodgrass and Snodgrass (2006).
- The goal for vegetated roof systems designed for stormwater management is to establish a full and vigorous cover of low-maintenance vegetation that is self-sustaining and requires minimal mowing, trimming, and weeding.
- The vegetated roof design shall include non-vegetated walkways (e.g., permeable paver blocks) to allow for easy access to the roof for weeding and making spot repairs.

Table C-60. Green Roof Plant List.

Latin Name	Common Name	Size	Spacing	Moisture	Color	Height
Flowering Perennials						
Asclepias tuberosa	Butterfly milkweed	Plugs – 1 gal.	1 plant/ 18 in. o.c.	Dry-moist	Orange	2’
Echinacea pallida	Pale purple coneflower	Plugs – 1 gal.	1 plant/ 18 in. o.c.	dry	Purple	2-3’
Echinacea purpurea	Purple coneflower	Plugs – 1 gal.	1 plant/ 18 in. o.c.	Moist-dry	Purple	3-4’
Liatris spicata	Dense blazingstar	Plugs – 1 gal.	1 plant/24 in. o.c.	Moist-dry	Purple	1.5’
Monarda didyma	Bee balm	Plugs – 1 gal.	1 plant/24 in. o.c.	Wet-moist	Red	3’

Monarda fistulosa	Wild bergamot	Plugs – 1 gal.	1 plant/ 18 in. o.c.	Moist-dry	Purple	1-3’
Oenethera speciosa	Evening primrose	Plugs – 1 gal.	1 plant/ 18 in. o.c.	Moist-dry	Pink	1-2’
Penstemon digitalis	Smooth white beardtongue	Plugs – 1 gal.	1 plant/24 in. o.c.	Wet	White	2-3’
Rudbeckia hirta	Black-eyed Susan	Plugs – 1 gal.	1 plant/ 18 in. o.c.	Moist-dry	Yellow	3’
Grasses/Sedges						
Equisetum hyemal	Horsetail	Plugs – 1 gal.	1 plant/ 18 in. o.c.	Wet	Green	3’
Carex sp.	Sedge	1 gal.	1 plant/24 in. o.c.	Moist	Green	2-3’
Panicum virgatum	Switchgrass	1-3 gal.	1 plant/48 in. o.c.	Moist-dry	Yellow	5-7’
Schizachyrium scoparium	Little Blue Stem	1 gal.	1 plant/24 in. o.c.	Moist-dry	Yellow	3’
Sorghastrum nutans	Indian grass	1 gal.	1 plant/24 in. o.c.	Moist-dry	Green	2-4’
Native Shrubs						
Callicarpa americana	American Beautyberry	NA	NA	Dry-moist	Lilac	4-6’
Hypericum prolificum	Shrubby St. John’s Wort	NA	NA	Dry-moist	Yellow	3’
Rhododendron sp.	Rhododendron/azalea	NA	NA	Moist-dry	Various	4-6’

5. Construction

5.1 Construction Sequence

Given the diversity of extensive vegetated roof designs, there is no typical step-by-step construction sequence for proper installation. The following general construction considerations are noted:

- Construct the roof deck with the appropriate slope and material.
- Install the waterproofing method, according to manufacturer’s specifications.
- Conduct a flood test to ensure the system is watertight by placing at least 2 inches of water over the membrane for 48 hours to confirm the integrity of the waterproofing system.
- Add additional system components (e.g., insulation, optional root barrier, drainage layer and interior drainage system, and filter fabric), taking care not to damage the waterproofing. Drain collars and protective flashing shall be installed to ensure free flow of excess stormwater.
- The growing media shall be mixed prior to delivery to the site. Media should be spread evenly over the filter fabric surface. The growing media shall be covered until planting to prevent weeds from growing. Sheets of exterior grade plywood can also be laid over the growing media to accommodate foot or wheelbarrow traffic. Foot traffic and equipment traffic should be limited over the growing media to reduce compaction.
- The growing media shall be moistened prior to planting, and then planted with the ground cover and other plant materials, per the planting plan, or in accordance with ASTM E2400. Plants shall be watered immediately after installation and routinely during establishment.
- It generally takes 12 to 18 months to fully establish the vegetated roof. An initial fertilization using slow-release fertilizer (e.g., 14-14-14) with adequate minerals is often needed to support growth. Watering is needed during the first summer. Hand weeding is also critical in the first

two years (see Table 10.1 of Weiler and Scholz-Barth, 2009, for a photo guide of common rooftop weeds).
- Most construction contracts should contain a Care and Replacement Warranty that specifies a 75% minimum survival after the first growing season of species planted and a minimum effective vegetative ground cover of 75% for flat roofs and 90% for pitched roofs.

5.2 Construction Inspection

Inspections during construction are needed to ensure that the vegetated roof is built in accordance with these specifications. Detailed inspection checklists shall be used that include signoffs by qualified individuals at critical stages of construction and confirm that the contractor's interpretation of the plan is consistent with the intent of the designer and/or manufacturer.

An experienced installer shall be retained to construct the vegetated roof system. The vegetated roof shall be constructed in sections for easier inspection and maintenance access to the membrane and roof drains. Careful construction supervision is needed during several steps of vegetated roof installation, as follows:

- During placement of the waterproofing layer, to ensure that it is properly installed and watertight.
- During placement of the drainage layer and drainage system.
- During placement of the growing media, to confirm that it meets the specifications and is applied to the correct depth.
- Upon installation of plants, to ensure they conform to the planting plan.
- Before issuing use and occupancy approvals.
- At the end of the first or second growing season to ensure desired surface cover specified in the Care and Replacement Warranty has been achieved.

6. Maintenance

6.1 Maintenance Plans

It is recommended that a Long-Term Maintenance Plan (LTMP) be developed by the design engineer. The LTMP contains a description of the stormwater system components and information on the required inspection and maintenance activities.

6.2 Maintenance Inspections and Ongoing Operations

A vegetated roof shall be inspected twice a year during the growing season to assess vegetative cover, and to look for leaks, drainage problems and any rooftop structural concerns (see Table C-61 below). In addition, the vegetated roof shall be hand-weeded to remove invasive or volunteer plants, and plants/media shall be added to repair bare areas (refer to ASTM E2400). Many practitioners also recommend an annual application of slow-release fertilizer in the first five years after the vegetated roof is installed.

If a roof leak is suspected, it is advisable to perform an electric leak survey (i.e., Electrical Field Vector Mapping) to pinpoint the exact location, make localized repairs, and then reestablish system components and ground cover.

The use of herbicides, insecticides, and fungicides shall be avoided since their presence could hasten degradation of the waterproof membrane. Also, power-washing, and other exterior maintenance operations shall be avoided so that cleaning agents and other chemicals do not harm the vegetated roof plant communities.

Table C-61. Typical Maintenance Activities Associated with Green Roofs.

Activity	Schedule
Water to promote plant growth and survival.	As needed

Inspect the vegetated roof and replace any dead or dying vegetation.	Following Construction
Inspect the waterproof membrane for leaking or cracks	Semi-annually
Annual fertilization (first five years).	Annually for first five years
Weeding to remove invasive plants.	As needed
Inspect roof drains, scuppers, and gutters to ensure they are not overgrown or have organic matter deposits. Remove any accumulated organic matter or debris	Semi-annually
Inspect the green roof for dead, dying, or invasive vegetation. Plant replacement vegetation as needed.	Annually

7. As-Built Requirements

During and after the green roof construction, the developer must have inspections and an as-built certification of the green roof conducted by a registered professional engineer and provided to the city before receipt of the Certificate of Occupancy. The as-built certification verifies that the BMP was installed as designed and approved. The following components are vital components of a properly working green roof and must be addressed in the as-built certification:

- Protection of vulnerable areas (abutting vertical walls, roof vent pipes, outlets, air conditioning units and perimeter areas) from leakage.
- Profile view of facility including typical cross-sections with dimensions.
- Growing medium specification including dry and saturated weight.
- Filter fabric specification.
- Drainage layer specification.
- Waterproof membrane specification, including root barriers.
- Stormwater piping associated with the site, including pipe materials, sizes, slopes, invert elevations at bends and connections.
- Planting and irrigation plan.

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Appendix D: Erosion and Sediment Controls

Erosion Control

Soil Loss Equation Tables

Table D-1 Soil Erodibility Factors for Pulaski County Soil Types

Symbol	Soil Name	Slope Range (%)	Soil Texture	K Factor
Cr	Crevasse	0-2	FS	0.15
Bs	Bruno	0-2	SL	0.17
Bu	Bruno	0-1	SL	0.17
CMC	Mountainburg	3-12	ST-FSL	0.17
CMF	Mountainburg	12-40	ST-FSL	0.17
LRE	Mountainburg	12-25	ST-FSL	0.17
MoD	Mountainburg	3-12	ST-FSL	0.17
SfC	Saffell	3-8	GR-FSL	0.20
LRE	Linker	12-25	GR-FSL	0.24
LkC	Linker	3-8	GR-FSL	0.24
LnC	Linker	3-8	GR-FSL	0.24
MuD	Mountainburg	3-12	FSL	0.24
MuE	Mountainburg	12-40	FSL	0.24
StC	Smithdale	3-8	FSL	0.28
StD	Smithdale	8-12	FSL	0.28
SuC	Carnasaw	3-8	FSL	0.28
CMC	Carnasaw	3-12	GR-SIL	0.32
CMF	Carnasaw	12-40	GR-SIL	0.32
CaC	Carnasaw	3-8	GR-SIL	0.32
CaD	Carnasaw	8-12	GR-SIL	0.32
CbC	Carnasaw	3-8	GR-SIL	0.32
CbD	Carnasaw	8-12	GR-SIL	0.32
La	Latainer	0-1	SIC	0.32
Me	Moreland	0-1	SIC	0.32
No	Norwood	0-1	SICL	0.32
Pe	Perry	0-1	C	0.32
Pu	Perry	0-1	C	0.32
RpB	Perry	0-3	C	0.32
SgB	Sallisaw	1-3	GR-SIL	0.32
SgC	Sallisaw	3-5	GR-SIL	0.32
ShC	Sallisaw	3-5	GR-SIL	0.32
TaB	Tiak	1-3	FSL	0.32
TuC	Tiak	3-8	FSL	0.32

Legend:		
	C = Clay	FS = Fine Sand
	GR-FSL = Gravelly Fine Sandy Loam	SIC = Silty Clay
	GR-SIL = Gravelly Silt Loam	SICL = Silty Clay Loam
	ST-FSL Stoney Fine Sandy Loam	SL = Silt Loam

USDA Soil Survey

Table D-1 Cont.

Symbol	Soil Name	Slope Range (%)	Soil Texture	K Factor
Re	Rexor	0-2	SIL	0.37
Rf	Rexor	0-2	SIL	0.37
SkC	Sallisaw	2-6	SIL	0.37
Am	Amy	0-1	SIL	0.43
Ao	Amy	0-1	SIL	0.43
ApB	Amy	0-1	SIL	0.43
Au	Amy	0-1	SIL	0.43
GeB	Guthrie	0-2	SIL	0.43
GeB	Leadvale	0-3	SIL	0.43
Ko	Keo	0-1	SIL	0.43
Ku	Keo	0-1	SIL	0.43
LdB	Leadvale	1-3	SIL	0.43
LdC	Leadvale	3-8	SIL	0.43
LeB	Leadvale	1-3	SIL	0.43
LeC	Leadvale	3-5	SIL	0.43
RmA	Rilla	0-1	SIL	0.43
RmC	Rilla	3-5	SIL	0.43
RpB	Rilla	0-5	SIL	0.43
RuA	Rilla	0-1	SIL	0.43
SkC	Leadvale	2-8	SIL	0.43
Wt	Wrightsville	0-1	SIL	0.49
Wu	Wrightsville	0-1	SIL	0.49

Legend:		
	C = Clay	FS = Fine Sand
	GR-FSL = Gravelly Fine Sandy Loam	SIC = Silty Clay
	GR-SIL = Gravelly Silt Loam	SICL = Silty Clay Loam
	ST-FSL Stoney Fine Sandy Loam	SL = Silt Loam

USDA Soil Survey

Table D-2

TABLE 3.—*Values of the topographic factor, LS, for specific combinations of slope length and steepness*[1]

Percent slope	Slope length (feet)											
	25	50	75	100	150	200	300	400	500	600	800	1,000
0.2	0.060	0.069	0.075	0.080	0.086	0.092	0.099	0.105	0.110	0.114	0.121	0.126
0.5	.073	.083	.090	.096	.104	.110	.119	.126	.132	.137	.145	.152
0.8	.086	.098	.107	.113	.123	.130	.141	.149	.156	.162	.171	.179
2	.133	.163	.185	.201	.227	.248	.280	.305	.326	.344	.376	.402
3	.190	.233	.264	.287	.325	.354	.400	.437	.466	.492	.536	.573
4	.230	.303	.357	.400	.471	.528	.621	.697	.762	.820	.920	1.01
5	.268	.379	.464	.536	.656	.758	.928	1.07	1.20	1.31	1.52	1.69
6	.336	.476	.583	.673	.824	.952	1.17	1.35	1.50	1.65	1.90	2.13
8	.496	.701	.859	.992	1.21	1.41	1.72	1.98	2.22	2.43	2.81	3.14
10	.685	.968	1.19	1.37	1.68	1.94	2.37	2.74	3.06	3.36	3.87	4.33
12	.903	1.28	1.56	1.80	2.21	2.55	3.13	3.61	4.04	4.42	5.11	5.71
14	1.15	1.62	1.99	2.30	2.81	3.25	3.98	4.59	5.13	5.62	6.49	7.26
16	1.42	2.01	2.46	2.84	3.48	4.01	4.92	5.68	6.35	6.95	8.03	8.98
18	1.72	2.43	2.97	3.43	4.21	3.86	5.95	6.87	7.68	8.41	9.71	10.9
20	2.04	2.88	3.53	4.08	5.00	5.77	7.07	8.16	9.12	10.0	11.5	12.9

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Table D-3

TABLE 9.—*Mulch factors and length limits for construction slopes*[1]

Type of mulch	Mulch Rate	Land Slope	Factor C	Length limit[2]
	Tons per acre	*Percent*		*Feet*
None	0	all	1.0	—
Straw or hay,	1.0	1-5	0.20	200
tied down by	1.0	6-10	.20	100
anchoring and				
tacking	1.5	1-5	.12	300
equipment[3]	1.5	6-10	.12	150
Do.	2.0	1-5	.06	400
	2.0	6-10	.06	200
	2.0	11-15	.07	150
	2.0	16-20	.11	100
	2.0	21-25	.14	75
	2.0	26-33	.17	50
	2.0	34-50	.20	35
Crushed stone,	135	<16	.05	200
¼ to 1½ in	135	16-20	.05	150
	135	21-33	.05	100
	135	34-50	.05	75
Do.	240	<21	.02	300
	240	21-33	.02	200
	240	34-50	.02	150
Wood chips	7	<16	.08	75
	7	16-20	.08	50
Do.	12	<16	.05	150
	12	16-20	.05	100
	12	21-33	.05	75
Do.	25	<16	.02	200
	25	16-20	.02	150
	25	21-33	.02	100
	25	34-50	.02	75

[1] From Meyer and Ports (24). Developed by an interagency workshop group on the basis of field experience and limited research data.

[2] Maximum slope length for which the specified mulch rate is considered effective. When this limit is exceeded, either a higher application rate or mechanical shortening of the effective slope length is required.

[3] When the straw or hay mulch is not anchored to the soil, C values on moderate or steep slopes of soils having K values greater than 0.30 should be taken at double the values given in this table.

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Table D-4

TABLE 10.—*Factor C for permanent pasture, range, and idle land*[1]

Vegetative canopy		Cover that contacts the soil surface						
			Percent ground cover					
Type and height[2]	Percent cover[3]	Type[4]	0	20	40	60	80	95+
No appreciable canopy		G	0.45	0.20	0.10	0.042	0.013	0.003
		W	.45	.24	.15	.091	.043	.011
Tall weeds or short brush with average drop fall height of 20 in	25	G	.36	.17	.09	.038	.013	.003
		W	.36	.20	.13	.083	.041	.011
	50	G	.26	.13	.07	.035	.012	.003
		W	.26	.16	.11	.076	.039	.011
	75	G	.17	.10	.06	.032	.011	.003
		W	.17	.12	.09	.068	.038	.011
Appreciable brush or bushes, with average drop fall height of 6½ ft	25	G	.40	.18	.09	.040	.013	.003
		W	.40	.22	.14	.087	.042	.011
	50	G	.34	.16	.08	.038	.012	.003
		W	.34	.19	.13	.082	.041	.011
	75	G	.28	.14	.08	.036	.012	.003
		W	.28	.17	.12	.078	.040	.011
Trees, but no appreciable low brush. Average drop fall height of 13 ft	25	G	.42	.19	.10	.041	.013	.003
		W	.42	.23	.14	.089	.042	.011
	50	G	.39	.18	.09	.040	.013	.003
		W	.39	.21	.14	.087	.042	.011
	75	G	.36	.17	.09	.039	.012	.003
		W	.36	.20	.13	.084	.041	.011

[1] The listed C values assume that the vegetation and mulch are randomly distributed over the entire area.

[2] Canopy height is measured as the average fall height of water drops falling from the canopy to the ground. Canopy effect is inversely proportional to drop fall height and is negligible if fall height exceeds 33 ft.

[3] Portion of total-area surface that would be hidden from view by canopy in a vertical projection (a bird's-eye view).

[4] G: cover at surface is grass, grasslike plants, decaying compacted duff, or litter at least 2 in deep.
W: cover at surface is mostly broadleaf herbaceous plants (as weeds with little lateral-root network near the surface) or undecayed residues or both.

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Table D-5 Practice Factor Value (P) for Gradient Terraces, Earth Dikes, Interceptor Dikes, and Swales

Land Slope (%)	Practice Factor
1-2	0.60
3-8	0.50
9-12	0.60
13-16	0.70
17-20	0.80
21-25	0.90

From City of Little Rock

Table D-6 Practice Factor Value (P) for Buffer Zones, Filter Strips, and Natural Vegetation

Land Slope (%)	Strip Width (ft)	Slope Length Max (ft)	Practice Factor
1-2	25 50	150 300	0.60 0.60
3-5	25 50	150 300	0.50 0.50
6-8	25 50	100 200	0.50 0.50
9-12	25 50	75 150	0.60 0.60
13-16	25 50	50 100	0.70 0.70
17-20	25 50	50 100	0.80 0.80
21-25	25 50	25 50	0.90 0.90

From City of Little Rock

Construction Temporary BMPs

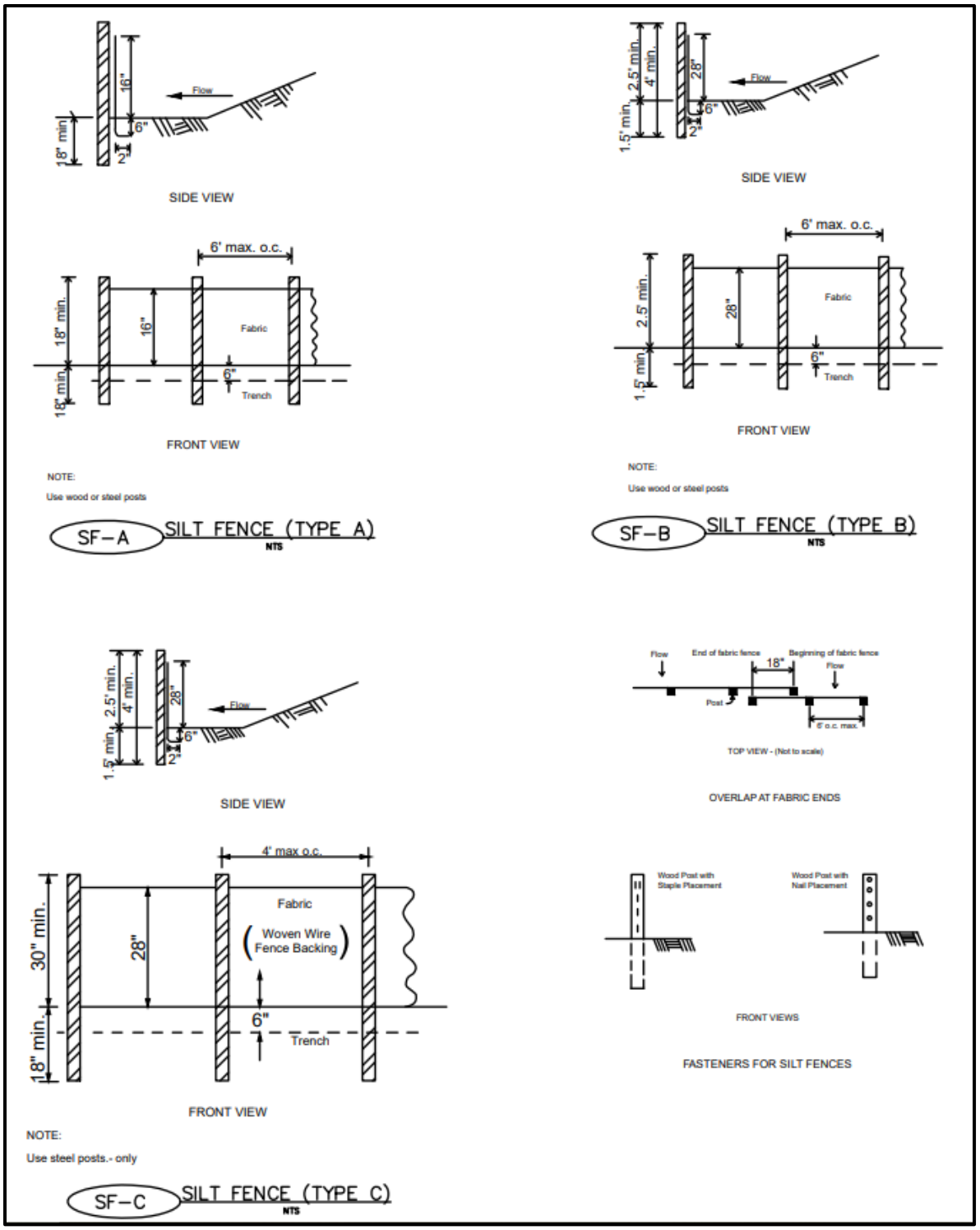


Figure D-1. Silt Fence.

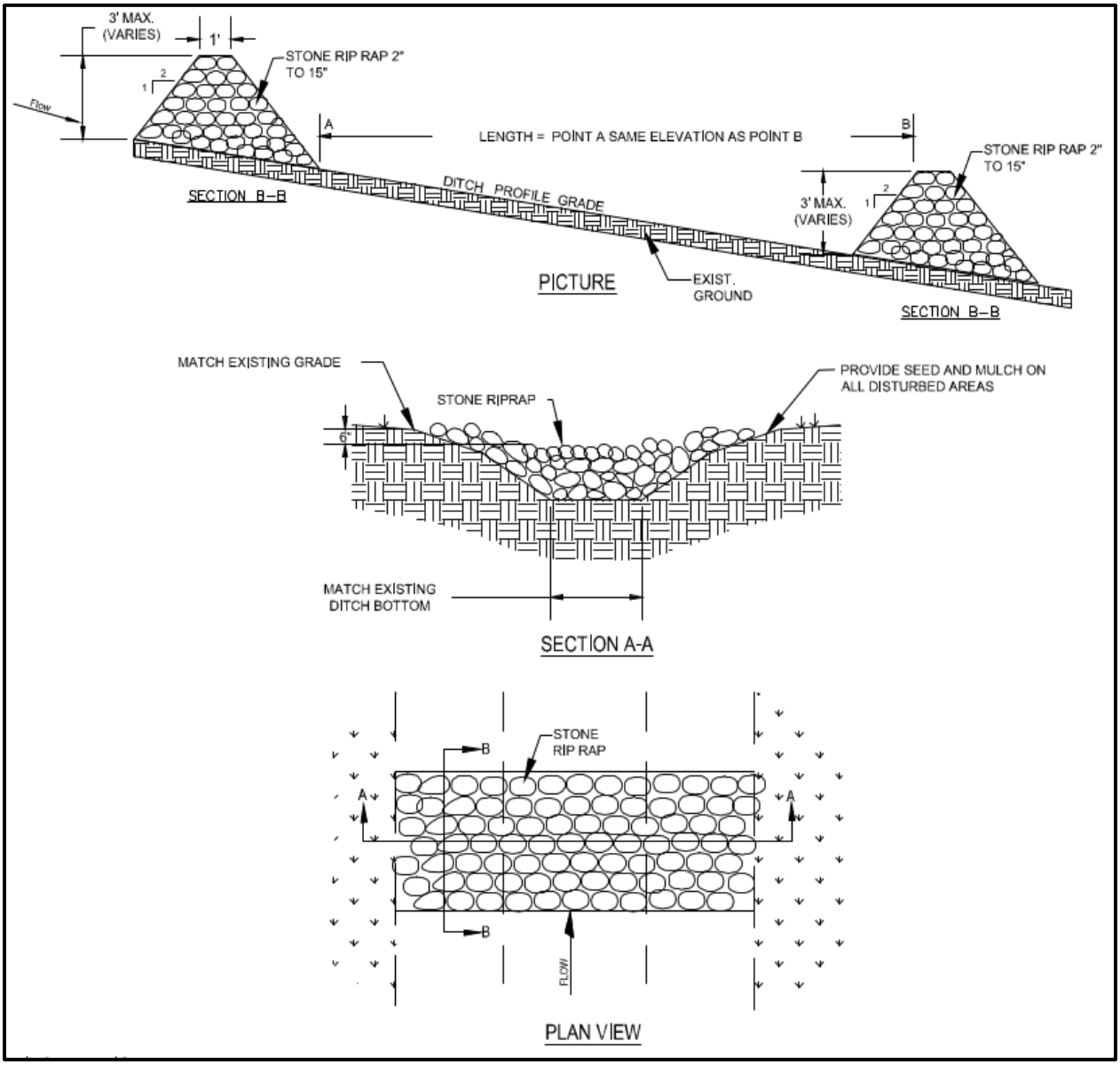


Figure D-2. Rock Check Dam.

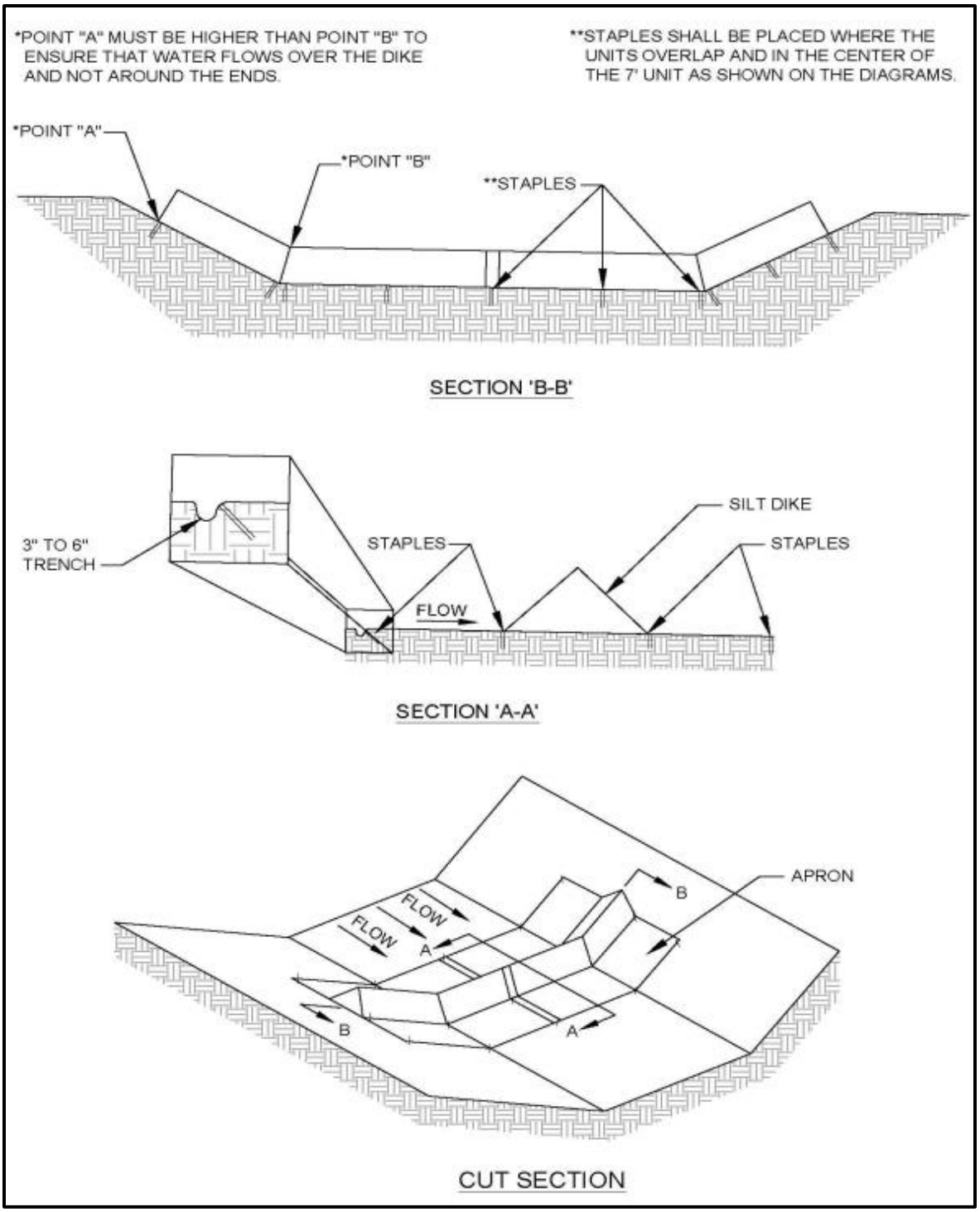


Figure D-3. Silt Dike Check Dam.

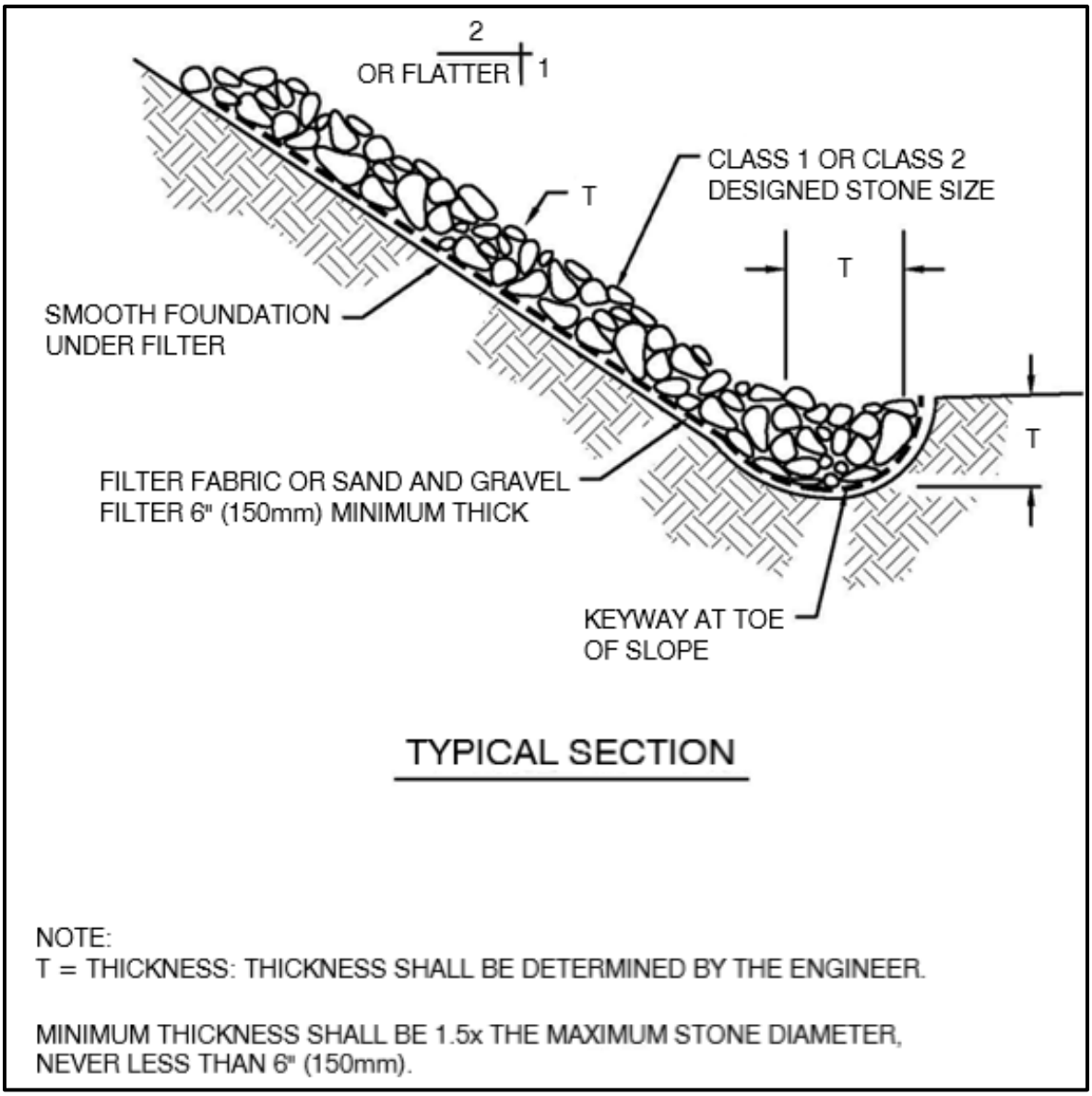


Figure D-4. Riprap Protection.

1. LOCATE STONE STABILIZED PAD AT ANY POINT WHERE VEHICULAR TRAFFIC WILL BE LEAVING THE CONSTRUCTION SITE ONTO A PUBLIC RIGHT–OF–WAY, STREET, ALLEY, SIDEWALK, OR PARKING AREA OR ANY OTHER AREA WHERE THERE IS A TRANSITION FROM BARE SOIL TO A PAVED AREA.
2. WIDTH – 20'–0 MINIMUM BUT NOT LESS THAN FULL WIDTH OF ALL POINTS OF VEHICULAR EGRESS. LENGTH – 50'–0 MINUMUM
3. MAINTAIN THE EXIT TO PREVENT TRACKING OR FLOW OF MUD ONTO PUBLIC RIGHTS– OF–WAY. PROVIDE PERIODIC TOP DRESSING WITH 1.5 – 3.5" STONE, AS CONDITIONS DEMAND. IMMEDIATELY REMOVE ALL MATERIALS SPILLED, DROPPED, WASHED, OR TRACKED FROM VEHICLE OR SITE ONTO ROADWAY OR INTO STORM DRAINS. CLEAN WHEELS TO REMOVE MUD PRIOR TO ENTRANCE ON TO PUBLIC RIGHTS–OF–WAY.
4. WHEN WASHING IS REQUIRED, DO SO ON AREAS STABILIZED WITH CRUSHED STONE DRAINING INTO AN APPROVED SEDIMENT TRAP OR SEDIMENT BASIN.

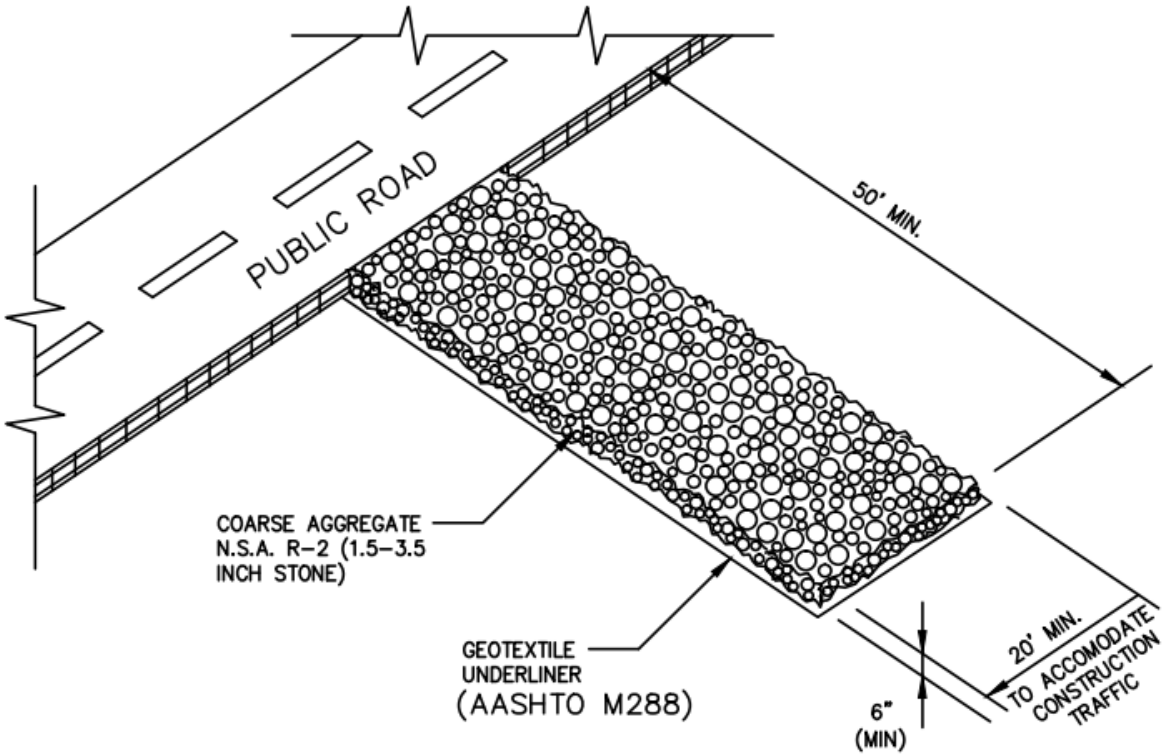


Figure D-5. Construction Entrance/Exit.

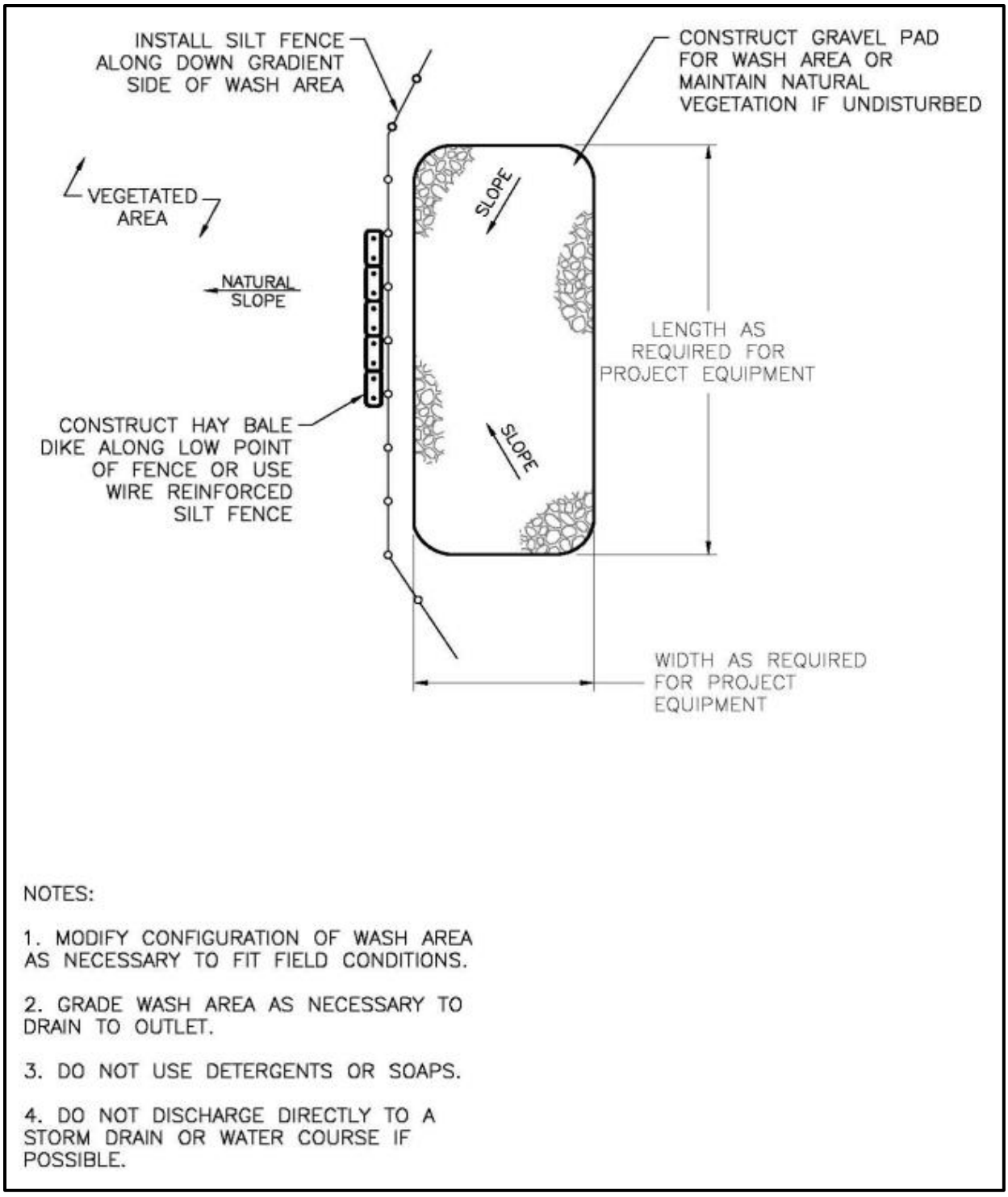


Figure D-6. Equipment Wash Area.

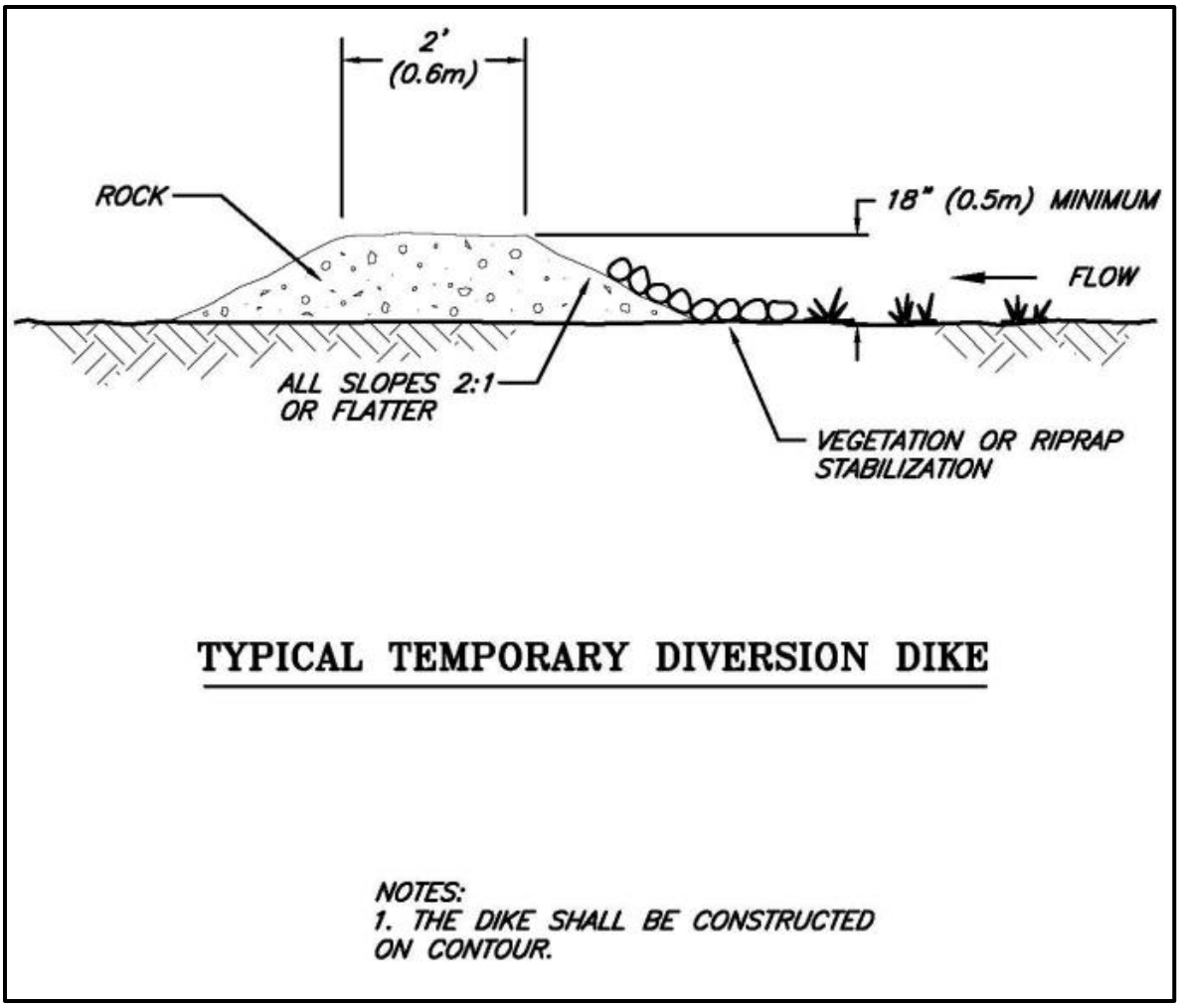


Figure D-7. Rock Dike.

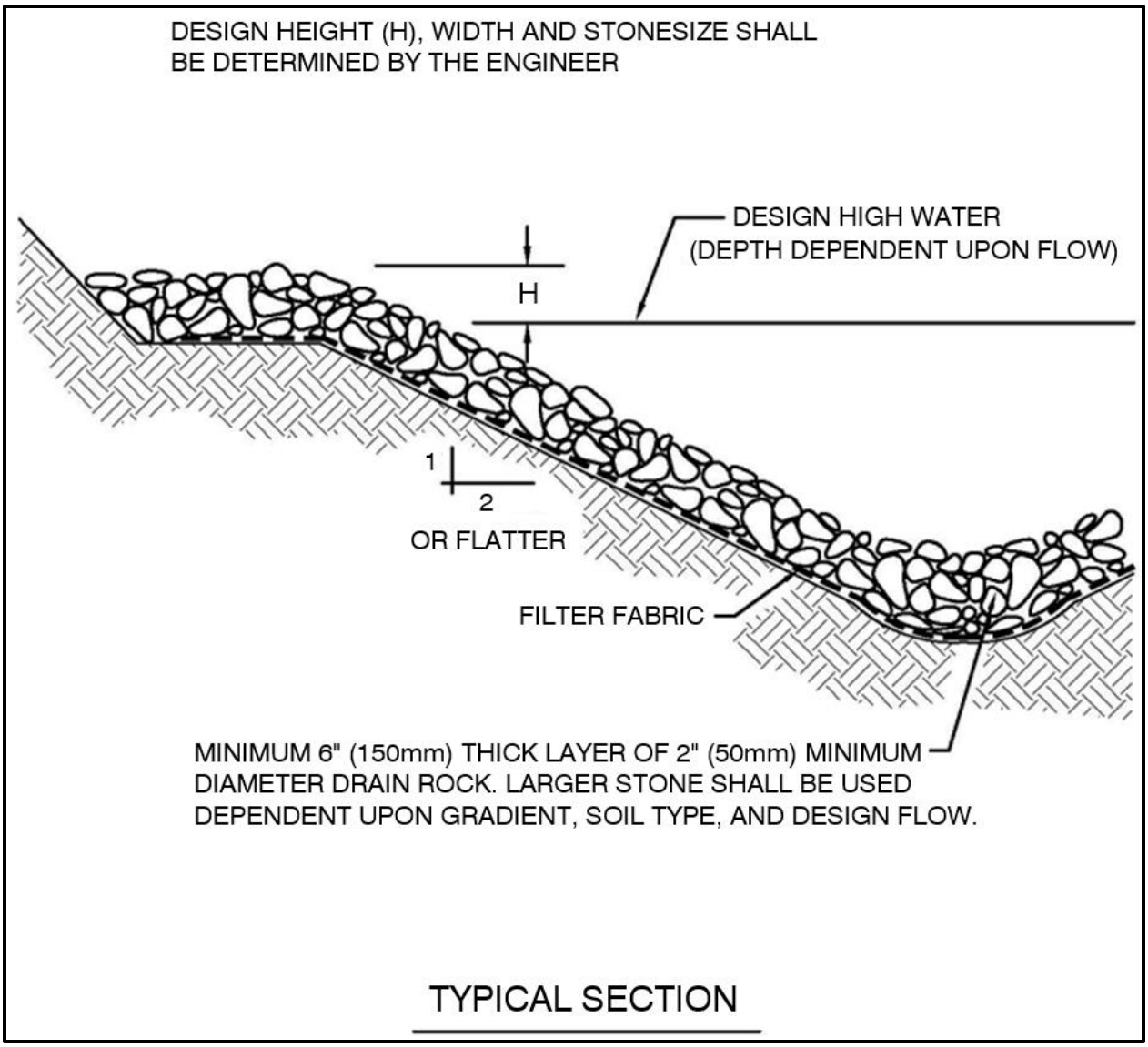


Figure D-8. Rock Lined Channel.

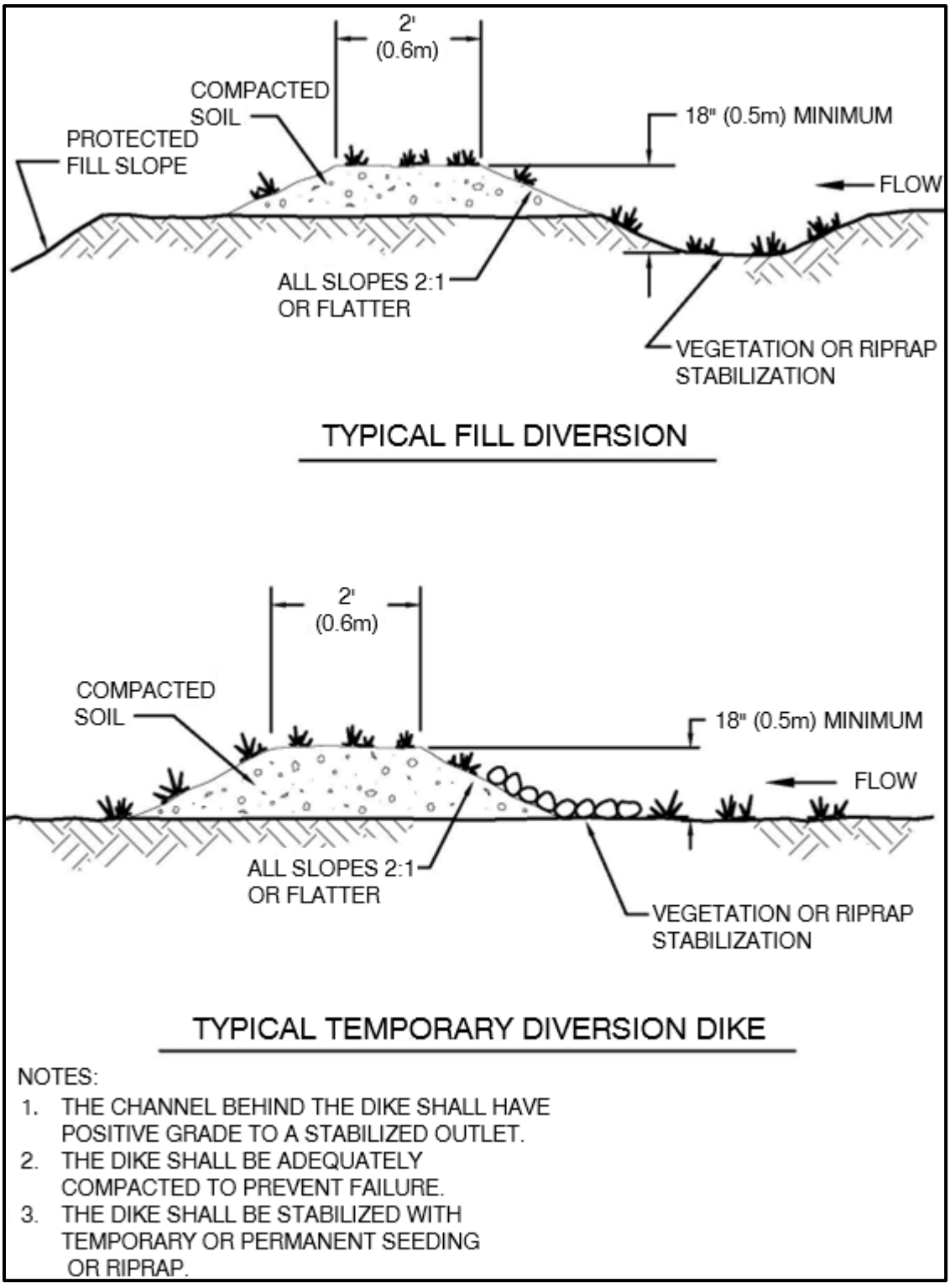


Figure D-9. Earth Dike.

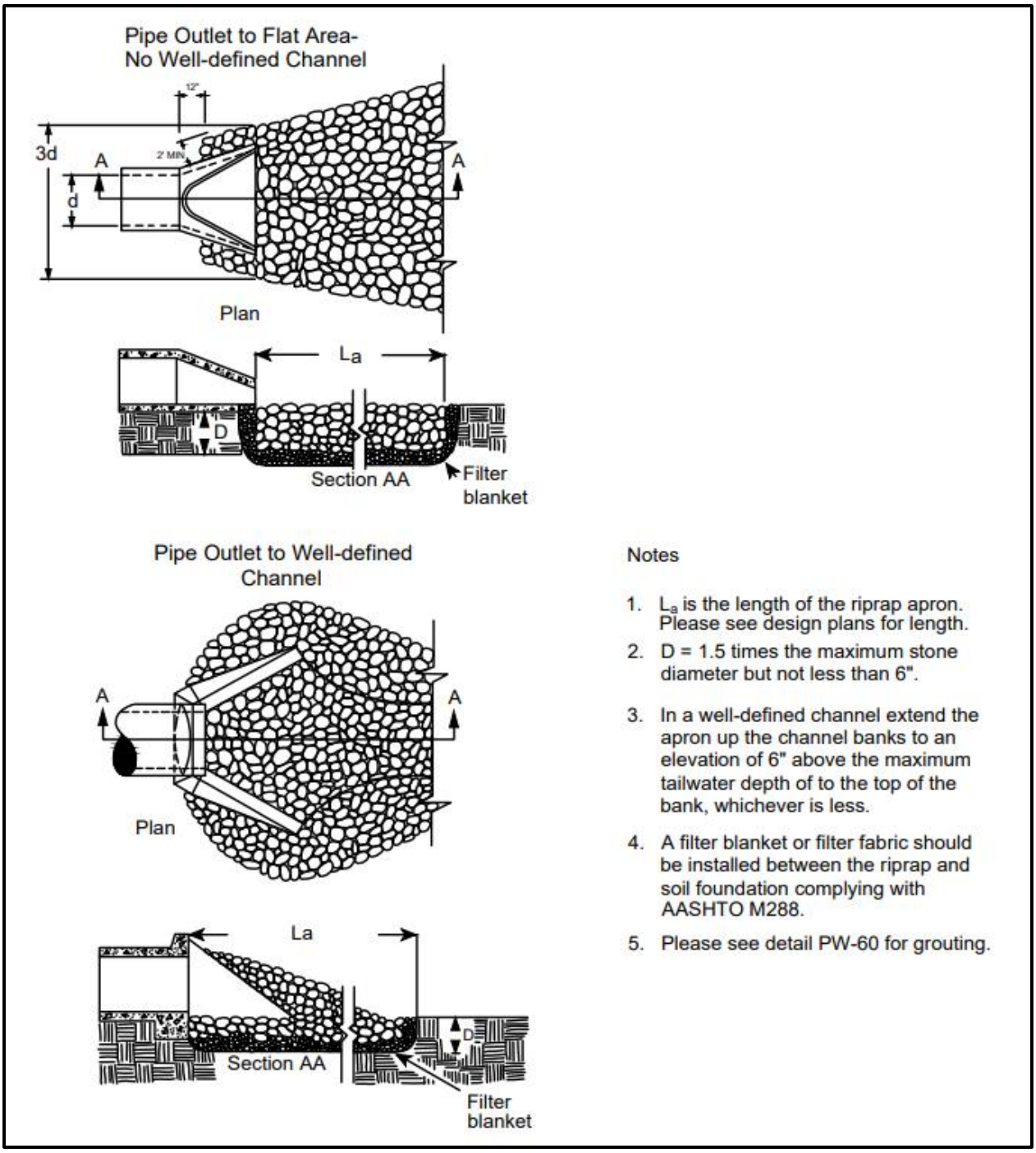


Figure D-10. Storm Drain Outlet Protection.

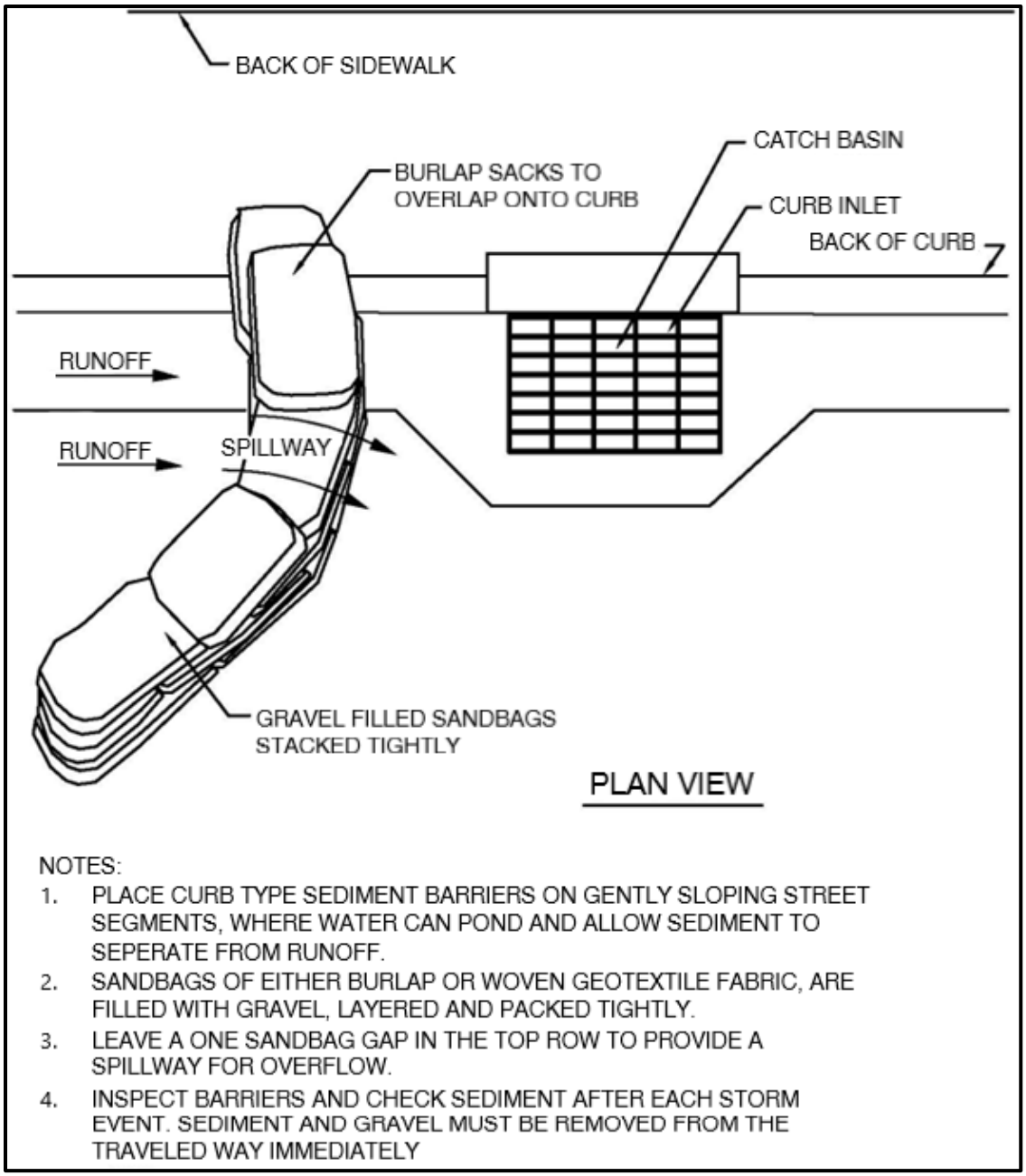
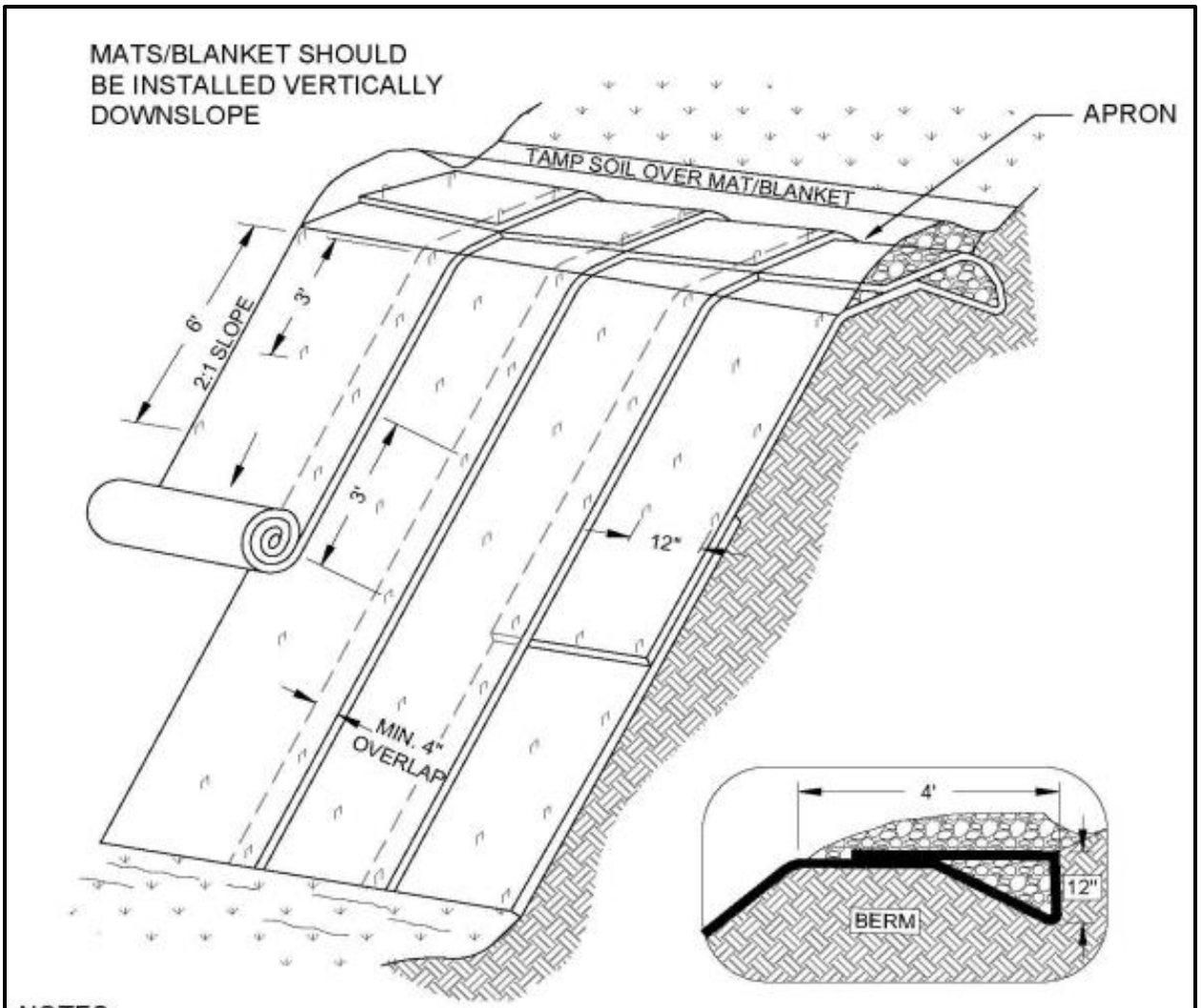


Figure D-11. Curb Gutter Protection.



NOTES:

1. SLOPE SURFACE SHALL BE FREE OF ROCKS, CLODS, STICKS, AND GRASS. MATS/BLANKETS SHALL HAVE GOOD SOIL CONTACT.

2. APPLY PERMANENT SEEDING BEFORE PLACING BLANKETS.

3. LAY BLANKETS LOOSELY AND STAKE OR STAPLE TO MAINTAIN DIRECT CONTACT WITH THE SOIL. DO NOT STRETCH.

4. REFER TO MANUFACTURER'S RECOMMENDED STAPLING PATTERN FOR STEEPNESS AND LENGTH OF SLOPE TO BE BLANKETED.

Figure D-12. Erosion Blanket Installation.

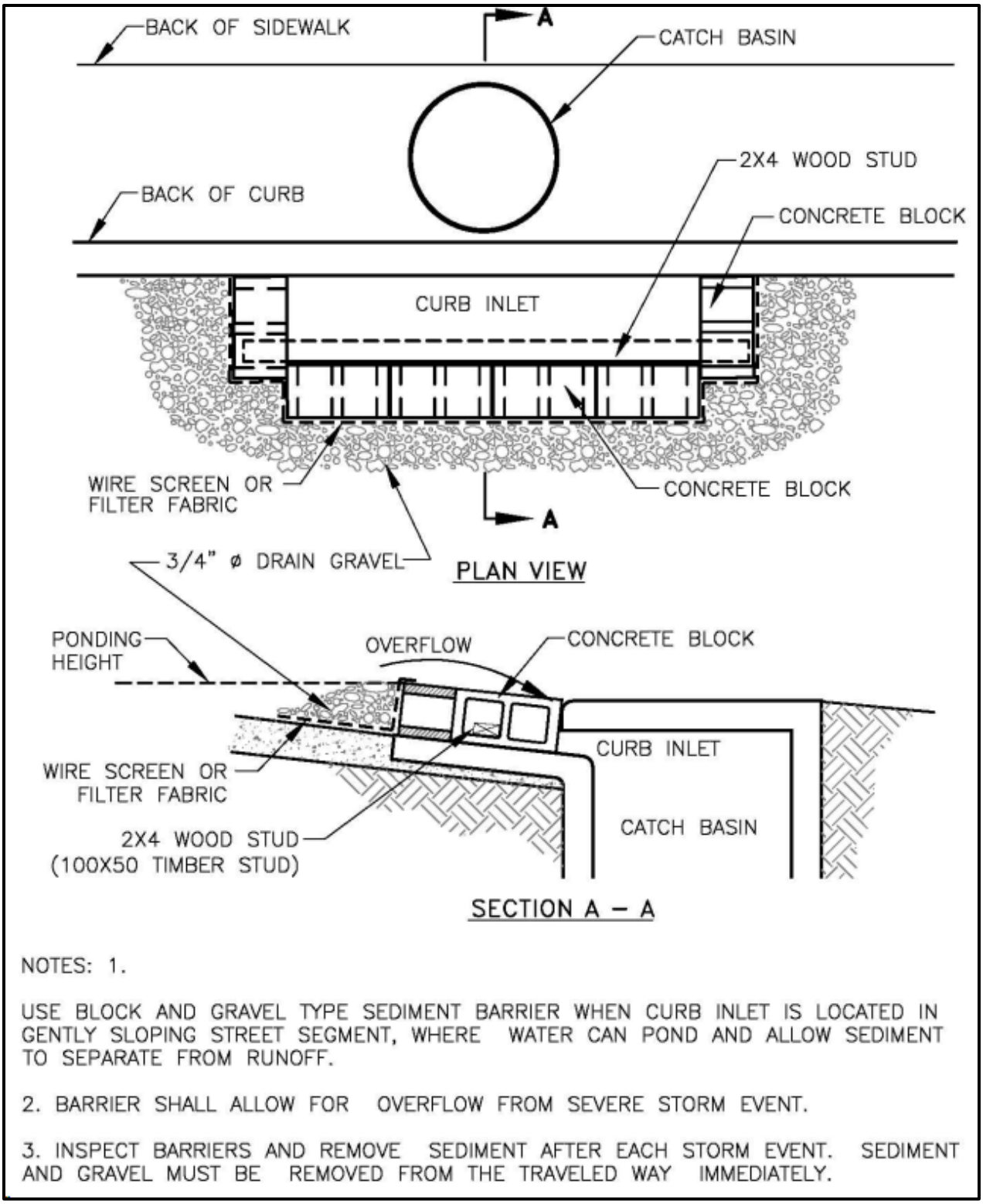
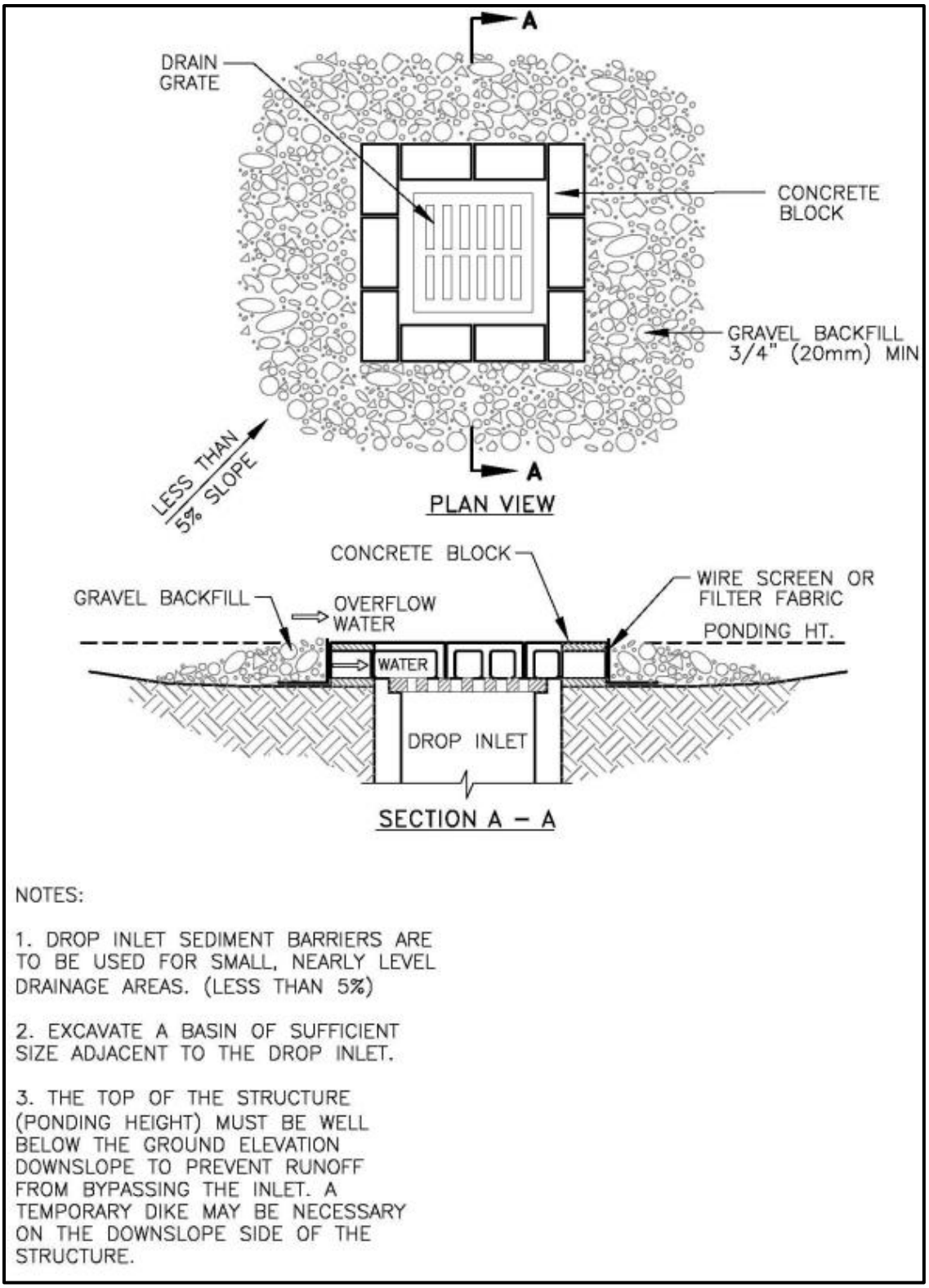


Figure D-13. Curb Inlet Protection.



NOTES:

1. DROP INLET SEDIMENT BARRIERS ARE TO BE USED FOR SMALL, NEARLY LEVEL DRAINAGE AREAS. (LESS THAN 5%)

2. EXCAVATE A BASIN OF SUFFICIENT SIZE ADJACENT TO THE DROP INLET.

3. THE TOP OF THE STRUCTURE (PONDING HEIGHT) MUST BE WELL BELOW THE GROUND ELEVATION DOWNSLOPE TO PREVENT RUNOFF FROM BYPASSING THE INLET. A TEMPORARY DIKE MAY BE NECESSARY ON THE DOWNSLOPE SIDE OF THE STRUCTURE.

Figure D-14. Block and Gravel Drop Inlet Sediment Barrier.

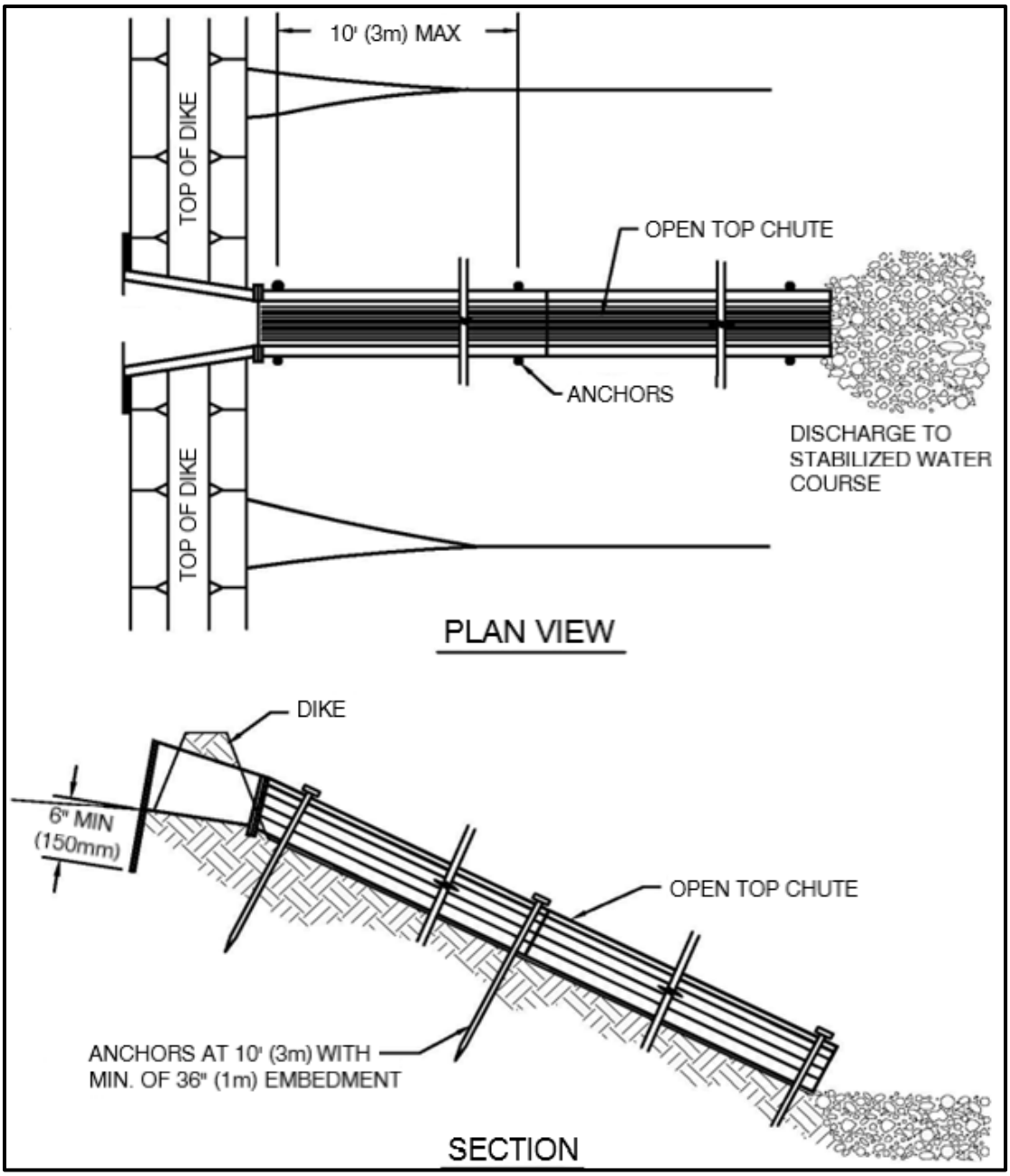


Figure D-15. Flexible Down Drain.

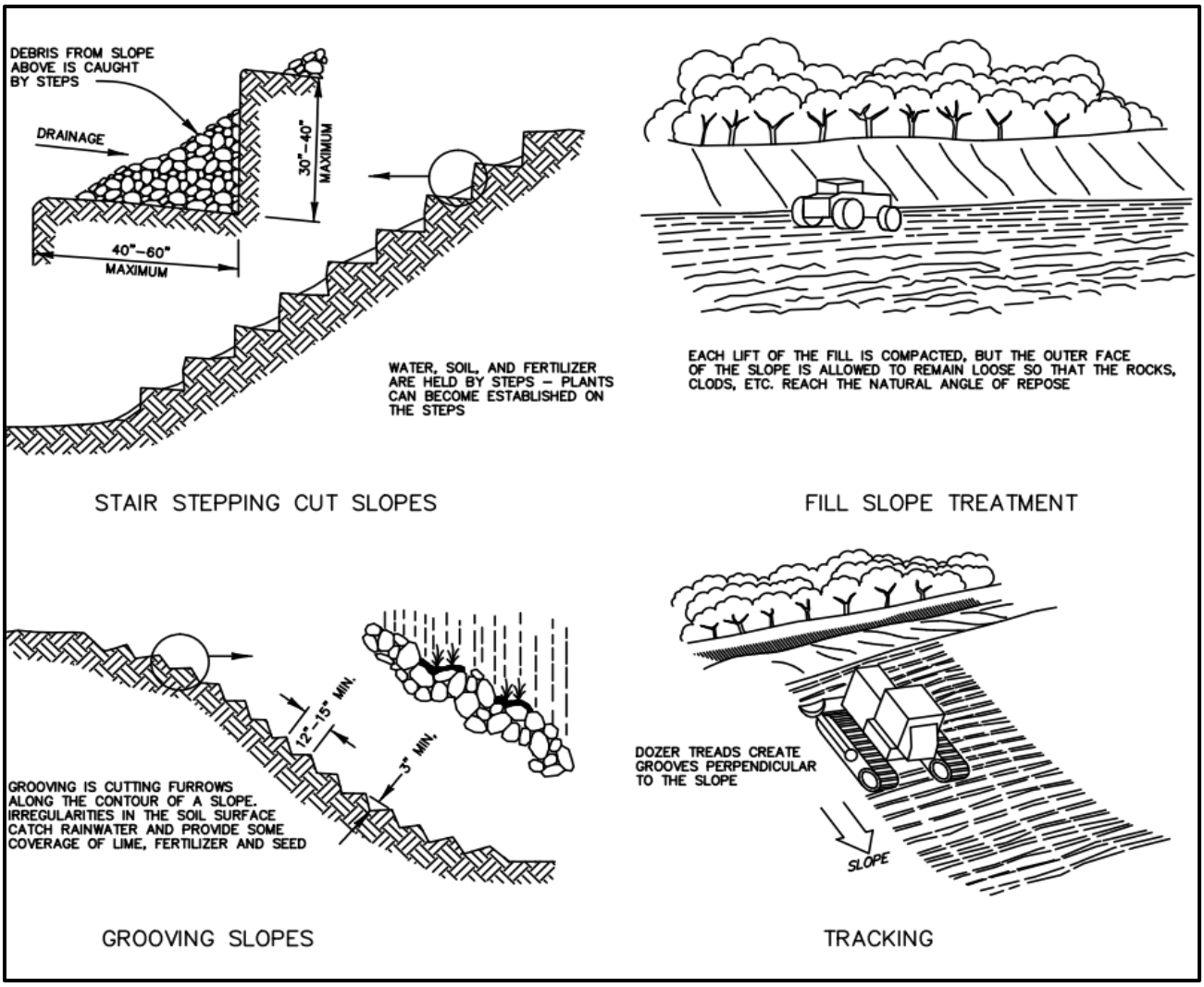


Figure D-16. Surface Roughening.

For more information or examples of construction BMPs, visit the EPA's website and browse through the National Menu of Best Management Practices (BMPs) for Stormwater-Construction. National Menu of Best Management Practices (BMPs) for Stormwater-Construction | US EPA

Appendix E: References

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