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Instruction Sheet

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Title: 2000 Maryland Stormwater Design Manual, Vol. I & II
(Maryland Department of the Environment, April 2000)

Proposed Supplement No. Notice of Final Action

To: INTERESTED PARTIES

Instructions: The document 2000 Maryland Stormwater Design Manual, Volumes I & II is being proposed for adoption. This submittal includes the following **REVISED** pages (all pages are inclusive):

Chapter 1:	1.15—1.16
Chapter 2:	2.3—2.8 2.13—2.14
Chapter 3:	3.9—3.16 3.39—3.40
Chapter 4:	4.9—4.10 4.13—4.14
Chapter 5:	5.1—5.2 5.5—5.6 5.9—5.12
Appendix B.2:	B.2.1—B.2.4
Appendix B.3:	Title Page B.3.5—B.3.10
Appendix C.1:	C.1.1—C.1.2
Appendix C.2:	Title Page C.2.1—C.2.29
Appendix D.1:	D.1.3—D.1.4

Check to make certain that you have all the pages listed above. Please remove and replace the existing pages with these revised pages.

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prevention plan requirement applies to both existing and new industrial sites.

Standard No. ***13*** *Stormwater discharges from land uses or activities with higher potential for pollutant loadings, defined as hotspots in Chapter 2, may require the use of specific structural BMPs and pollution prevention practices. In addition, stormwater from a hotspot land use may not be infiltrated without proper pretreatment.*

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Standard No. ***14*** *In Maryland, local governments are usually responsible for most stormwater management review authority. Therefore, prior to design, applicants should always consult with their local reviewing agency to determine if they are subject to additional stormwater design requirements. In addition, certain earth disturbances may require NPDES construction general permit coverage from MDE (see Appendix D.7).*

Section 1.3 How to Use the Manual

The Maryland Stormwater Design Manual is provided in two volumes. This *first volume* provides designers a general overview on how to size, design, select and locate BMPs at a new development site to comply with State stormwater performance standards. The *second volume* contains appendices with more detailed information on landscaping, BMP construction specifications, step-by-step BMP design examples and other assorted design tools.

Section 1.3.1 VOLUME ONE: STORMWATER MANAGEMENT CRITERIA

The first volume of the manual is organized as follows:

Chapter 1. Introduction to the Manual.

Chapter 2. Unified Stormwater Sizing Criteria. This chapter explains the five new sizing criteria for water quality, recharge, channel protection, overbank flood control, and extreme flood management in the State of Maryland. The chapter also outlines the basis for design calculations. Three step-by-step design examples are provided to familiarize the reader with the new procedures for computing storage volumes under the five sizing criteria. The chapter also briefly outlines the six groups of acceptable BMPs that can be used to meet recharge and water quality volume sizing criteria. Acceptable BMP groups are:

- Stormwater Ponds
- Stormwater Wetlands
- Infiltration Practices
- Filtering Systems
- Open Channel Practices
- Non-structural Practices

Lastly, the chapter presents a list of land uses or site activities that have been designated as "stormwater hotspots." If a development site is considered a "hotspot," it may have special requirements for pollution prevention and groundwater protection.

Chapter 3. Performance Criteria for Urban BMP Design. The third chapter presents specific performance criteria and guidelines for the design of five groups of structural BMPs. The performance criteria for each group of BMPs are based on six factors:

- General Feasibility
- Conveyance
- Pretreatment
- Treatment Geometry
- Landscaping
- Maintenance

Figure 2.1 Location of the Eastern and Western Rainfall Zones in Maryland (For use in selecting the appropriate WQ_v equation.)

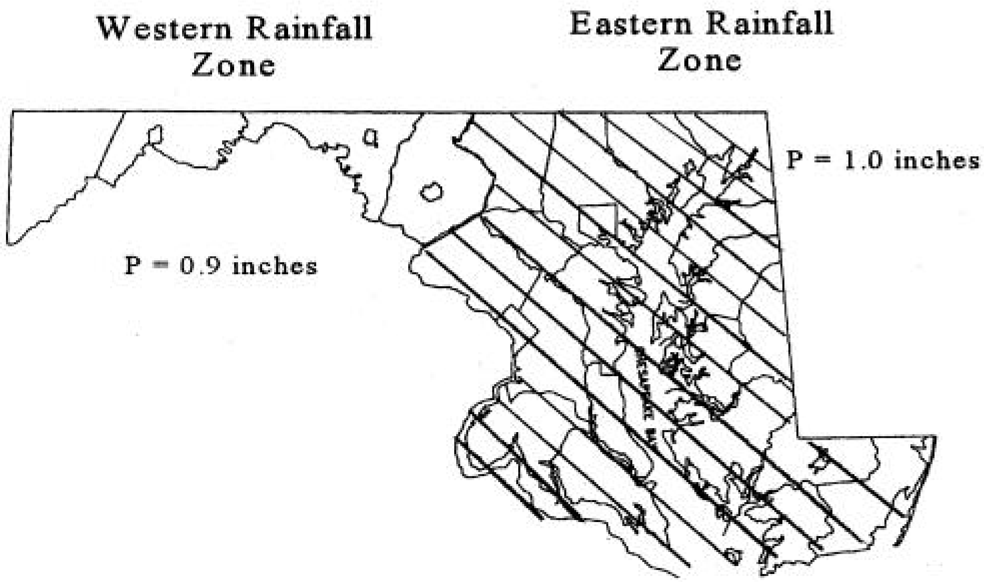
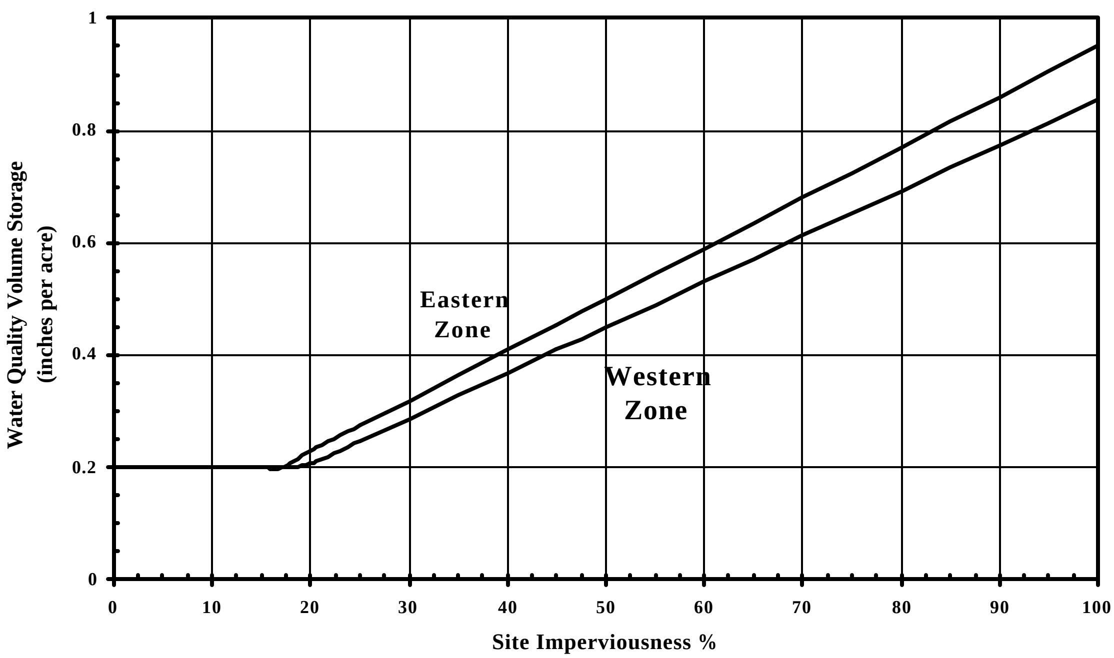


Figure 2.2 Relationship between Impervious Cover and the Water Quality Volume



Basis for Determining Water Quality Treatment Volume

As a basis for design, the following assumptions may be made:

- **Measuring Impervious Cover**: the measured area of a site plan that does not have vegetative or permeable cover shall be considered total impervious cover. Where direct measurement of impervious cover is impractical, NRCS land use/impervious cover relationships can be used to estimate impervious cover (see Table 2.2a in TR-55, NRCS, 1986). Estimates should be based on actual land use and homogeneity.

- **Multiple Drainage Areas**: When a project contains or is divided by multiple drainage areas, the WQ_v volume shall be addressed for each drainage area. See the design examples in Chapter 2, Section 2.6.

- **Offsite Drainage Areas***:* The WQ_v shall be based on the impervious cover of the proposed site. Offsite existing impervious areas may be excluded from the calculation of the water quality volume requirements.

- **Sensitive Streams**: Consult with the appropriate local review authority to determine if a greater WQ_v is warranted to protect sensitive streams.

- **BMP Treatment**: The final WQ_v shall be treated by an acceptable BMP(s) from the list presented in Chapter 2, Section 2.7, or an equivalent practice allowed by the appropriate review authority.

- **Subtraction for Structural Practices***:* Where structural practices for treating the Re_v are employed upstream of a BMP, the Re_v may be subtracted from the WQ_v used for design.

- **Subtraction for Non-structural Practices**: Where non-structural practices are employed in the site design, the WQ_v volume can be reduced in accordance with the conditions outlined in Chapter 5.

- **Determining Peak Discharge for WQ_v Storm**: When designing flow splitters for off-line practices, consult the small storm hydrology method provided in Appendix D.10.

- **Extended Detention for Water Quality Volume**: The water quality requirement can be met by providing a 24 hour drawdown of a portion of the water quality volume (WQ_v) in conjunction with a stormwater pond or wetland system as described in Chapter 3. Referred to as ED, this is different than providing the extended detention of the one-year storm for the channel protection volume (Cp_v). The ED portion of the WQ_v may be included when routing the Cp_v.

Section 2.2 Recharge Volume Requirements (Re_v)

The criteria for maintaining recharge is based on the average annual recharge rate of the hydrologic soil group(s) (HSG) present at a site as determined from USDA, NRCS Soil Surveys or from detailed site investigations. More specifically, each specific recharge factor is based on the USDA average annual recharge volume per soil type divided by the annual rainfall in Maryland (42 inches per year) and multiplied by 90%. This keeps the recharge calculation consistent with the WQ_v methodology. Thus, an annual recharge volume requirement is specified for a site as follows:

Site Recharge Volume Requirement

$Re_v = [(\boldsymbol{S})(\boldsymbol{R_v})(\boldsymbol{A})]/12$ (percent volume method)

where: R_v = 0.05 + 0.009(I) where I is percent impervious cover

A = site area in acres

Re_v = $(\boldsymbol{S})(\boldsymbol{A_i})$ (percent area method)

where: A_i = the measured impervious cover

Hydrologic Soil Group	Soil Specific Recharge Factor (*S*)
A	0.38
B	0.26
C	0.13
D	0.07

The recharge volume is considered part of the total WQ_v that must be provided at a site and can be achieved either by a structural practice (e.g., infiltration, bioretention), a non-structural practice (e.g., buffers, disconnection of rooftops), or a combination of both.

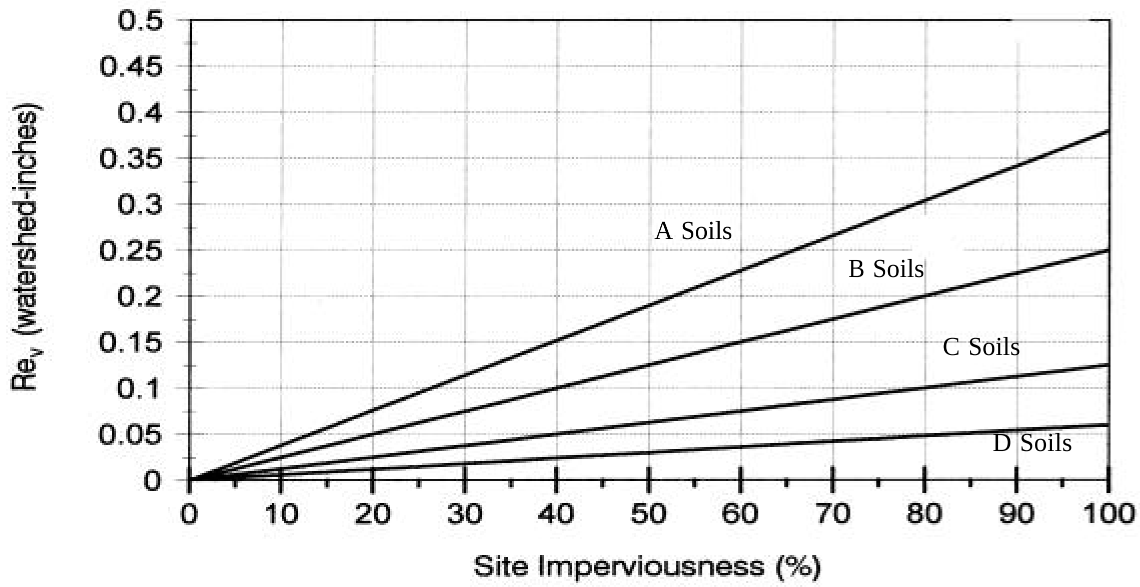
Drainage areas having no impervious cover and no proposed disturbance during development may be excluded from the Re_v calculations. Designers are encouraged to use these areas as non-structural practices for Re_v treatment (see Chapter 5, "Stormwater Credits for Innovative Site Planning").

Note: Re_v and WQ_v are inclusive. When treated separately, the Re_v may be subtracted from the WQ_v when sizing the water quality BMP (see page 2.4, 'Subtraction for Structural Practices).

The intent of the recharge criteria is to maintain existing groundwater recharge rates at development sites. This helps to preserve existing water table elevations thereby maintaining the hydrology of streams and wetlands during dry weather. The volume of recharge that occurs on a site depends on slope, soil type, vegetative cover, precipitation and evapo-transpiration. Sites with natural ground cover, such as forest and meadow, have higher recharge rates, less runoff, and greater transpiration losses under most conditions. Because development increases impervious surfaces, a net decrease in recharge rates is inevitable.

The relationship between Re_v and site imperviousness is shown in graphical form in Figure 2.3.

Figure 2.3 Relationship between Re_v and Site Impervious Cover



Basis for Determining Recharge Volume

- If more than one HSG is present at a site, a composite soil specific recharge factor shall be computed based on the proportion of total site area within each HSG. The recharge volume provided at the site shall be directed to the most permeable HSG available.

- The "percent volume" method is used to determine the Re_v treatment requirement when structural practices are used to provide recharge. These practices must provide seepage into the ground and may include infiltration and exfiltration structures (e.g., infiltration, bioretention, dry swales or sand filters with storage below the underdrain). Structures that require impermeable liners, intercept groundwater, or are designed for trapping sediment (e.g., forebays) may not be used. In this method, the volume of runoff treated by structural practices shall meet or exceed the computed recharge volume.

- The "percent area" method is used to determine the Re_v treatment requirements when non-structural practices are used. Under this method, the recharge requirement is evaluated by mapping the percent of impervious area that is effectively treated by an acceptable non-structural practice and comparing it to the minimum recharge requirements.

- Acceptable non-structural practices include filter strips that treat rooftop or parking lot runoff, sheet flow discharge to stream buffers, and grass channels that treat roadway runoff (see Chapter 5.)

- The recharge volume criterion does not apply to any portion of a site designated as a stormwater hotspot nor any project considered as redevelopment. In addition, the appropriate local review authority may alter or eliminate the recharge volume requirement if the site is situated on unsuitable soils (e.g., marine clays), karst or in an urban redevelopment area. In this situation, non-structural practices (percent area method) should be implemented to the maximum extent practicable and the remaining or untreated Re_v included in the WQ_v treatment.

- If Re_v is treated by structural or non-structural practices separate and upstream of the WQ_v treatment, the WQ_v is adjusted accordingly.

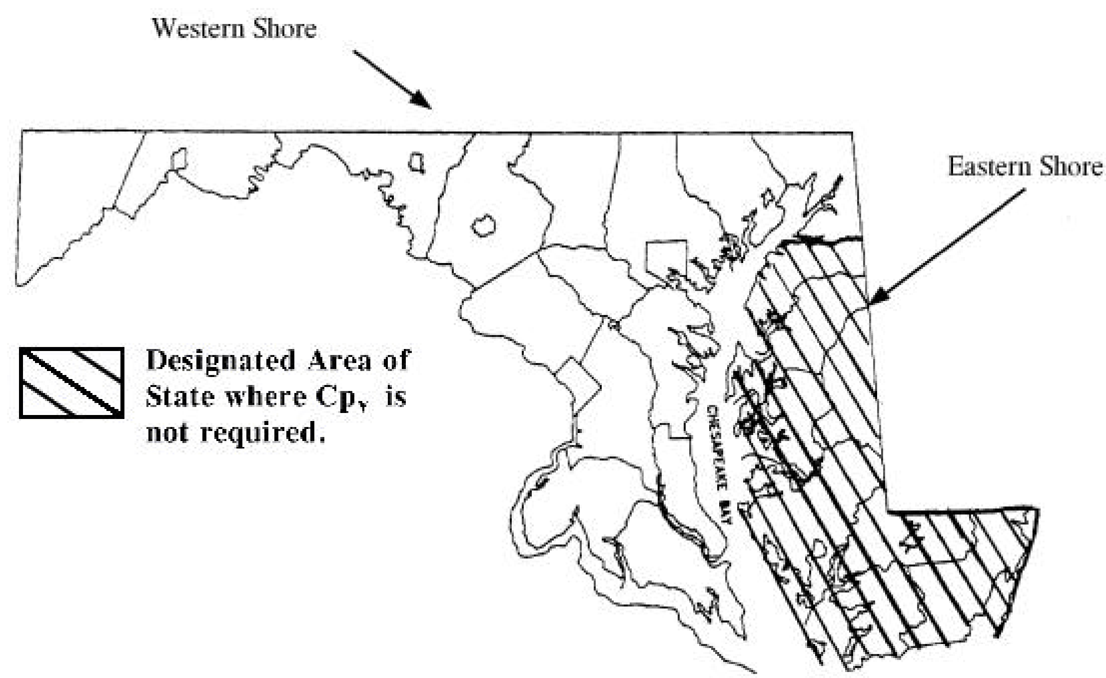
Section 2.3 Channel Protection Storage Volume Requirements (Cp_v)

To protect channels from erosion, **24 hour extended detention of the one-year, 24 hour storm event** (MDE, 1987) shall be provided. In Use III and IV watersheds, only 12 hours of extended detention shall be provided. The rationale for this criterion is that runoff will be stored and released in such a gradual manner that critical erosive velocities during bankfull and near-bankfull events will seldom be exceeded in downstream channels.

The Cp_v requirement does not apply to direct discharges to tidal water or Maryland's Eastern Shore (as defined in Figure 2.4) unless specified by an appropriate review authority on a case by case basis. Local governments may wish to use alternative methods to provide equivalent stream channel protection such as the Distributed Runoff Control method or bankfull capacity/duration criteria (MacRae, 1993).

The method for determining the Cp_v requirement is detailed in Appendix D.11. A detention pond or underground vault is normally needed to meet the Cp_v requirement (and subsequent Q_{p10} and Q_f criteria). Schematics of a typical design are shown in Figures 2.5.

Figure 2.4 Regions of Maryland Not Subject to the Channel Protection Requirement (Cp_v)



Section 2.5 Extreme Flood Volume (Q_f)

The intent of the extreme flood criteria is to (a) prevent flood damage from large storm events, (b) maintain the boundaries of the pre development 100-year Federal Emergency Management Agency (FEMA) and/or locally designated floodplain, and (c) protect the physical integrity of BMP control structures. This is typically done in two ways:

100-Year Control: requires storage to attenuate the post development 100-year, 24 hour peak discharge (Q_f) to pre development rates. The Q_f is the most stringent and expensive level of flood control and is generally not needed if the downstream development is located out of the 100-year floodplain. In many cases, the conveyance system leading to a stormwater structure is designed based on the discharge rate for the ten-year storm (Qp_{10}). In these situations, the conveyance systems may be the limiting hydrologic control.

Reserve Ultimate 100-Year Floodplain: 100-year storm control may be required by an appropriate review authority if:

- buildings or development are located within the ultimate 100-year floodplain, or
- the reviewing authority does not completely control the 100-year floodplain.

Hydraulic/hydrologic investigations may be required to demonstrate that downstream roads, bridges and public utilities are adequately protected from the Q_f storm. These investigations typically extend to the first downstream tributary of equal or greater drainage area or to any downstream dam, highway, or natural point of restricted stream flow.

Basis for Determining Extreme Flood Criteria

- Consult with the appropriate review authority to determine the analyses required for the Q_f storm.

- The same hydrologic and hydraulic methods used for overbank flood control shall be used to analyze Q_f.

- In addition, off-site areas should be modeled as "ultimate condition" when the 100-year design storm event is analyzed. Table 2.2 indicates the depth of rainfall (24 hour) associated with the 100-year storm event for all counties in the State of Maryland

Section 2.6 Design Examples: Computing Stormwater Storage Volumes

Design examples are provided only to illustrate how the five stormwater management sizing criteria are computed for hypothetical development projects. These design examples are also utilized elsewhere in the manual to illustrate structural and non-structural BMP design.

Design Example No. 1: Residential Development - Reker Meadows

Site data and the layout of the Reker Meadows subdivision are shown in Figure 2.6.

Step 1. Compute WQ_v Volume

$$WQ_v = \frac{(P)(R_v)(A)}{12}$$

Step 1a. Compute Volumetric Runoff Coefficient (R_v)

R_v = 0.05 + (0.009) (I); I = 13.8 acres/38.0 acres = 36.3%
= 0.05 + (0.009) (36.3) = 0.38

Step 1b. Determine Rainfall Zone for WQ_v Formula

Location	Rainfall (*P*)
Eastern Rainfall Zone	1.0 inches
Western Rainfall Zone	0.9 inches
Minimum WQv (I ≤ 15%)	0.2 inches

Because this site is located in the Western Rainfall Zone, use 0.9″ of rainfall to determine WQ_v.

Step 1c. Compute WQ_v

WQ_v = [(0.9″) (R_v) (A)]/12
= [(0.9″)(0.38)(38.0 ac)]/12
= 1.08 ac-ft

Check Minimum: [(0.2″)(38.0 ac)]/12 = 0.63 ac-ft < 1.08 ac-ft
∴ Use WQ_v = 1.08 ac-ft

6) *material and construction specifications for the embankment shall be in accordance with MD 378 code, except that fill material for the embankment shall conform to Unified Soil Classification GC, SC, SM, MH, ML, CH, or CL, and no cutoff trench is required.*

7) *woody vegetation is prohibited on the embankment.*

A pond structure requires review and approval by the MDE Dam Safety Division if any of the following conditions apply:

a) *the proposed embankment is twenty feet or greater in height from the upstream toe to the top of dam, or*

b) *the contributing drainage area is a square mile (640 acres) or greater, or*

c) *the structure is classified as "high" or "intermediate" hazard, according to the MD Dam Safety Manual, or*

d) *the proposed pond is in USE III waters.*

3.1.2 Pond Conveyance Criteria

When reinforced concrete pipe is used for the principal spillway to increase its longevity, "O-ring" gaskets (ASTM C-361) should be used to create watertight joints and should be inspected during installation.

Inlet Protection

Inlet pipes to the pond should not be fully submerged at normal pool elevations.

A forebay shall be provided at each inlet, unless the inlet provides less than 10% of the total design storm inflow to the pond.

Adequate Outfall Protection

Flared pipe sections that discharge at or near the stream invert or into a step-pool arrangement should be used at the spillway outlet.

The channel immediately below the pond outfall shall be modified to prevent erosion and conform to natural dimensions in the shortest possible distance, typically by use of large rip-rap placed over filter cloth.

A stilling basin or other outlet protection should be used to reduce flow velocities from the principal spillway to non-erosive velocities (see Appendix D.12 for critical non-erosive velocities for grass and soil).

If a pond daylights 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. Excessive use of rip-rap should be avoided to reduce stream warming.

Pond Liners

When a pond is located in karst topography, gravelly sands or fractured bedrock, a liner may be needed to sustain a permanent pool of water. If geotechnical tests confirm the need for a liner, acceptable options include: (a) 6 to 12 inches of clay soil (minimum 15% passing the #200 sieve and a maximum permeability of 1 x 10^{-5} cm/sec), (b) a 30 mil poly-liner, (c) bentonite, (d) use of chemical additives (see NRCS Agricultural Handbook No. 387, dated 1971, Engineering Field Manual), or (e) other suitable materials approved by the appropriate review authority.

3.1.3 Pond Pretreatment Criteria

Sediment Forebay

Each pond shall have a sediment forebay or equivalent upstream pretreatment. The forebay shall consist of a separate cell, formed by an acceptable barrier.

The forebay shall be sized to contain 0.1 inches per impervious acre of contributing drainage. The forebay storage volume counts toward the total WQ_v requirement. Exit velocities from the forebay shall be non-erosive.

Direct maintenance access for appropriate equipment shall be provided to the forebay.

The bottom of the forebay may be hardened (e.g., using concrete, paver blocks, etc.) to make sediment removal easier.

A fixed vertical sediment depth marker should be installed in the forebay to measure sediment deposition over time.

3.1.4 Pond Treatment Criteria

Minimum Water Quality Volume (WQ_v)

Ponds shall be designed to capture and treat the computed WQ_v through any combination of permanent pool, extended detention (ED) or wetland. If treated separately, the Re_v may be subtracted from the WQ_v for pond design.

It is generally desirable to provide water quality treatment off-line when topography, head and space permit (e.g., apart from stormwater quantity storage).

Water quality storage can be provided in multiple cells. Performance is enhanced when multiple treatment pathways are provided by using multiple cells, longer flowpaths, high surface area to volume ratios, complex microtopography, and/or redundant treatment methods (combinations of pool, ED, and wetland).

If ED is provided in a pond, storage for WQ_v and Cp_v shall be computed and routed separately (e.g., the WQ_v requirement cannot be met simply by providing Cp_v storage for the one-year storm).

Minimum Pond Geometry

Flowpaths from inflow points to outlets shall be maximized. Flowpaths of 1.5:1 (length relative to width) and irregular shapes are recommended.

3.1.5 Pond Landscaping Criteria

Pond Benches

The perimeter of all deep permanent pool areas (four feet or greater in depth) shall be surrounded by two benches with a combined minimum width of 15 feet:

- *A safety bench* that extends outward from the normal water edge to the toe of the pond side slope. The maximum slope of the safety bench shall be 6%.

- *An aquatic bench* that extends inward from the normal shoreline and has a maximum depth of eighteen inches below the normal pool water surface elevation. An aquatic bench is not required in forebays.

Landscaping Plan

A landscaping plan for a stormwater pond and its buffer shall be prepared to indicate how aquatic and terrestrial areas will be vegetatively stabilized and established. Landscaping guidance for stormwater ponds is provided in Appendix A.

Wherever possible, wetland plants should be encouraged in a pond design, either along the aquatic bench (fringe wetlands), the safety bench and side slopes (emergent wetlands) or within shallow areas of the pool itself.

The best elevations for establishing wetland plants, either through transplantation or volunteer colonization, are within six inches (plus or minus) of the normal pool.

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.

As a rule of thumb, planting holes should be at least six inches larger than the diameter of the rootball (of balled and burlap stock), and three inches wider for container grown stock. This practice should enable the stock to develop unconfined root systems. Avoid species that require full shade, are susceptible to winterkill, or are prone to wind damage. Extra mulching around the base of the tree or shrub is strongly recommended as a means of conserving moisture and suppressing weeds.

Pond Buffers and Setbacks

A pond buffer should be provided that extends 25 feet 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). An additional setback may be provided to permanent structures.

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.

Woody vegetation may not be planted on nor allowed to grow within 15 feet of the toe of the embankment and 25 feet of the principal spillway structure.

Annual mowing of the pond 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.

3.1.6 Pond Maintenance Criteria

Maintenance Measures

Maintenance responsibility for a pond and its buffer shall be vested with a responsible party by means of a legally binding and enforceable maintenance agreement that is executed as a condition of plan approval or local permitting processes.

The principal spillway shall be equipped with a trash rack that provides access for maintenance.

Sediment removal in the forebay shall occur when 50% of the total forebay capacity has been lost.

Sediments excavated from stormwater ponds that do not receive runoff from designated hotspots are not considered toxic or hazardous material and can be safely disposed by either land application or land filling. Sediment testing may be required prior to sediment disposal when a hotspot land use is present.

Sediment removed from stormwater ponds should be disposed of according to current erosion and sediment control regulations.

Maintenance Access

A maintenance right-of-way or easement shall extend to a pond from a public or private road.

Maintenance access should be at least 12 feet wide; have a maximum slope of no more than 15%; and be appropriately stabilized to withstand maintenance equipment and vehicles.

The maintenance access should extend to the forebay, safety bench, riser, and outlet and be designed to allow vehicles to turn around.

Non-clogging Low Flow Orifice

The low flow orifice shall have a minimum diameter of 3 inches and shall be adequately protected from clogging by an acceptable external trash rack. Two examples of approved external trash racks are provided in Detail No. 1 and 2 of Appendix D.8. *The low flow orifice diameter may be reduced to one inch if an internal orifice is used* (e.g., an over-perforated vertical standpipe that is protected by hardware cloth and a stone filtering jacket). A schematic design of an acceptable internal orifice protection design is provided in Detail No. 3 of Appendix D.8.

The preferred method is a submerged reverse-slope pipe that extends downward from the riser to an inflow point one foot below the normal pool elevation.

Alternative methods are to employ a broad crested rectangular, V-notch, or proportional weir, protected by a half-round corrugated metal pipe (CMP) or similar device that extends at least 12 inches below the normal pool. (See Detail No. 7 of Appendix D.8.)

The use of horizontal perforated pipe protected by geotextile and gravel is not recommended.

Vertical pipes may be used as an alternative if a permanent pool is present.

Riser

The riser shall be located within the embankment for maintenance access, safety and aesthetics.

Access to the riser is to be provided by lockable manhole covers and manhole steps within easy reach of valves and other controls. Riser openings should be fenced with pipe or rebar to prevent trash accumulation.

Pond Drain

Each pond shall have a drain pipe that can completely or partially drain the pond within 24 hours. This requirement is waived for the Lower Eastern Shore where positive drainage is difficult to achieve due to very low relief.

Care should be exercised during pond drawdowns to prevent downstream discharge of sediments or anoxic water and slope instability caused by rapid drawdown.

The approving jurisdiction shall be notified before draining a pond.

Valves

The pond drain shall be equipped with an adjustable valve (typically a handwheel activated knife or gate valve).

The pond drain should be sized one pipe size greater than the calculated design diameter.

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.

To prevent vandalism, the handwheel should be chained to a ringbolt, manhole step or other fixed object.

Safety Features

Fencing of ponds is not generally desirable but may be required by the local review authority. A preferred method is to manage the contours of the pond to eliminate dropoffs and other safety hazards.

Internal side slopes to the pond should not exceed 3:1 (h:v) and should terminate on a safety bench. Both the safety bench and the aquatic bench may be landscaped to prevent access to the pool. The bench requirement may be waived if slopes are 4:1 or gentler.

Riser openings shall not permit unauthorized access. Riser tops that are four feet or greater above the ground shall include railings for safety. Endwalls above pipe outfalls greater than 48 inches in diameter shall be fenced to prevent injury.

Warning signs prohibiting swimming and skating should be posted.

Section 3.2 Stormwater Wetlands

Definition: Practices that create shallow wetland areas to treat urban stormwater and often incorporate small permanent pools and/or extended detention storage to achieve the full WQ_v. Design variants include:

- W-1 shallow wetland (Figure 3.6)
- W-2 ED shallow wetland (Figure 3.7)
- W-3 pond/wetland system (Figure 3.8)
- W-4 pocket wetland (Figure 3.9)

Stormwater wetlands may also provide Cp_v and Q_p storage above the WQ_v storage.

IMPORTANT NOTES:

1) Except for specific minimum contributing drainage area and the use of these practices in coldwater streams (USE III AND IV), all of the pond performance criteria presented in section 3.1 also apply to the design of stormwater wetlands. Additional criteria that govern the geometry and establishment of created wetlands are presented in this section.

2) Any stormwater management BMP that uses an embankment for impounding water is required to follow the latest version of the NRCS-MD 378 Pond Code Standards And Specifications For Small Pond Design (Appendix B.1) and obtain approval from the local SCD or appropriate review authority.

The required sedimentation basin minimum surface area is computed using the following equation:

$$A_s = \frac{Q_o}{W} \times E'$$

where:

A_s = sedimentation basin surface area (ft^2)
Q_o = discharge rate from basin = (WQ_v/24 hr)
W = particle settling velocity (ft/sec)
for I $\leq$ 75%, use 0.0004 ft/sec (particle size = 20 microns)
for I > 75%use 0.0033 ft/sec (particle size = 40 microns) [1]
(I = percent impervious)
E¢ = sediment trapping efficiency constant; for a sediment trapping efficiency (*E)* of 90%, *E¢* = 2.30 [2]

1) Sites with greater than 75% imperviousness have a higher percentage of coarse-grained sediments (Shaver and Baldwin, 1991). Therefore, the target particle size for sedimentation basins may be increased to 40 microns and the surface area reduced.

2) The sediment trapping efficiency constant (*E¢)* may be calculated from the sediment trapping efficiency (*E*) using the following equation: *E¢* = -ln [1-(*E/100*)]

The equation reduces to:

A_{sf} = (0.066) (WQ_v) ft^2 for I $\leq$ 75%
A_{sp} = (0.0081) (WQ_v) ft^2 for I > 75%

where:

A_{sf} = sedimentation basin surface area full
A_{sp} = sedimentation basin surface area partial

Adequate pretreatment for bioretention systems (F-6) is provided when all of the following are provided: (a) 20' grass filter strip below a level spreader or optional sand filter layer, (b) gravel diaphragm and (c) a mulch layer.

3.4.4 Filtering Treatment Criteria

The entire treatment system (including pretreatment) shall temporarily hold at least 75% of the WQ_v prior to filtration.

The filter bed typically has a minimum depth of 18". *Sand filters shall have a minimum filter bed depth of 12".*

Filtering practices typically cannot provide Cp_v or Q_p under most site conditions.

The filter media shall conform to the specifications listed in Table B.3.1 (Appendix B.3).

The filter area for filter designs F-1 to F-5 shall be sized based on the principles of Darcy's Law. A coefficient of permeability (k) shall be used as follows:

Sand:	*3.5 ft/day (City of Austin 1988)*
Peat:	2.0 ft/day (Galli 1990)
Leaf compost:	*8.7 ft/day (Claytor and Schueler, 1996)*
Bioretention Soil:	*0.5 ft/day (Claytor and Schueler, 1996)*

Bioretention systems (F-6) shall consist of the following treatment components: A 2½ to 4 foot deep planting soil bed, a surface mulch layer, and a 12" deep surface ponding area.

The required filter bed area (A_f) is computed using the following equation:

$$A_f = (WQ_v)\ (d_f)\ /\ [\ (k)\ (h_f + d_f)\ (t_f)]$$

where:

A_f	=	Surface area of filter bed (ft^2)
WQ_v	=	water quality volume (ft^3)
d_f	=	filter bed depth (ft)
k	=	coefficient of permeability of filter media (ft/day)
h_f	=	average height of water above filter bed (ft)
t_f	=	design filter bed drain time (days)*

*1.67 days is recommended maximum for sand filters, 2.0 days for bioretention

3.4.5 Filtering Landscaping Criteria

A dense and vigorous vegetative cover shall be established over the contributing drainage area before runoff can be accepted into the facility.

Landscaping is critical to the performance and function of bioretention areas. Therefore, a landscaping plan shall be provided for bioretention areas per the guidance provided in Appendix-A.

Filters F-1, F-4 and F-5 may have a grass cover to aid in pollutant adsorption. The grass should be capable of withstanding frequent periods of inundation and drought (see Appendix A for grass species selection guide).

Planting recommendations for bioretention facilities are as follows:

- Native plant species should be specified over non-native species.
- Vegetation should be selected based on a specified zone of hydric tolerance.
- A selection of trees with an understory of shrubs and herbaceous materials should be provided.

Table 4.3 BMP Selection - Stormwater Treatment Suitability

CODE	BMP List	Re_v Ability	Cp_v Control	Qp Control	Additional Safety Concerns	SPACE	ACCEPT HOTSPOT RUNOFF
P-1	Micropool ED	No[1]	Yes	Yes	No	Yes	Yes[3]
P-2	Wet Pond	No[1]	Yes	Yes	Yes	Varies	Yes[3]
P-3	Wet ED Pond	No[1]	Yes	Yes	Yes	Yes	Yes[3]
P-4	Multiple Pond	No[1]	Yes	Yes	Yes	No	Yes[3]
P-5	Pocket Pond	No[1]	Yes	Yes	Varies	Yes	Yes[3]
W-1	Shallow Wetland	Varies[2]	Yes	Yes	No	No	Yes[3]
W-2	ED Wetland	Varies[2]	Yes	Yes	Varies	Varies	Yes[3]
W-3	Pond/Wetland	Varies[2]	Yes	Yes	Yes	No	Yes[3]
W-4	Pocket Wetland	No	Varies	Varies	No	Varies	Yes[3]
I-1	Infiltration Trench	Yes	Varies	Varies	No	Yes	No[3]
I-2	Infiltration Basin	Yes	Varies	Varies	No	Varies	No[3]
F-1	Surface Sand Filter	Varies[2]	Varies	Varies	No	Yes	Yes[4]
F-2	Underground SF	No	No	No	Varies	Yes	Yes
F-3	Perimeter SF	No	No	No	No	Yes	Yes
F-4	Organic Filter	Varies[2]	Varies	Varies	No	Yes	Yes[4]
F-5	Pocket Sand Filter	Varies[2]	Varies	Varies	No	Yes	Yes[4]
F-6	Bioretention	Yes	Varies	Varies	No	Varies	Yes[4]
O-1	Dry Swale	Yes	No	No	No	Varies	Yes[4]
O-2	Wet Swale	No	No	No	No	Varies	No

1 Structures that require impermeable liners or that intercept groundwater may not be used for groundwater recharge.

2 Re_v may be provided by exfiltration (see Chapter 3.4).

3 Not allowed unless pretreatment to remove hydrocarbons, trace metals, and toxicants is provided.

4 Yes, but only if bottom of facility is lined with impermeable filter fabric that prevents leachate infiltration.

Section 4.4 Physical Feasibility Factors

At this point, the designer has narrowed the BMP list to a manageable size and can evaluate the remaining options given the physical conditions at a site. This table cross-references testing protocols needed to confirm physical conditions at the site. The six primary factors are:

Soils. The key evaluation factors are based on an initial investigation of the USDA hydrologic soils groups at the site. Note that more detailed geotechnical tests are usually required for infiltration feasibility and during design to confirm permeability and other factors (see Appendix D.1).

Water Table. This column indicates the minimum depth to the seasonally high water table from the bottom or floor of a BMP.

Drainage Area. This column indicates the recommended minimum or maximum drainage area that is considered suitable for the practice. If the drainage area present at a site is slightly greater than the maximum allowable drainage area for a practice, some leeway is permitted or more than one practice can be installed. The minimum drainage areas indicated for ponds and wetlands are flexible depending on water availability (baseflow or groundwater) or the mechanisms employed to prevent clogging.

Slope Restriction. This column evaluates the effect of slope on the practice. Specifically, the slope restrictions refer to how flat the area where the practice may be.

Head. This column provides an estimate of the elevation difference needed at a site (from the inflow to the outflow) to allow for gravity operation within the practice.

Ultra-Urban Sites. This column identifies BMPs that work well in the ultra-urban environment, where space is limited and original soils have been disturbed. These BMPs are frequently used at redevelopment sites.

Table 4.5 BMP Selection - Community and Environmental Factors

CODE	BMP LIST	EASE OF MAINTENANCE	COMMUNITY ACCEPTANCE	COST (Relative To Drainage Area)	HABITAT QUALITY	OTHER FACTORS
P-1	Micropool ED	Medium	Medium	Low	Medium	Trash/debris
P-2	Wet Pond	Easy	High	Low	High	
P-3	Wet ED Pond	Easy	High	Low	High	
P-4	Multiple Pond	Easy	High	Medium	High	
P-5	Pocket Pond	Difficult	Medium	Low	Low	Drawdowns
W-1	Shallow Wetland	Medium	High	Medium	High	
W-2	ED Wetland	Medium	Medium	Medium	High	Limit ED depth
W-3	Pond/Wetland	Difficult	High	Medium	High	
W-4	Pocket Wetland	Medium	Low	Low	Medium	Drawdowns
I-1	Infiltration Trench	Difficult	High	Medium	Low	Avoid large stone
I-2	Infiltration Basin	Medium	Low	Medium	Low	Frequent pooling
F-1	Surface SF	Medium	Medium	High	Low	
F-2	Underground SF	Difficult	High	High	Low	Underground ∴Out of sight
F-3	Perimeter SF	Difficult	High	High	Low	Traffic Bearing
F-4	Organic Filter	Medium	High	High	Low	Filter Media Replacement
F-5	Pocket SF	Medium	Medium	Medium	Low	
F-6	Bioretention	Medium	Medium	Medium	Low	Landscaping
O-1	Dry Swale	Easy	High	Medium	Low	
O-2	Wet Swale	Easy	High	Low	Low	Mosquitoes Possible

Section 4.6 Checklist: Location and Permitting Factors

In the last step, a designer assesses the physical and environmental features at the site to determine the optimal location for the selected BMP or group of BMPs. The checklist below provides a condensed summary of current BMP restrictions as they relate to common site features that may be regulated under local, State or federal law. These restrictions fall into one of three general categories:

1. Locating a BMP within an area that is expressly **prohibited** by law.

2. Locating a BMP within an area that is **strongly discouraged** and is only allowed on a case by case basis. Local, State and/or federal permits shall be obtained and the applicant will need to supply additional documentation to justify locating the BMP within the regulated area.

3. BMPs must be **setback** a fixed distance from the site feature.

This checklist is only intended as a general guide to location and permitting requirements as they relate to siting stormwater BMPs. Consultation with the appropriate regulatory agency is the best strategy.

The symbol "✔" denotes when an MDE Nontidal Wetland And Waterways Permit shall be obtained.

5.0 Stormwater Credits

In Maryland, there are many programs at both the State and local level that seek to minimize the impact of land development. Critical Areas, forest conservation, and local stream buffer requirements are designed to reduce nonpoint source pollution. Non-structural practices can play a significant role in reducing water quality impacts and are increasingly recognized as a critical feature of every stormwater BMP plan, particularly with respect to site design. In most cases, non-structural practices must be combined with structural practices to meet stormwater requirements. The key benefit of non-structural practices is that they can reduce the generation of stormwater from the site; thereby reducing the size and cost of stormwater storage. In addition, they can provide partial removal of many pollutants. Non-structural practices have been classified into six broad groups and are designed to mesh with existing state and local programs (e.g., forest conservation, stream buffers etc.). To promote greater use, a series of six stormwater credits are provided for designers that use these site planning techniques.

Credit 1.	Natural Area Conservation
Credit 2.	Disconnection of Rooftop Runoff
Credit 3.	Disconnection of Non Rooftop Runoff
Credit 4.	Sheet Flow to Buffers
Credit 5.	Open Channel Use
Credit 6.	Environmentally Sensitive Development

This chapter describes each of the credits for the six groups of non-structural practices, specifies minimum criteria to be eligible for the credit, and provides an example of how the credit is calculated. Designers should check with the appropriate approval authority to ensure that the credit is applicable to their jurisdiction. Clearly both of the site designs used to illustrate the credits could be more creative to provide more non-structural opportunities.

In general, the stormwater sizing criteria provide a strong incentive to reduce impervious cover at development sites (e.g., Re_v, WQ_v, Cp_v or Qp and Qf). Storage requirements for all five stormwater sizing criteria are directly related to impervious cover. Thus, significant reductions in impervious cover result in smaller required storage volumes and, consequently, lower BMP construction costs.

These and other site design techniques can help to reduce impervious cover, and consequently, the stormwater treatment volume needed at a site. The techniques presented in this chapter are considered options to be used by the designer to help reduce the need for stormwater BMP storage capacity. Due to local safety codes, soil conditions, and topography, some of these site design features will be restricted. Designers are encouraged to consult with the appropriate approval authority to determine restrictions on non-structural strategies.

NOTE: In this chapter, *italics* indicate mandatory performance criteria, whereas suggested design criteria are shown in normal typeface.

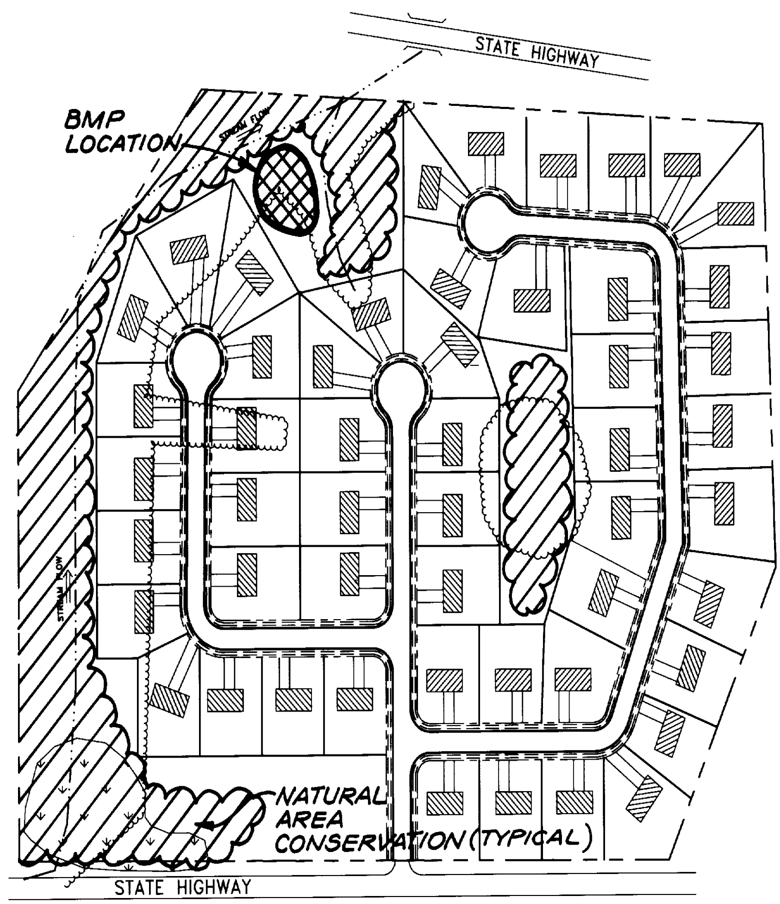
These credits are an integral part of a project's overall stormwater management plan and BMP storage volume calculation. Therefore, use of these credits shall be documented at the initial (concept) design stage, documented with submission of final grading plans, and verified with "as-built" certifications. If a planned credit is not implemented, then BMP volumes shall be increased appropriately to meet Re_v, WQ_v, Cp_v, and Q_p where applicable.

Table 5.1 Summary of Stormwater Credits

Stormwater Credit	WQ_v	Re_v	Cp_v or Q_p
Natural Area Conservation	Reduce Site Area	No credit. Use as receiving area w/Percent Area Method.	Forest/meadow CN for natural areas
Disconnection of Rooftop Runoff	Reduced R_v	No credit. Use with Percent Area Method.	Longer t_c (increased flow path). CN credit.
Disconnection of Non-Rooftop Runoff	Reduced R_v	No credit. Use with Percent Area Method.	Longer t_c (increased flow path) CN credit
Sheet Flow to Buffers	Subtract contributing site area to BMP	Reduced Re_v	CN credit
Open Channel Use	May meet WQ_v	Meets Re_v	Longer t_c (increased flow path) No CN credit
Environmentally Sensitive Development	Meets WQ_v	Meets Re_v	No CN credit t_c may increase

Example of Calculating Natural Area Credit

Site Data - 51 Single Family Lots
Area = 38 ac.
Conservation Area = 7.0 ac
Impervious Area = 13.8 ac
R_v = .38, P = 0.9”
Post dev. CN = 78
Original WQ_v = 1.08 ac-ft.
Original Re_v = .25 ac-ft.
Original Cp_v = 1.65 ac-ft.
Original Qp_{10} = 2.83 ac-ft.



Computation of Stormwater Credits
WQ_v = [(P)(R_v)(A)]/12
= [(0.9)(.38)(38.0 - 7.0 ac.)]/12
= 0.89 ac-ft

Re_v = Same as original
(However, area draining to Natural Area may used with the Percent Area Method)

Cp_v and Q_{p10} (total site): CN reduced from 78 to 75

Section 5.2 Disconnection of Rooftop Runoff Credit

Disconnection of Rooftop Runoff Credit

A credit is given when rooftop runoff is disconnected and then directed to a pervious area where it can either infiltrate into the soil or filter over it. The credit is typically obtained by grading the site to promote overland filtering or by providing bioretention areas on single family residential lots.

If a rooftop is adequately disconnected, the disconnected impervious area may be deducted from total impervious cover (therefore reducing WQ_v). In addition, disconnected rooftops can be used to meet the Re_v requirement as a non-structural practice using the percent area method (see Chapter 2).

Post development CN's for disconnected rooftop areas used to compute Cp_v and Qp can be assumed to be woods in good condition.

Criteria for Disconnection of Rooftop Runoff Credit

The credit is subject to the following restrictions:

- *Rooftop cannot be within a designated hotspot,*
- *Disconnection shall cause no basement seepage,*
- *The contributing area of rooftop to each disconnected discharge shall be 500 square feet or less,*
- *The length of the "disconnection" shall be 75' or greater, or compensated using Table 5.2,*
- *Dry wells, french drains, rain gardens, or other similar storage devices may be utilized to compensate for areas with disconnection lengths less than 75 feet. (See Table 5.2 and Figure 5.1, dry wells are prohibited in "D" soils),*
- *In residential development applications, disconnections will only be credited for lot sizes greater than 6000 sq. ft.,*
- *The entire vegetative "disconnection" shall be on an average slope of 5% or less,*
- *The disconnection must drain continuously through a vegetated channel, swale, or through a filter strip to the property line or BMP,*
- *Downspouts must be at least 10 feet away from the nearest impervious surface to discourage "re-connections", and*
- *For those rooftops draining directly to a buffer, only the rooftop disconnection credit or the buffer credit may be used, not both.*

Section 5.3 Disconnection of Non Rooftop Runoff Credit

Disconnection of Non Rooftop Runoff Credit

Credit is given for practices that disconnect surface impervious cover runoff by directing it to pervious areas where it is either infiltrated into the soil or filtered (by overland flow). This credit can be obtained by grading the site to promote overland vegetative filtering or providing bioretention areas on single family residential lots.

These "disconnected" areas can be subtracted from the impervious area when computing WQ_v. In addition, disconnected surface impervious cover can be used to meet the Re_v requirement as a non-structural practice using the percent area method (See Chapter 2).

Criteria for Disconnection of Non Rooftop Runoff Credit

The credit is subject to the following restrictions:

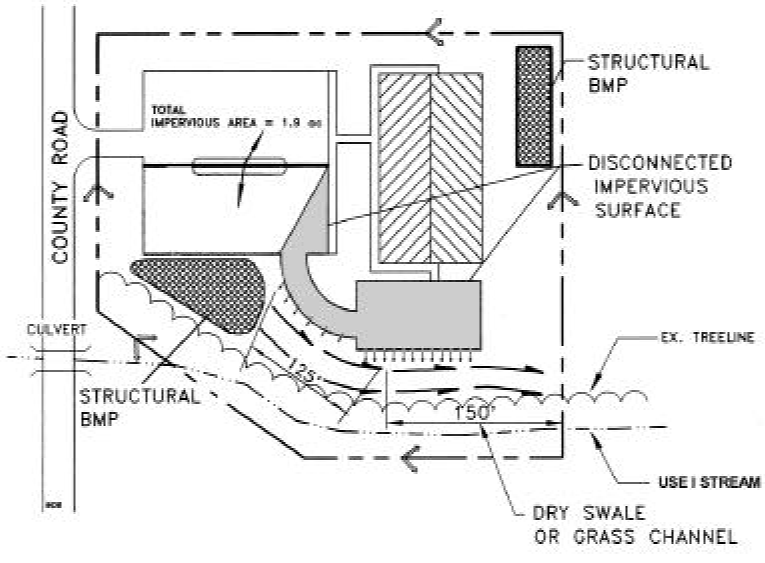
- *Runoff cannot come from a designated hotspot,*
- *The maximum contributing impervious flow path length shall be 75 feet,*
- *The disconnection shall drain continuously through a vegetated channel, swale, or filter strip to the property line or BMP,*
- *The length of the "disconnection" must be equal to or greater than the contributing length,*
- *The entire vegetative "disconnection" shall be on an average slope of 5% or less,*
- *The surface impervious area to any one discharge location cannot exceed 1,000 ft^2.*
- *Disconnections are encouraged on relatively permeable soils (HSG's A and B),*
- *If the site cannot meet the required disconnect length, a spreading device, such as a french drain, rain garden, gravel trench or other storage device may be needed for compensation, and*
- *For those areas draining directly to a buffer, only the non rooftop disconnection credit or the stream buffer credit can be used, not both.*

Example of Calculating the Non Rooftop Disconnection Credit

Site Data -Community Center
Area = 3.0 ac
Original Impervious Area = 1.9 ac. = 63.3%
Original R_v = .62
Post dev. CN = 83
B Soils, S = 0.26
Original WQ_v = 6752 ft^3
Original Re_v = 1688 ft^3
Original Cp_v = N/A
Original Qp_2 = 10,630 ft^3

0.33 ac of surface imperviousness disconnected

Net impervious area reduction
1.9 - 0.33 = 1.57 ac.



Computation of Stormwater Credit:

New R_v = $0.05 + .009\ (1.57\ ac/3.0\ ac) = .52$
$\therefore WQ_v = [(1.0)(0.52)(3.0\ ac)]/12 = 0.13$ ac-ft (5662.8 cf)

Required Re_v (Percent area method)
$Re_v = (S)(A_i) = (0.26)(1.9\ ac.) = 0.49$ acres
Re_v treated by disconnection = 0.33 acres
Re_v remaining for treatment = 0.16 acres non structurally or 551.2 cf structurally

Cp_v and Q_p Post developed CN may be reduced

Section 5.4 Sheetflow to Buffer Credit

Sheetflow to Buffer Credit

This credit is given when stormwater runoff is effectively treated by a natural buffer to a stream or forested area. Effective treatment is achieved when pervious and impervious area runoff is discharged to a grass or forested buffer through overland flow. The use of a filter strip is also recommended to treat overland flow in the green space of a development site. The credits include:

1. The area draining by sheet flow to a buffer is subtracted from the total site area in the WQ_v calculation.
2. The area draining to the buffer contributes to the recharge requirement, Re_v.
3. A *wooded* CN can be used for the contributing area if it drains to a forested buffer.

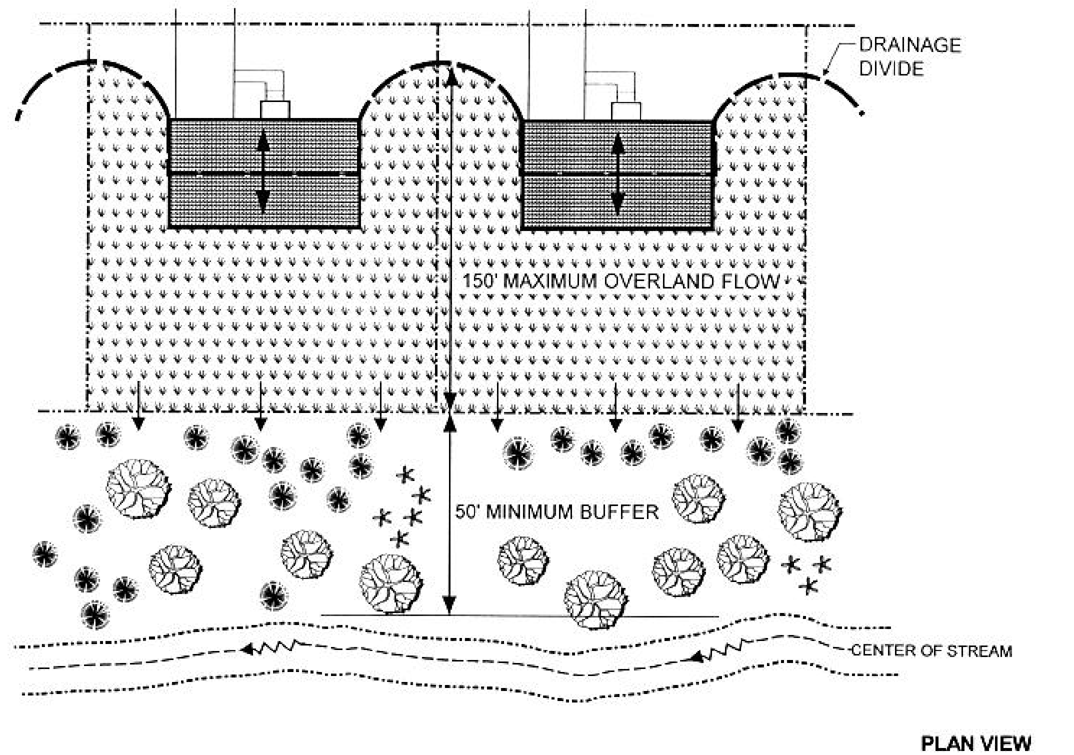
Criteria for Sheetflow to Buffer Credit

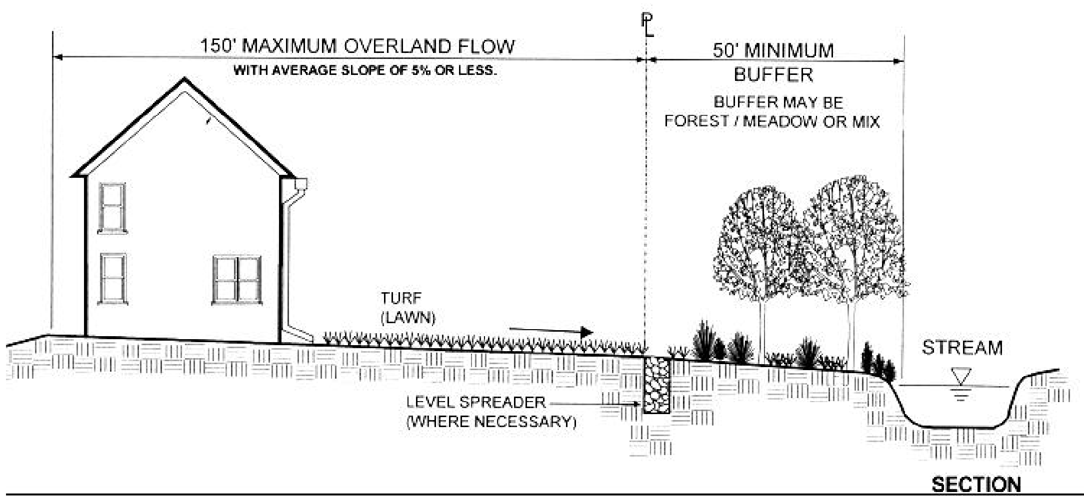
The credit is subject to the following conditions:

- *The minimum buffer width shall be 50 feet as measured from bankfull elevation or centerline of the buffer,*
- *The maximum contributing length shall be 150 feet for pervious surfaces and 75 feet for impervious surfaces,*
- *Runoff shall enter the buffer as sheet flow. Either the average contributing overland slope shall be 5.0% or less, or a level spreading device shall be used where sheet flow can no longer be maintained (see Detail No. 9 in Appendix D.8),*
- *Not applicable if rooftop or non rooftop disconnection is already provided (see Credits 2 & 3),*
- *Buffers shall remain unmanaged other than routine debris removal, and*
- *Shall be protected by an acceptable conservation easement or other enforceable instrument that ensures perpetual protection of the proposed area. The easement must clearly specify how the natural area vegetation shall be managed and boundaries will be marked* [Note: managed turf (e.g., playgrounds, regularly maintained open areas) is not an acceptable form of vegetation management].

Figure 5.2 illustrates how a buffer or filter strip can be used to treat stormwater from adjacent pervious and impervious areas.

Figure 5.2 Example of Sheetflow to Buffer Credit





B.2.A Infiltration Trench General Notes and Specifications

An infiltration trench may not receive run-off until the entire contributing drainage area to the infiltration trench has received final stabilization.

1. Heavy equipment and traffic shall be restricted from traveling over the proposed location of the infiltration trench to minimize compaction of the soil.

2. Excavate the infiltration trench to the design dimensions. Excavated materials shall be placed away from the trench sides to enhance trench wall stability. Large tree roots must be trimmed flush with the trench sides in order to prevent fabric puncturing or tearing of the filter fabric during subsequent installation procedures. The side walls of the trench shall be roughened where sheared and sealed by heavy equipment.

3. A Class "C" geotextile or better (see Section 24.0, Material Specifications, 1994 Standards and Specifications for Soil Erosion and Sediment Control, MDE, 1994) shall interface between the trench side walls and between the stone reservoir and gravel filter layers. A partial list of non-woven filter fabrics that meet the Class "C" criteria follows. Any alternative filter fabric must be approved by the plan approval authority.

Amoco 4552	Carthage FX-80S
GEOLON N70	Mirafi 180-N
WEBTEC N07	

   The width of the geotextile must include sufficient material to conform to trench perimeter irregularities and for a 6-inch minimum top overlap. The filter fabric shall be tucked under the sand layer on the bottom of the infiltration trench for a distance of 6 to 12 inches. Stones or other anchoring objects should be placed on the fabric at the edge of the trench to keep the trench open during windy periods. When overlaps are required between rolls, the uphill roll should lap a minimum of 2 feet over the downhill roll in order to provide a shingled effect.

4. If a 6 inch sand filter layer is placed on the bottom of the infiltration trench, the sand for the infiltration trench shall be washed and meet AASHTO-M-43, Size No. 9 or No. 10. Any alternative sand gradation must be approved by the plan approval authority.

5. The stone aggregate should be placed in a maximum loose lift thickness of 12 inches. The gravel (rounded "bank run" gravel is preferred) for the infiltration trench shall be washed and meet one of the following AASHTO-M-43, Size No. 2 or No. 3.

6. Following the stone aggregate placement, the filter fabric shall be folded over the stone aggregate to form a 6-inch minimum longitudinal lap. The desired fill soil or stone

aggregate shall be placed over the lap at sufficient intervals to maintain the lap during subsequent backfilling.

7. Care shall be exercised to prevent natural or fill soils from intermixing with the stone aggregate. All contaminated stone aggregate shall be removed and replaced with uncontaminated stone aggregate.

8. Voids may occur between the fabric and the excavation sides shall be avoided. Removing boulders or other obstacles from the trench walls is one source of such voids. Therefore, natural soils should be placed in these voids at the most convenient time during construction to ensure fabric conformity to the excavation sides.

9. Vertically excavated walls may be difficult to maintain in areas where soil moisture is high or where soft cohesive or cohesionless soils are dominant. These conditions may require laying back of the side slopes to maintain stability.

10. PVC distribution pipes shall be Schedule 40 and meet ASTM-D-1785. All fittings shall meet ASTM-D-2729. Perforations shall be 3/8 inch in diameter. A perforated pipe shall be provided only within the infiltration trench and shall terminate 1 foot short of the infiltration trench wall. The end of the PVC pipe shall be capped. **Note:** PVC pipe with a wall thickness classification of SDR-35 meeting ASTM-D-3034 is an acceptable substitute for the Schedule 40 pipe.

11. The observation well is to consist of 6-inch diameter perforated PVC Schedule 40 pipe (M 278 OR F758, Type PS 28) with a cap set 6 inches above ground level and is to be located near the longitudinal center of the infiltration trench. The pipe shall have a plastic collar with ribs to prevent rotation when removing the cap. The screw top lid shall be a cleanout with a locking mechanism or special bolt to discourage vandalism. The depth to the invert shall be marked on the lid. The pipe shall be placed vertically within the gravel portion of the infiltration trench and a cap provided at the bottom of the pipe. The bottom of the cap shall rest on the infiltration trench bottom.

12. Corrugated metal distribution pipes shall conform to AASHTO-M-36, and shall be aluminized in accordance with AASHTO-M-274. Aluminized pipe in contact with concrete shall be coated with an inert compound capable of preventing the deleterious effect of the aluminum on the concrete. Perforated distribution pipes shall conform to AASHTO-M-36, Class 2 and shall be provided only within the infiltration trench and shall terminate 1 foot short of the infiltration trench wall. An aluminized metal plate shall be welded to the end of the pipe.

13. If a distribution structure with a wet well is used, a 4-inch drain pipe shall be provided at opposite ends of the infiltration trench distribution structure. Two (2) cubic feet of porous backfill meeting AASHTO-M-43, Size No. 57 shall be provided at each drain.

14. If a distribution structure is used, the manhole cover shall be bolted to the frame.

B.2.B Infiltration Basins Notes and Specifications

An infiltration basin may not receive run-off until the entire contributing drainage area to the basin has received final stabilization.

1. The sequence of various phases of basin construction shall be coordinated with the overall project construction schedule. A program should schedule rough excavation of the basin with the rough grading phase of the project to permit use of the material as fill in earthwork areas. The partially excavated basin, however, **cannot** serve as a sedimentation basin.

   Specifications for basin construction should state: (1) the earliest point in progress when storm drainage may be directed to the basin, and (2) the means by which this delay in use is to be accomplished. Due to the wide variety of conditions encountered among projects, each should be separately evaluated in order to postpone use as long as is reasonably possible.

2. Initial basin excavation should be carried to within 2 feet of the final elevation of the basin floor. Final excavation to the finished grade should be deferred until all disturbed areas on the watershed have been stabilized or protected. The final phase excavation should remove all accumulated sediment. Relatively light tracked equipment is recommended for this operation to avoid compaction of the basin floor. After the final grading is completed, the basin should provide a well-aerated, highly porous surface texture.

3. Infiltration basins may be lined with a 6- to 12-inch layer of filter material such as coarse sand (AASHTO-M-43, Sizes 9 or 10) to help prevent the buildup of impervious deposits on the soil surface. The filter layer can be replaced or cleaned when it becomes clogged. When a 6-inch layer of coarse organic material is specified for discing (such as hulls, leaves, stems, etc.) or spading into the basin floor to increase the permeability of the soils, the basin floor should be soaked or inundated for a brief period, then allowed to dry subsequent to this operation. This induces the organic material to decay rapidly, loosening the upper soil layer.

4. Establishing dense vegetation on the basin side slopes and floor is recommended. A dense vegetative stand will not only prevent erosion and sloughing, but will also provide a natural

means of maintaining relatively high infiltration rates. Erosion protection of inflow points to the basin shall also be provided.

5. Selection of suitable vegetative materials for the side slope and all other areas to be stabilized with vegetation and application of soil amendments (e.g., lime, fertilizer, etc.) shall be done in accordance with the NRCS Standards and Specifications for Critical Area Planting (MD-342) or the 1994 Maryland Standards and Specifications for Soil Erosion and Sediment Control.

6. Grasses of the fescue family are recommended for seeding primarily due to their adaptability to dry sandy soils, drought resistance, hardiness, and ability to withstand brief inundations. The use of fescues will also permit long intervals between mowings. This is important due to the relatively steep slopes which make mowing difficult. Mowing twice a year, once in June and again in September, is generally satisfactory. Refertilization with 10-6-4 ratio fertilizer at a rate of 500 lb per acre (11 lb per 1000 sq ft) may be required the second year after seeding.

Appendix B.3

Construction Specifications for Sand Filters, Bioretention and Open Channels

rubber tires with large lugs, or high pressure tires will cause excessive compaction resulting in reduced infiltration rates and is not acceptable. Compaction will significantly contribute to design failure.

Compaction can be alleviated at the base of the bioretention facility by using a primary tilling operation such as a chisel plow, ripper, or subsoiler. These tilling operations are to refracture the soil profile through the 12 inch compaction zone. Substitute methods must be approved by the engineer. Rototillers typically do not till deep enough to reduce the effects of compaction from heavy equipment.

Rototill 2 to 3 inches of sand into the base of the bioretention facility before backfilling the optional sand layer. Pump any ponded water before preparing (rototilling) base.

When backfilling the topsoil over the sand layer, first place 3 to 4 inches of topsoil over the sand, then rototill the sand/topsoil to create a gradation zone. Backfill the remainder of the topsoil to final grade.

When backfilling the bioretention facility, place soil in lifts 12” to 18”. Do not use heavy equipment within the bioretention basin. Heavy equipment can be used around the perimeter of the basin to supply soils and sand. Grade bioretention materials with light equipment such as a compact loader or a dozer/loader with marsh tracks.

4. Plant Material

Recommended plant material for bioretention areas can be found in Appendix A, Section A.2.3.

5. Plant Installation

Mulch should be placed to a uniform thickness of 2” to 3”. Shredded hardwood mulch is the only accepted mulch. Pine mulch and wood chips will float and move to the perimeter of the bioretention area during a storm event and are not acceptable. Shredded mulch must be well aged (6 to 12 months) for acceptance.

Root stock of the plant material shall be kept moist during transport and on-site storage. The plant root ball should be planted so 1/8th of the ball is above final grade surface. The diameter of the planting pit shall be at least six inches larger than the diameter of the planting ball. Set and maintain the plant straight during the entire planting process. Thoroughly water ground bed cover after installation.

Trees shall be braced using 2” by 2” stakes only as necessary and for the first growing season only. Stakes are to be equally spaced on the outside of the tree ball.

Grasses and legume seed should be drilled into the soil to a depth of at least one inch. Grass and legume plugs shall be planted following the non-grass ground cover planting specifications.

The topsoil specifications provide enough organic material to adequately supply nutrients from natural cycling. The primary function of the bioretention structure is to improve water quality. Adding fertilizers defeats, or at a minimum, impedes this goal. Only add fertilizer if wood chips or mulch are used to amend the soil. Rototill urea fertilizer at a rate of 2 pounds per 1000 square feet.

6. Underdrains

Underdrains are to be placed on a 3'-0" wide section of filter cloth. Pipe is placed next, followed by the gravel bedding. The ends of underdrain pipes not terminating in an observation well shall be capped.

The main collector pipe for underdrain systems shall be constructed at a minimum slope of 0.5%. Observation wells and/or clean-out pipes must be provided (one minimum per every 1000 square feet of surface area).

7. Miscellaneous

The bioretention facility may not be constructed until all contributing drainage area has been stabilized.

Table B.3.2 Materials Specifications for Bioretention

Material	Specification	Size	Notes
Plantings	see Appendix A, Table A.4	n/a	plantings are site-specific
planting soil [2.5' to 4' deep]	sand 35 - 60% silt 30 - 55% clay 10 - 25%	n/a	USDA soil types loamy sand, sandy loam or loam
mulch	shredded hardwood		aged 6 months, minimum
pea gravel diaphragm and curtain drain	pea gravel: ASTM-D-448 ornamental stone: washed cobbles	pea gravel: No. 6 stone: 2" to 5"	
geotextile	Class "C" - apparent opening size (ASTM-D-4751), grab tensile strength (ASTM-D-4632), puncture resistance (ASTM-D-4833)	n/a	for use as necessary beneath underdrains only
underdrain gravel	AASHTO M-43	0.375" to 0.75"	
underdrain piping	F 758, Type PS 28 or AASHTO M-278	4" to 6" rigid schedule 40 PVC or SDR35	3/8" perf. @ 6" on center, 4 holes per row; minimum of 3" of gravel over pipes; not necessary underneath pipes
poured in place concrete (if required)	MSHA Mix No. 3; f'_c = 3500 psi @ 28 days, normal weight, air-entrained; reinforcing to meet ASTM-615-60	n/a	on-site testing of poured-in-place concrete required: 28 day strength and slump test; all concrete design (cast-in-place or pre-cast) *not using previously approved State or local standards* requires design drawings sealed and approved by a professional structural engineer licensed in the State of Maryland - design to include meeting ACI Code 350.R/89; vertical loading [H-10 or H-20]; allowable horizontal loading (based on soil pressures); and analysis of potential cracking
sand [1' deep]	AASHTO-M-6 or ASTM-C-33	0.02" to 0.04"	Sand substitutions such as Diabase and Graystone #10 are not acceptable. No calcium carbonated or dolomitic sand substitutions are acceptable. No "rock dust" can be used for sand.

B.3.C Specifications for Open Channels and Filter Strips

1. Material Specifications

The recommended construction materials for open channels and filter strips are detailed in Table B.3.3.

2. Dry Swales

Permeable soil mixture (20” to 30” deep) should meet the bioretention specifications.

Check dams, if required, shall be placed as specified.

System to have 6” of freeboard, minimum above 2 year water surface elevation.

Side slopes to be 3:1 maximum; (4:1 or flatter is preferred).

No gravel or perforated pipe is to be placed under driveways.

Bottom of facility to be above the seasonally high water table per Table 2 of Appendix D.1.

Seed with flood/drought resistant grasses; see Appendix A, Section 2.4.

Longitudinal slope to be 4%, maximum.

Bottom width to be 8’ maximum to avoid braiding; larger widths may be used if proper berming is supplied. Width to be 2’ minimum.

3. Wet Swales

Follow above information for dry swales, with the following exceptions: the seasonally high water table may inundate the swale; but not above the design bottom of the channel [NOTE: if the water table is stable within the channel, the WQ_v storage may start at this point – see Figure 3.19]

Excavate into undisturbed soils; do not use an underdrain system.

4. Filter Strips

Construct pea gravel diaphragms 12” wide, minimum, and 24” deep minimum.

Pervious berms to be a sand/gravel mix [sand (35-60%), silt (30-55%), and gravel (10-25%)]. Berms to have overflow weirs with 6 inch minimum head.

Slope range to be 2% minimum to 6% maximum.

5. Plant Selection

Recommended grass species for use in establishing permanent ground cover are provided in Section 2.4 of Appendix A.

Table B.3.3 Open Channel Systems and Filter Strip Materials Specifications

Material	Specification	Size	Notes
dry swale soil	USCS; ML, SM, SC	n/a	soil with a higher percent organic content is preferred
dry swale sand	ASTM C-33 fine aggregate concrete sand	0.02" to 0.04"	
check dam (pressure treated)	AWPA Standard C6	6" by 6" or 8" by 8"	do not coat with creosote; embed at least 3' into side slopes
check dam (natural wood)	Black Locust, Red Mulberry, Cedars, Catalpa, White Oak, Chestnut Oak, Black Walnut	6" to 12" diameter; notch as necessary	do not use the following, as these species have a predisposition towards rot: Ash, Beech, Birch, Elm, Hackberry, hemlock, Hickories, Maples, Red and Black Oak, Pines, Poplar, Spruce, Sweetgum, Willow
filter strip sand/gravel pervious berm	sand: per dry swale sand gravel; AASHTO M-43	sand: 0.02" to 0.04" gravel: ½ " to 1"	mix with approximately 25% loam soil to support grass cover crop; sand (35-60%), silt (30-55%), and gravel (10-25%) see Bioretention planting soil notes for more detail.
pea gravel diaphragm and curtain drain	ASTM D 448	varies (No. 6) or (1/8" to 3/8")	use clean bank-run gravel
underdrain gravel	AASHTO M-43	0.25" to 0.75"	
underdrain	F 758 Type PS 28 or AASHTO M-278	4" to 6" rigid schedule 40 PVC or SDR35	3/8" perf. @ 6" on center, 4 holes per row; minimum of 3" of gravel over pipes; not necessary underneath pipes
geotextile	Class "C" - apparent opening size (ASTM-D-4751), grab tensile strength (ASTM-D-4632), puncture resistance (ASTM-D-4833)	n/a	
rip rap	per county criteria; if none given, use MSHA Standards and Specs Section 905	size per county DOT requirements based on 10-year design flows	

Design Example 1 – Shallow Wetland (W-1)

The following example demonstrates the process for the design of a shallow wetland (W-1) BMP.

Site Specific Data

Clevenger Community Center is a recreational center located in Charles County, Maryland. The site area and drainage area to the proposed stormwater management facility is 5.3 acres. The project consists of constructing the community center and parking for a total impervious area of 1.94 acres. Existing ground at the outlet of the facility is 44.5' above mean sea level (MSL). Soil borings indicate that the seasonally high water table is at elevation 41'. The underlying soils are loams. TR-55 calculations for the existing and developed hydrologic conditions are shown in Figures C.1.2 and C.1.3.

Confirm Design Criteria

The site is within the Eastern Rainfall Zone and located on the Western Shore of the Chesapeake Bay (see Volume I, Chapter 2, Figures 2.1 and 2.4). Additionally, the site is located within a USE I watershed. Therefore, the following criteria apply:

1. WQ_v treatment is required. In the Eastern Rainfall Zone, P = 1".
2. Re_v treatment is required.
3. Cp_v treatment is required.
4. Q_{p10} may be required by the local jurisdiction. For this example, Q_{p10} will be required.
5. Q_f may be required by the local jurisdiction. For this example, Q_f will not be required. However, safe conveyance of the 100-year design storm is required through the proposed stormwater management facility.

Preliminary Design

Step 1. Compute WQ_v

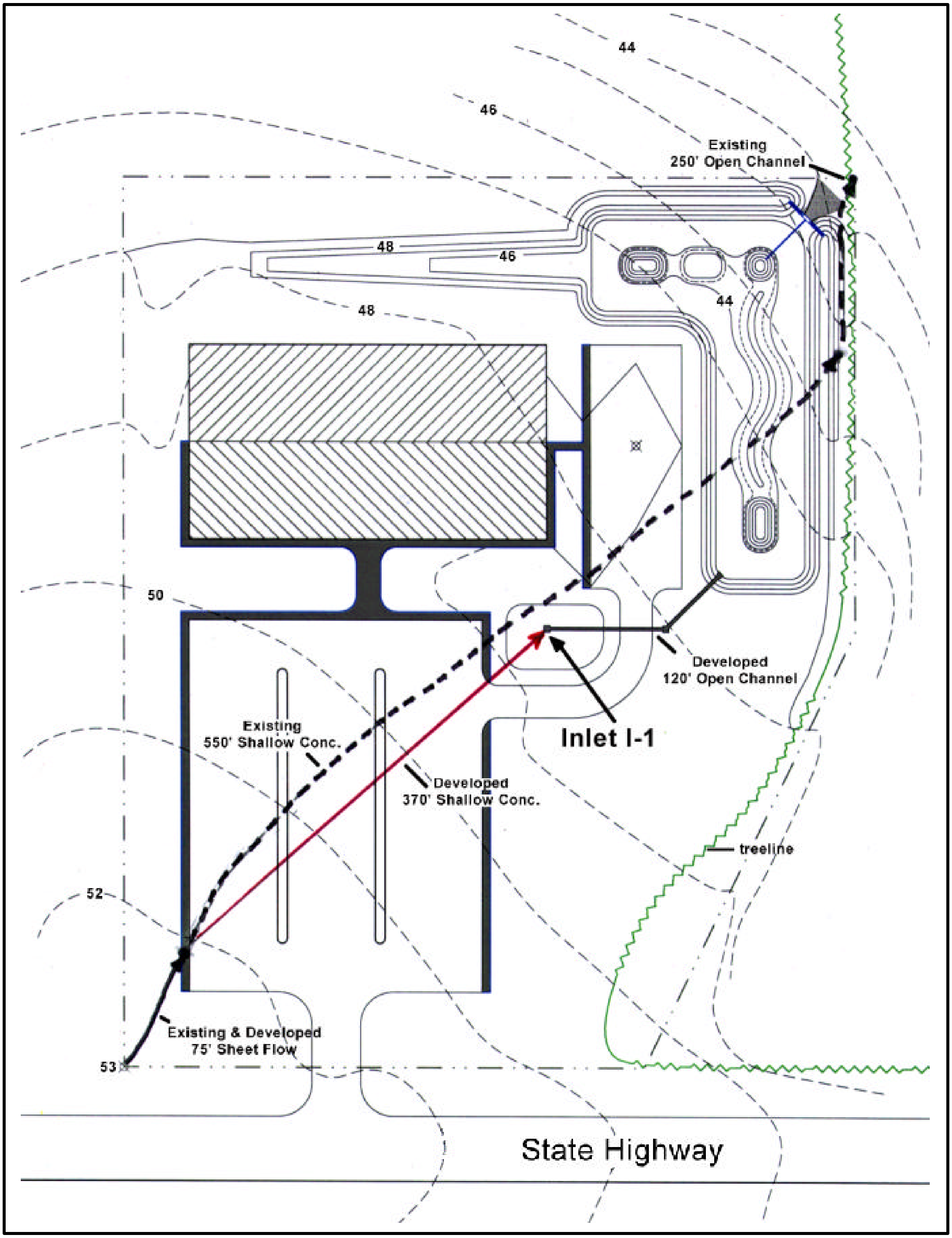
Step 1a. Compute Volumetric Runoff Coefficient (R_v)

$$R_v = 0.05 + (0.009)(I);\ I = 1.94 \text{ acres} / 5.3 \text{ acres} = 0.366 \text{ or } 36.6\%$$
$$= 0.05 + (0.009)(36.6) = 0.379$$

Step 1b. Compute WQ_v

$$WQ_v = [(P)(R_v)(A)]/12$$
$$= [(1")(0.379)(5.3 \text{ ac})]/12$$
$$= \underline{0.167 \text{ ac-ft}}\ (7{,}292 \text{ cf.})$$

Figure C.1.1 Clevenger Community Center Site Plan



Appendix

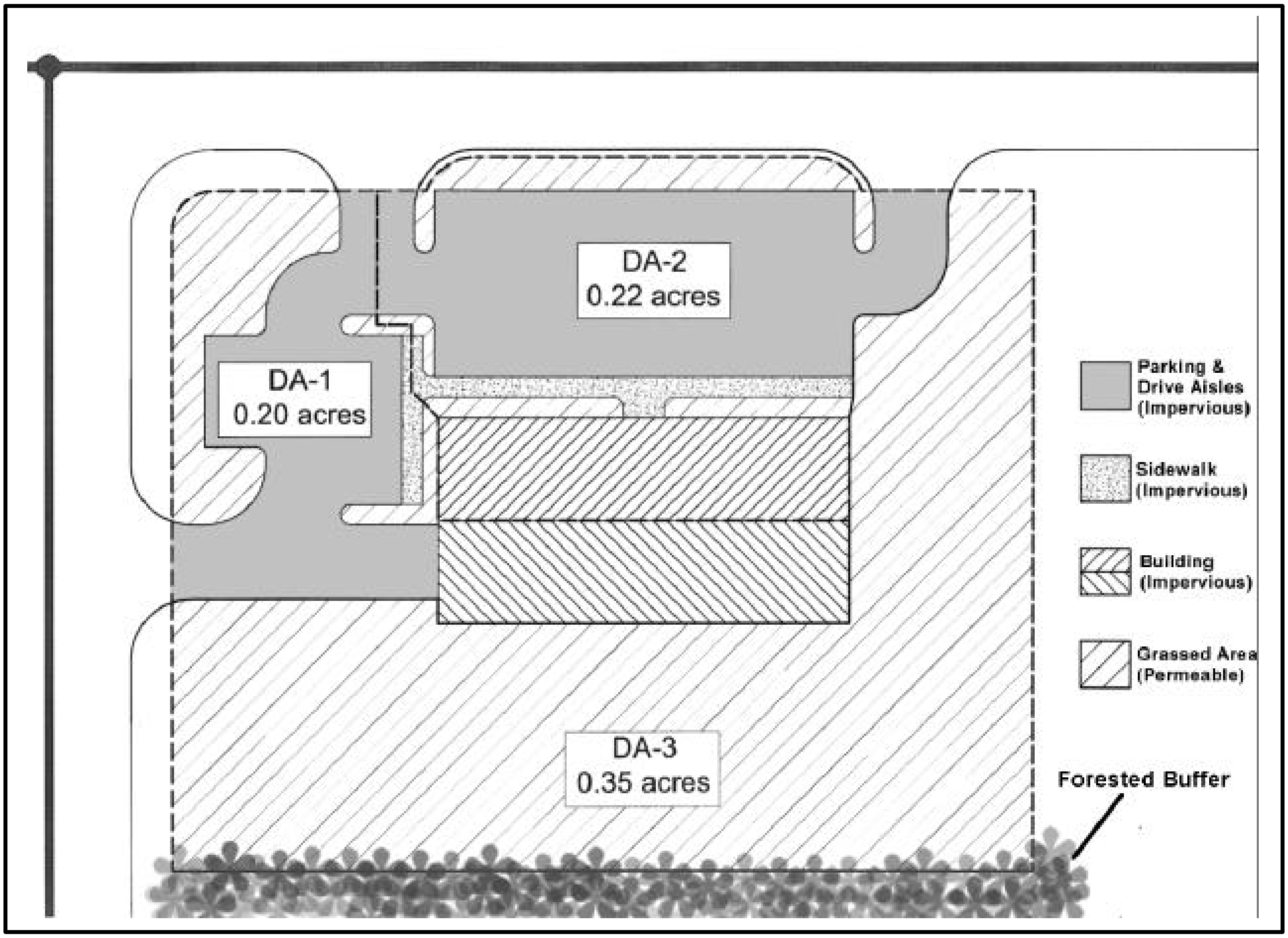
C.2

Design Example 2 – Water Quality BMPs

Design Example 2 – Water Quality BMPs

The following example demonstrates the design of several different BMPs for WQ_v and Re_v treatment including filtering, infiltration, and open channel practices.

Figure C.2.1 Comstock Commercial Center Site Plan



Site Specifc Data

Comstock Commercial Center is a 0.77 acre retail store located in Howard County, Maryland. The developed area of the site may be divided into two drainage areas of 0.20 and 0.22 acres respectively with a remaining drainage area of 0.35 acres. Total impervious area for the development is 0.36 acres; 0.16 acres in DA-1 and 0.20 acres in DA-2. Existing and proposed topography are not given for this exercise; it may be assumed that these conditions are amenable for each specific design. Likewise, the seasonally high water table will not be a factor in infiltration designs. The underlying soils are loams (HSG B). TR-55 calculations for the developed hydrologic conditions are shown in Figures C.2.2, C.2.3 and C.2.4.

C.2.1 Design Criteria

The site is within the Eastern Rainfall Zone and located on the Western Shore of the Chesapeake Bay (see Volume I, Chapter 2, Figures 2.1 and 2.4). Additionally, the site is located within a USE I watershed. Therefore, the following criteria apply:

1. WQ_v treatment is required. In the Eastern Rainfall Zone, P = 1”.
2. Re_v treatment is required.
3. Cp_v treatment is required.
4. Q_{p10} may be required by the local jurisdiction. For this example, Q_{p10} will not be required.
5. Q_f may be required by the local jurisdiction. For this example, Q will not be required. However, safe conveyance of the 100-year design storm is required through the proposed stormwater management facility.

C.2.2 Preliminary Design

Step 1. Compute WQ_v

Step 1a. Compute Volumetric Runoff Coefficient (R_v)

R_v = 0.05 + (0.009)(I); I = (0.36 acres / 0.77 acres) = 0.468or 46.8%
= 0.05 + (0.009)(46.8) = 0.471

Step 1b. Compute WQ_v

WQ_v = [(P)(R_v)(A)]/12
= [(1”)(0.471)(0.77 ac)]/12
= 0.0302 ac-ft (1,316.5 cf.)

Figure C.2.2 Comstock Commercial Center – Developed Conditions
(source: TR-55 computer printouts)

```
              RUNOFF CURVE NUMBER COMPUTATION          Version 2.10
Project : COMSTOCK COMMERCIAL                    User: SRC      Date: 09-17-1999
County  : HOWARD              State: MD      Checked: ____     Date: ________
Subtitle: DEVELOPED CONDITIONS
-------------------------------------------------------------------------------

                                                    Hydrologic Soil Group
         COVER DESCRIPTION                       A        B        C        D
                                                        Acres (CN)
--------------------------------------------------------------------------------
FULLY DEVELOPED URBAN AREAS (Veg Estab.)
Open space (Lawns,parks etc.)
  Good condition; grass cover > 75%              -  0.41(61)       -        -
Impervious Areas
  Paved parking lots, roofs, driveways           -  0.36(98)       -        -

Total Area (by Hydrologic Soil Group)               .77
--------------------------------------------------------------------------------
                TOTAL DRAINAGE AREA: .77 Acres       WEIGHTED CURVE NUMBER: 78*

                       GRAPHICAL PEAK DISCHARGE METHOD          Version 2.10


Project : COMSTOCK COMMERCIAL CENTER             User: SRC      Date: 12-07-1999
County  : HOWARD              State: MD      Checked: ____     Date: ________
Subtitle: DEVELOPED CONDITIONS

  Data: Drainage Area        :  .77   Acres
        Runoff Curve Number  :  78
        Time of Concentration:  0.10   Hours (MINIMUM VALUE)
        Rainfall Type        :  II
        Pond and Swamp Area  :  NONE

==========================================================================
| Storm Number          |   1  |   2  |   3  |   4  |   5  |   6  |   7  |
|-----------------------|------|------|------|------|------|------|------|
| Frequency  (yrs)      |   1  |   2  |   5  |  10  |  25  |  50  |  100 |
| 24-Hr Rainfall (in)   |  2.6 |  3.2 |  4.2 |  5.1 |  5.6 |  6.3 |  7.2 |
| Ia/P Ratio            | 0.22 | 0.18 | 0.13 | 0.11 | 0.10 | 0.09 | 0.08 |
|            Used       | 0.22 | 0.18 | 0.13 | 0.11 | 0.10 | 0.10 | 0.10 |
| Runoff (in)           | 0.85 | 1.27 | 2.05 | 2.80 | 3.23 | 3.85 | 4.66 |
| Unit Peak Discharge   |1.511 |1.534 |1.558 |1.572 |1.578 |1.578 |1.578 |
|    (cfs/acre/in)      |      |      |      |      |      |      |      |
| Pond and Swamp Factor| 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 |
|    0.0% Ponds Used    |      |      |      |      |      |      |      |
|-----------------------|------|------|------|------|------|------|------|
| Peak Discharge (cfs) |    1 |    2 |    2 |    3 |    4 |    5 |    6 |
==========================================================================

* - Value(s) provided from TR-55 system routines
```

Figure C.2.3 Comstock Commercial Center – Drainage Area (DA) 1
(source: TR-55 computer printouts)

```
              RUNOFF CURVE NUMBER COMPUTATION            Version 2.10
Project : COMSTOCK COMMERCIAL                      User: SRC      Date: 09-27-1999
County  : HOWARD               State: MD      Checked: ____     Date: ________
Subtitle: DRAINAGE AREA DA-1
-------------------------------------------------------------------------------
                                                    Hydrologic Soil Group
         COVER DESCRIPTION                        A        B        C        D
                                                        Acres (CN)
-------------------------------------------------------------------------------
FULLY DEVELOPED URBAN AREAS (Veg Estab.)
Open space (Lawns,parks etc.)
  Good condition; grass cover > 75%               -   .04 (61)      -        -

Impervious Areas
  Paved parking lots, roofs, driveways            -   0.16(98)      -        -

Total Area (by Hydrologic Soil Group)                 .20
                                                      ====
-------------------------------------------------------------------------------
                TOTAL DRAINAGE AREA: .20 Acres      WEIGHTED CURVE NUMBER: 91*
-------------------------------------------------------------------------------
* - Generated for use by GRAPHIC method

                       GRAPHICAL PEAK DISCHARGE METHOD           Version 2.10


Project : COMSTOCK COMMERCIAL CENTER               User: SRC      Date: 12-07-1999
County  : HOWARD               State: MD      Checked: ____     Date: ________
Subtitle: DEVELOPED CONDITIONS DA-1

  Data: Drainage Area        :  .2   Acres
        Runoff Curve Number  :  91
        Time of Concentration:  0.10   Hours
        Rainfall Type        :  II
        Pond and Swamp Area  :  NONE

=========================================================================
| Storm Number          |   1  |   2  |   3  |   4  |   5  |   6  |   7  |
|-----------------------|------|------|------|------|------|------|------|
| Frequency  (yrs)      |   1  |   2  |   5  |  10  |  25  |  50  |  100 |
| 24-Hr Rainfall (in)   |  2.6 |  3.2 |  4.2 |  5.1 |  5.6 |  6.3 |  7.2 |
| Ia/P Ratio            | 0.08 | 0.06 | 0.05 | 0.04 | 0.04 | 0.03 | 0.03 |
|            Used       | 0.10 | 0.10 | 0.10 | 0.10 | 0.10 | 0.10 | 0.10 |
| Runoff (in)           | 1.70 | 2.26 | 3.21 | 4.08 | 4.57 | 5.25 | 6.14 |
| Unit Peak Discharge   |1.578 |1.578 |1.578 |1.578 |1.578 |1.578 |1.578 |
|    (cfs/acre/in)      |      |      |      |      |      |      |      |
| Pond and Swamp Factor| 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 |
|    0.0% Ponds Used    |      |      |      |      |      |      |      |
|-----------------------|------|------|------|------|------|------|------|
| Peak Discharge (cfs) |     1 |     1 |     1 |     1 |     1 |     2 |     2 |
=========================================================================
```

Figure C.2.4 Comstock Commercial Center – Drainage Area (DA) 2 (source: TR-55 computer printouts)

```
                RUNOFF CURVE NUMBER COMPUTATION            Version 2.10
Project : COMSTOCK COMMERCIAL CENTER             User: SRC      Date: 09-21-1999
County  : HOWARD                State: MD     Checked: ____     Date: ________
Subtitle: DRAINAGE AREA DA-2 DEVELOPED
-------------------------------------------------------------------------------
                                                    Hydrologic Soil Group
         COVER DESCRIPTION                        A        B        C        D
                                                        Acres (CN)
-------------------------------------------------------------------------------
FULLY DEVELOPED URBAN AREAS (Veg Estab.)
Open space (Lawns,parks etc.)
  Good condition; grass cover > 75%               -  0.02(61)       -        -

Impervious Areas
  Paved parking lots, roofs, driveways            -  0.20(98)       -        -

Total Area (by Hydrologic Soil Group)                 .22
                                                     ====
-------------------------------------------------------------------------------
                TOTAL DRAINAGE AREA: .22 Acres       WEIGHTED CURVE NUMBER: 95*
-------------------------------------------------------------------------------
* - Generated for use by GRAPHIC method

                       GRAPHICAL PEAK DISCHARGE METHOD           Version 2.10
  Data: Drainage Area        :  .22 * Acres
        Runoff Curve Number  :  95 *
        Time of Concentration:  0.10   Hours (MINIMUM VALUE)
        Rainfall Type        :  II
        Pond and Swamp Area  :  NONE

=========================================================================
| Storm Number          |   1  |   2  |   3  |   4  |   5  |   6  |   7  |
|-----------------------|------|------|------|------|------|------|------|
| Frequency  (yrs)      |   1  |   2  |   5  |  10  |  25  |  50  |  100 |
| 24-Hr Rainfall (in)   |  2.6 |  3.2 |  4.2 |  5.1 |  5.6 |  6.3 |  7.2 |
| Ia/P Ratio            | 0.04 | 0.03 | 0.03 | 0.02 | 0.02 | 0.02 | 0.01 |
|            Used       | 0.10 | 0.10 | 0.10 | 0.10 | 0.10 | 0.10 | 0.10 |
| Runoff (in)           | 2.06 | 2.64 | 3.63 | 4.52 | 5.01 | 5.71 | 6.60 |
| Unit Peak Discharge   |1.578 |1.578 |1.578 |1.578 |1.578 |1.578 |1.578 |
|    (cfs/acre/in)      |      |      |      |      |      |      |      |
| Pond and Swamp Factor| 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 |
|    0.0% Ponds Used    |      |      |      |      |      |      |      |
|-----------------------|------|------|------|------|------|------|------|
| Peak Discharge (cfs) |    1 |    1 |    1 |    2 |    2 |    2 |    2 |
=========================================================================

* - Value(s) provided from TR-55 system routines
```

Step 2. Compute Re_v

Step 2a. Determine Soil Specific Recharge Factor (S) Based on Hydrologic Soil Group

Soils found throughout the site are loams (HSG B) therefore $S = 0.26$

Step 2b. Compute Re_v Using Percent Volume Method

$$
\begin{aligned}
Re_v &= [(S)(R_v)(A)]/12 \\
&= [(0.26)(0.471)(0.77)]/12 \\
&= \underline{0.0078 \text{ ac-ft.}} \ (342.3 \text{ cf})
\end{aligned}
$$

Step 2c. Compute Re_v Using Percent Area Method

$$
\begin{aligned}
Re_v &= (S)(A_i) \\
&= (0.26)(0.36 \text{ ac.}) \\
&= 0.094 \text{ acres } (4{,}095 \text{ sf.})
\end{aligned}
$$

The Re_v requirement may be met by: a) treating 342.3 cubic feet using structural methods, b) treating 4,095 square feet using non-structural methods, or c) a combination of both.

Step 3. Compute Cp_v

The proposed community center is located within a USE I watershed, therefore use an extended detention time (T) of 24 hours for the one-year storm event. The time of concentration (t_c) and one-year runoff (Q_a) are 0.10 hours and 0.85" respectively.

Use the MDE Method to Compute Storage Volume (Appendix D.11):

Initial abstraction (I_a) for CN of 78 is 0.564: (TR-55) [$I_a = (200/CN)-2$]

$I_a/P = (0.564)/2.6" = 0.22$
$t_c = 0.10$ hours

$q_u = 975$ csm/in. (Figure D.11.1, Appendix D.11)

$$
\begin{aligned}
q_i &= q_u A Q_a \quad \text{where } A \text{ is the drainage area in square miles} \\
&= (975 \text{ csm})(0.0012 \text{ square miles})(0.85") \\
&= 1.0 \text{ cfs}; \ q_i < 2.0 \text{ cfs} \therefore Cp_v \text{ is not required.}
\end{aligned}
$$

Step 4. Compute Requirements for Sub-Drainage Areas DA-1, DA-2 and DA-3

DA-1: $R_v = 0.05 + (0.009)(I)$; $I = 0.16$ acres / 0.20 acres = 0.80 or 80%
$= 0.05 + (0.009)(80.0) = 0.77$

$WQ_v = [(P)(R_v)(A)]/12$
= [(1")(0.77)(0.20 ac)]/12
= 0.0128 ac-ft (557.5 cf.)

$Re_v = [(S)(R_v)(A)]/12$
= [(0.26)(0.77)(0.20 ac)]/12
= 0.0033 ac-ft (145 cf.)

DA-2: $R_v = 0.05 + (0.009)(I)$; $I = 0.20$ acres / 0.22 acres = 0.91 or 91%
$= 0.05 + (0.009)(91) = 0.87$

$WQ_v = [(P)(R_v)(A)]/12$
= [(1")(0.87)(0.22 ac.)]/12
= 0.0160 ac-ft (694.8 cf.)

$Re_v = [(S)(R_v)(A)]/12$
= [(0.26)(0.87)(0.22 ac.)]/12
= 0.0041 ac-ft (180.6 cf.)

DA-3: $R_v = 0.05 + (0.009)(I)$; $I = 0.0$ acres / 0.35 acres = 0.0 or 0%
$= 0.05 + (0.009)(0.0) = 0.05$

Because $I < 15\%$, WQ_v = 0.2"/acre
WQ_v = [(0.2")(0.35 ac.)]/12
= 0.0058 ac-ft (254.1 cf.)

$Re_v = [(S)(R_v)(A)]/12$
= [(0.26)(0.05)(0.35 ac.)]/12
= 0.0004 ac-ft (16.5 cf.)

NOTE: Although DA-3 has no proposed impervious surfaces, portions of DA-3 will be disturbed to construct structural BMPs for DA-1 and DA-2. As a result, WQ_v and Re_v must be addressed for DA-3. For this example, the portion of DA-3 not disturbed for BMP construction shall be treated by promoting sheet flow into the adjacent forested buffer (see Chapter 5.4, "Sheetflow to Buffer Credit").

Table C.2.1 Summary of General Storage Requirements for Comstock Commercial Center

Requirement	Drainage Area	Volume Required (cubic feet)	Notes
WQ_v*	Total	1,316.5	The sum of treatment volumes for DA-1, DA-2 and DA-3 is greater than that calculated for the entire site.
	DA-1	557.5	
	DA-2	694.8	
	DA-3	254.1	
Re_v*	Total	342.3 (or 4,095 sf.)	volume is included within the WQ_v storage
	DA-1	145.6 (or 1,812 sf.)	
	DA-2	180.6 (or 2,265 sf.)	
	DA-3	16.1	
Cp_v		N/A	Cp_v inflow rate is $<$ 2.0 cfs
Q_{p10}		N/A	not required
Q_f		N/A	provide safe passage for the 100-year event in final design

C.2.3 BMP Design Option 1

The first option consists of the design of a perimeter sand filter (F-3) for DA-1 and a pocket sand filter (F-5) for DA-2. In both designs, Re_v storage will be provided below the filter's underdrain system. As a result, the entire WQ_v must be considered in the design of each filter system. A plan view for Option 1 is shown in Figure C.2.5

C.2.3.1 Perimeter Sand Filter (F-3) for DA-1

Pretreatment

The pretreatment requirements for a perimeter sand filter are as follows:

The pretreatment volume (*Vp*) for the perimeter sand filter shall be at least 25% of the computed WQ_v:

$$\begin{aligned} V_p &= (0.25)(WQ_v) \\ &= (0.25)(557.5 \text{ cf.}) \\ &= 139.4 \text{ cf.} \end{aligned}$$

The minimum required surface area as computed by the Camp-Hazen equation:

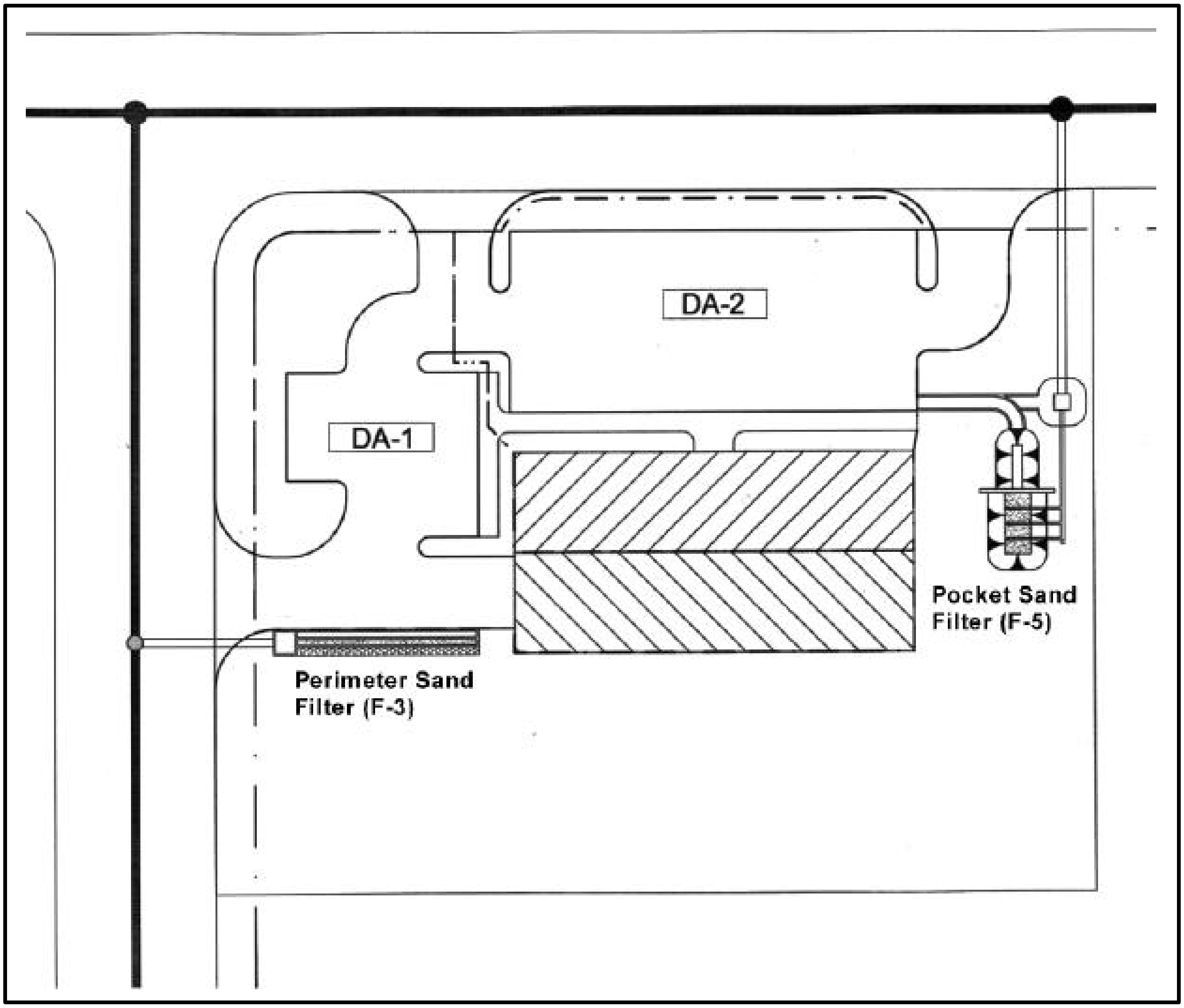
$$A_s = \frac{Q_o}{W} \times E' \qquad \text{(see Section 3.4.3 for terms)}$$

For imperviousness (I) > 75%, this equation reduces to:

$$\begin{aligned} A_{sp} &= (0.0081)(WQ_v) \\ &= (0.0081)(557.5 \text{ cf.}) \\ &= 4.52 \text{ sf.} \end{aligned}$$

Using a width (w) = 1.5 ft. and length (l) = 45 ft., the required depth for the sedimentation chamber = 139.4 cf. / (1.5 ft.)(45 ft.) = 2.06 ft.; **Use a sedimentation chamber 1.5 ft. by 45 ft. by 2.1 ft.**

Figure C.2.5 Design Option 1 - Plan View



Treatment

The treatment requirements for the perimeter sand filter are as follows:

The entire treatment system (including pretreatment) shall temporarily hold at least 75% of the WQ_v prior to filtration:

$$
\begin{aligned}
V_{temp} &= (0.75)(WQ_v) \\
&= (0.75)(557.5 \text{ cf.}) \\
&= 418.1 \text{ cf.}
\end{aligned}
$$

The required filter bed area (A_f) is computed using the following equation:

$$A_f = {(WQ_v)(d_f)} \big/ {[k \times (h_f + d_f) \times t_f)]}$$ (see Section 3.4.4)

Minimum filter bed depth (d_f) = 12”; for this design use d_f = 12” (1.0 ft)

The coefficient of permeability (k) for sand filters = 3.5 ft./day

The average height of water above the filter bed (h_f) = (0.5)(design ponding depth). For this design, the ponding depth $=1.0$ ft. $\therefore$ hf = 0.5 ft.

The design filter bed drain time (t_f) = 1.67 days

Therefore: $A_f = \dfrac{(557.5\ cf.)(1.0\ ft.)}{[(3.5\ {}^{ft.}\!/\!_{day})(0.5\ ft. + 1.0\ ft.)(1.67\ days)]}$ = 63.6 sf.

Setting the filter chamber width (w) to 1.5 ft., the length (l) of the filter chamber = 63.6 sf./1.5 ft. = 42.4 ft; **Use a filter chamber 1.5 ft. by 45 ft.**

Check V_{temp}:

$$V_{temp} = V_p + V_{treatment}$$
$$= 139.4 + [(1.0)(1.5)(45) + (1.0)(1.5)(45)(0.4)] = 236.5 \text{ cf.}$$

note: 0.4 is the porosity of the filter media

Approximately 182 cf. of additional storage is needed to meet this requirement. Either increase the storage in one or both chambers or design parking area to provide additional storage. For this design, the pretreatment chamber width will be increased to 3.5 ft.

$$V_{temp} = V_p + V_{treatment}$$
$$= (3.5)(45.0)(2.1) + [(1.0)(1.5)(45) + (1.0)(1.5)(45)(0.4)] = 425.25 \text{ cf.}$$

Groundwater Recharge (Re_v)

Re_v storage will be provided within a stone-filled trench adjacent to the perimeter sand filter. Setting the trench length (l) = 45 ft., and the width (w) = 2.0 ft, the trench depth (d) needed to store the Re_v volume ($V = 145.6$ cf.) is:

$$d = \frac{V}{l \times w \times n}$$ where n is the porosity of stone; use n = 0.4

Therefore, d = 145.6 cf./(45.0 ft. $\times$ 2.0 ft. $\times$ 0.4) = 4.04 ft.; **use a stone-filled trench 45.0 ft. by 2.0 ft. by 4.1 ft.**

Overflow

Flow splitters and overflow devices may be designed using volume or flow rate. For this example, a weir discharging from the sedimentation chamber into the clear well will provide volume overflow for the ten-year storm. For DA-1, the ten-year flow (Q_{10}) = 1.0 cfs. Using a weir length of 1.5 ft., the head required to safely convey Q_{10} may be calculated using the weir equation: $Q = Clh^{3/2}$ where C = 3.1, l = weir length (1.5 ft.), and h = head. By rearranging the weir equation and solving for h; $h = [Q/(C \times l)]^{2/3}$ = 0.40 ft. **Design perimeter sand filter with at least 0.4 ft. freeboard to safely convey Q10.**

Design details for the perimeter sand filter are shown in Figures C.2.6.

C.2.3.2 Pocket Sand Filter (F-5) for DA-2

Pretreatment

The pretreatment requirements for a pocket sand filter are as follows:

V_p for the pocket sand filter shall be at least 25% of the computed WQ_v:

$$
\begin{aligned}
V_p &= (0.25)(WQ_v) \\
&= (0.25)(694.8 \text{ cf.}) \\
&= 173.7 \text{ cf.}
\end{aligned}
$$

The minimum required surface area as computed by the Camp-Hazen equation:

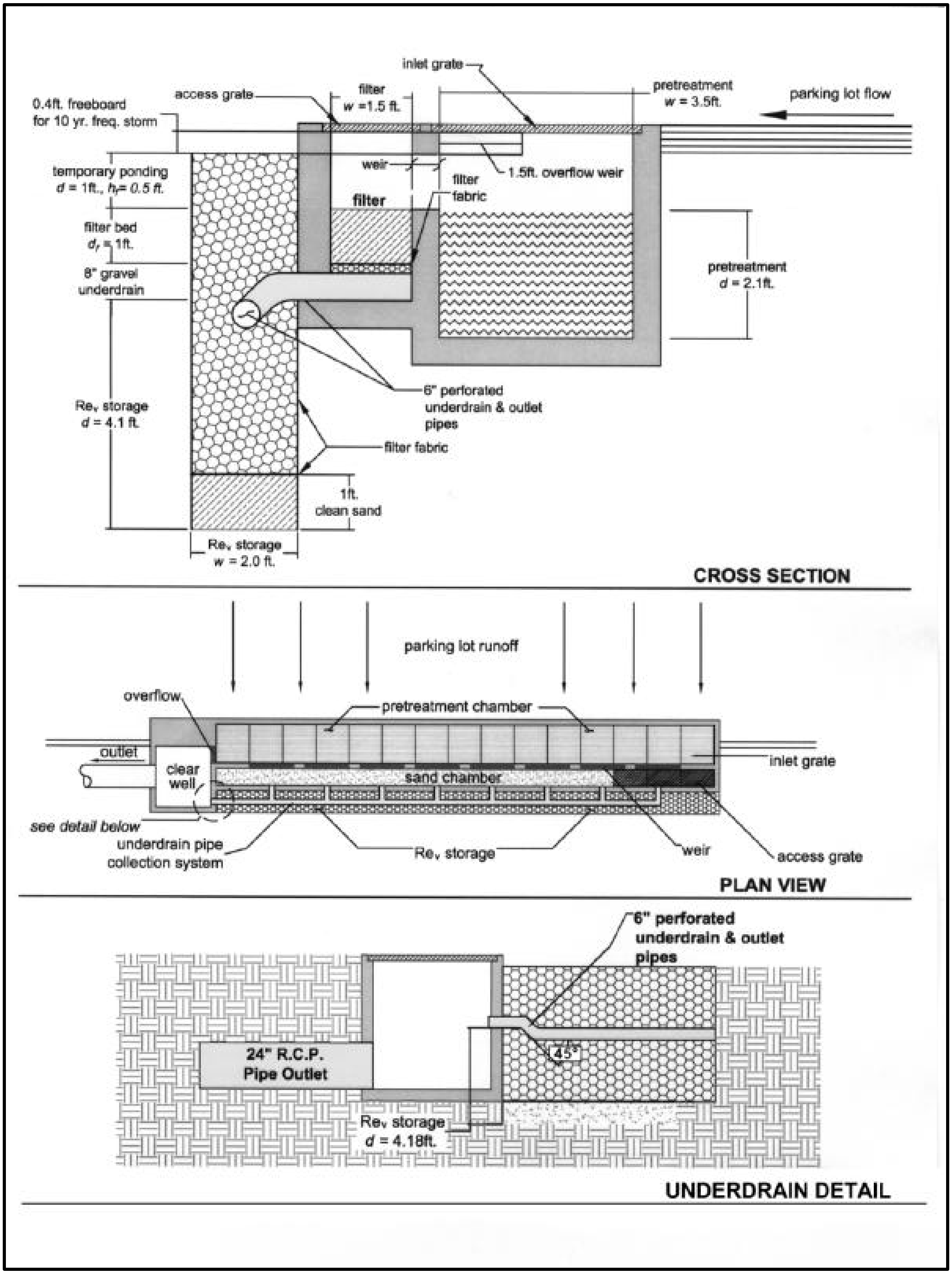
$$A_s = \frac{Q_o}{W} \times E'$$

For $I > 75\%$, this equation reduces to:

$$
\begin{aligned}
A_{sp} &= (0.0081)(WQ_v) \\
&= (0.0081)(694.8 \text{ cf.}) \\
&= 5.62 \text{ sf.}
\end{aligned}
$$

Maintaining at least a 2:1 ratio (l:w); set w = 6.5 ft. and l = 13.0 ft. The required d for the sedimentation area = 173.7 cf. /(6.5 ft.)(13.0 ft.) = 2.0 ft.; **Use a sedimentation chamber 6.5 ft. by 13.0 ft. by 2.0 ft.**

Figure C.2.6 Perimeter Sand Filter Design Details



Treatment

The treatment requirements for the pocket sand filter are as follows:

The entire treatment system (including pretreatment) shall temporarily hold at least 75% of the WQv prior to filtration:

$$V_{temp} = (0.75(WQv)$$
$$= (0.75)(694.8 \text{ cf.})$$
$$= 521.1 \text{ cf.}$$

The required filter bed area is computed using the following equation:

The minimum d_f for a pocket sand filter = 18"; for this design use d_f = 18" (1.5').

$$A_f = \frac{(WQ_v)(d_f)}{[k \times (h_f + d_f) \times t_f)]}$$

The coefficient of permeability (k) for sand filters = 3.5 ft/day

The average height of water (h_f) above the filter bed for this design = 0.5 ft.

The design filter bed drain time (t_f) = 1.67 days.

Therefore: $$A_f = \frac{(694.8\ cf.)(1.5\ ft.)}{[(3.5\ ^{ft.}/_{day})(0.5\ ft. + 1.5\ ft.)(1.67\ days)]} = 89.2\ sf.$$

Setting the filter chamber width (w) to 6.5 ft. l = 89.2 ft./6.5 ft. = 13.7 ft.; **Use a filter chamber 6.5 ft. by 13.7 ft.**

Check V_{temp}: $$V_{temp} = V_p + V_{treatment}$$
$$= 173.7 + [(1.0)(6.5)(13.7) + (1.5)(6.5)(13.7)(0.4)] = 316.1 \text{ cf.}$$
note: 0.4 is the porosity of the filter media

Approximately 205 cf. of additional storage is needed to meet this requirement. Either increase the storage in one or both chambers or design parking area to provide additional storage. For this design, the pretreatment chamber width will be increased to 9.0 ft. and the depth increased to 3.0 ft.

$$V_{temp} = V_p + V_{treatment}$$
$$= (9.0)(13.0)(3.0) + [(1.5)(6.5)(13.7) + (1.5)(6.5)(13.7)(0.4)] = 538.0 \text{ cf}$$

Groundwater Recharge (Re_v)

Re_v storage will be provided within a stone-filled reservoir directly below the filter chamber's underdrain system. Using $w = 6.5$ ft. and $l = 13.7$ ft., the depth needed to store the Re_v volume ($V = 180.6$ cf.) is:

$$d = \frac{V}{l \times w \times n}$$ where n is the porosity of stone; use $n = 0.4$

Therefore, $d = 180.6/(13.7 \text{ ft.} \times 6.5 \text{ ft.} \times 0.4) = 5.1$ ft.; **Use a stone-filled reservoir 6.5 ft. by 13.7 ft. by 5.1 ft.**

Overflow/Bypass

As the pocket sand filter will be located "off-line" from the main conveyance system, a flow splitter will be required to divert the WQv into the filter. Flow splitters may be designed using volume or flow rate. For this example, use a concrete flume with a bottom width of 4.0 ft designed to divert the flow associated with the WQ_v. The head required to divert the WQv flow may be calculated using the weir equation: $Q = Clh^{3/2}$ where Q is flow associated with WQv (using Appendix D.10, $Q = 0.3$ cfs), C = 3.1, $l = 4.0$ ft., and $h = $ head. By rearranging the equation and solving for h; $h = [Q/(C \times l)]^{2/3} = 0.084$ ft. **Design flow splitter with a 1 inch high diversion.** NOTE: With this type of flow splitter, runoff in excess of the WQ_v may continue to flow into the sand filter.

Design details for the pocket sand filter are shown in Figures C.2.7 and C.2.8.

Figure C.2.7 Pocket Sand Filter – Plan View

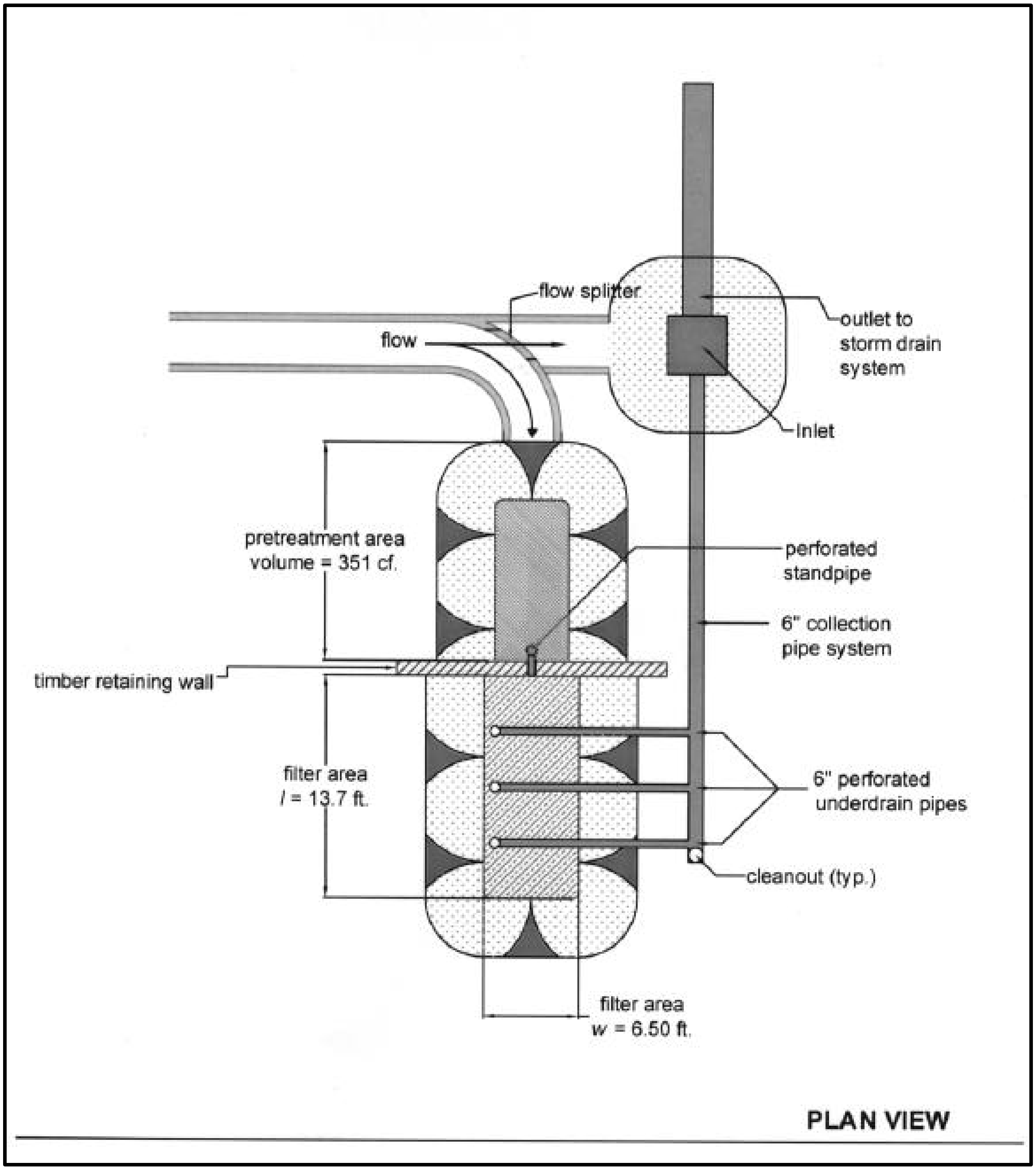
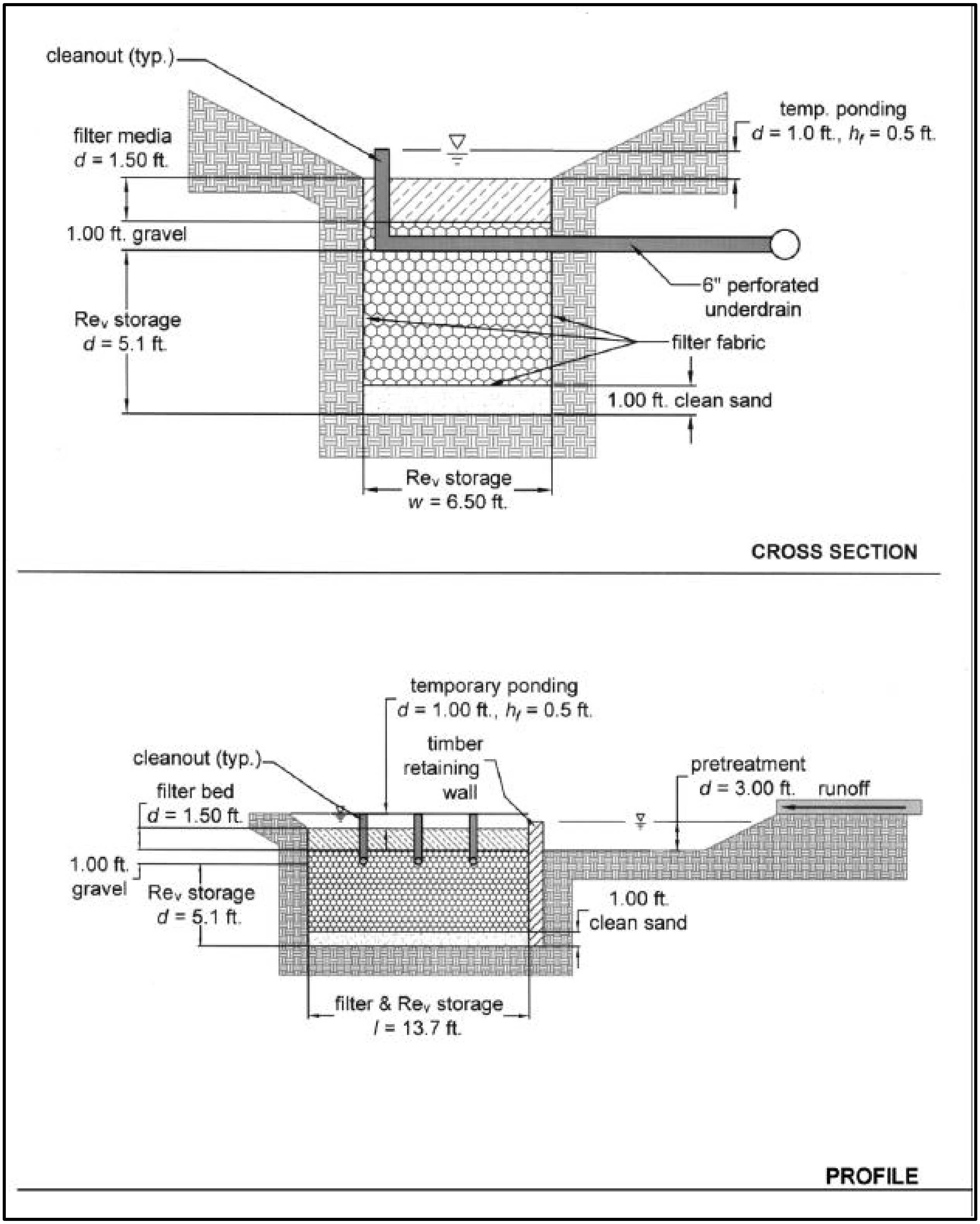


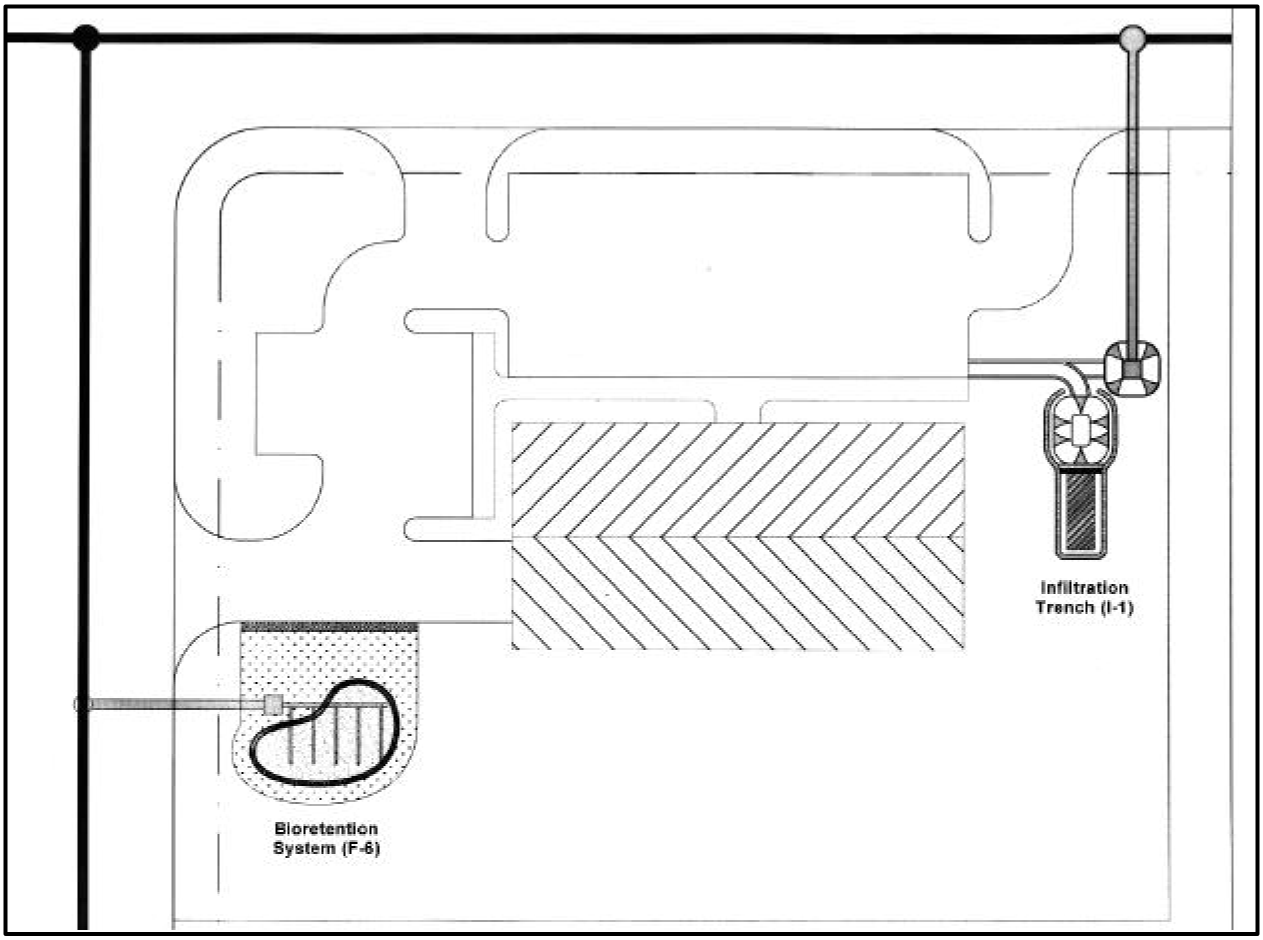
Figure C.2.8 Pocket Sand Filter Design Details



C.2.4 BMP Design Option 2.

The second option consists of the design of a bioretention area (F-6) for DA-1 and an infiltration trench (I-1) for DA-2. For the bioretention system, Re_v storage will be provided below the underdrain system, and as a result, the entire WQ_v will be used as the design. The infiltration trench automatically meets the Re_v requirement. A plan view of Option 2 is shown in Figure C.2.9.

Figure C.2.9 Design Option 2 – Plan View



C.2.4.1 Bioretention System (F-6) for DA-1

Pretreatment

Adequate pretreatment for a bioretention system is provided when all of the following are provided:

1. 20 ft. grass filter strip below a level spreader or an optional sand filter layer;
2. gravel diaphragm; and
3. 2” to 3” mulch layer.

Treatment

The treatment requirements for the bioretention system are as follows (Section 3.4.3 & 4):

The entire treatment system (including pretreatment) shall temporarily hold at least 75% of the WQv prior to filtration:

$$\begin{aligned} V_{temp} &= (0.75)(WQ_v) \\ &= (0.75)(557.5 \text{ cf.}) \\ &= 418.1 \text{ cf.} \end{aligned}$$

The required filter bed area (A_f) is computed using the following equation:

$$A_f = (WQ_v)(d_f) \Big/ [k \times (h_f + d_f) \times t_f)]$$

Recommended filter bed depth (d_f) for a bioretention system is 2.5 to 4.0 ft. For this design, use d_f = 3.0 ft.

The coefficient of permeability (k) for bioretention systems = 0.5 ft./day

The average height of water above the filter bed (h_f) = 0.5 ft. (Note: The maximum ponding depth for a bioretention system is 1.0 ft.)

The design filter bed drain time (t_f) = 2.0 days

Therefore: $$A_f = \frac{(557.5\ cf.)(3.0\ ft.)}{[(0.5\ {}^{ft.}\!/_{day})(0.5\ ft. + 3.0\ ft.)(2.00\ days)]} = 477.9\ sf.$$

Use a bioretention system with minimum surface area = 478 sf.

Check V_{temp}: $V_{temp} = V_{treatment} = (1.0)(478\ sf) + (3.0)(478\ sf)(0.4) =$ 1051.6 cf.

note: 0.4 is the porosity of the filter media

Groundwater Recharge (Re_v)

Re_v storage will be provided in a stone-filled reservoir directly below the underdrain system. Setting the reservoir area (A_r) = 478 sf., the depth (d) needed to store the Re_v volume ($V = 145.6$ cf.) is:

$$d = \frac{V}{A_r \times n} \qquad \text{where } n \text{ is the porosity of stone; use } n = 0.4$$

Therefore; $d = 145.6 \text{ cf.} / (478.0 \text{ft.} \times 0.4) = 0.76 \text{ ft.}$; **Use a stone-filled reservoir 478 sf. by 0.76 ft.**

Overflow

Overflow for the ten-year storm shall be provided to a non-erosive outlet. For this design, a standard inlet will be used to bypass the volume in excess of the WQ_v by setting the inlet invert at the elevation corresponding to the WQ_v treatment volume (1.0 ft. above the bioretention system filter bed).

Design details and a planting plan for the bioretention system are shown in Figures C.2.10 and C.2.11.

Figure C.2.10 Bioretention System Details

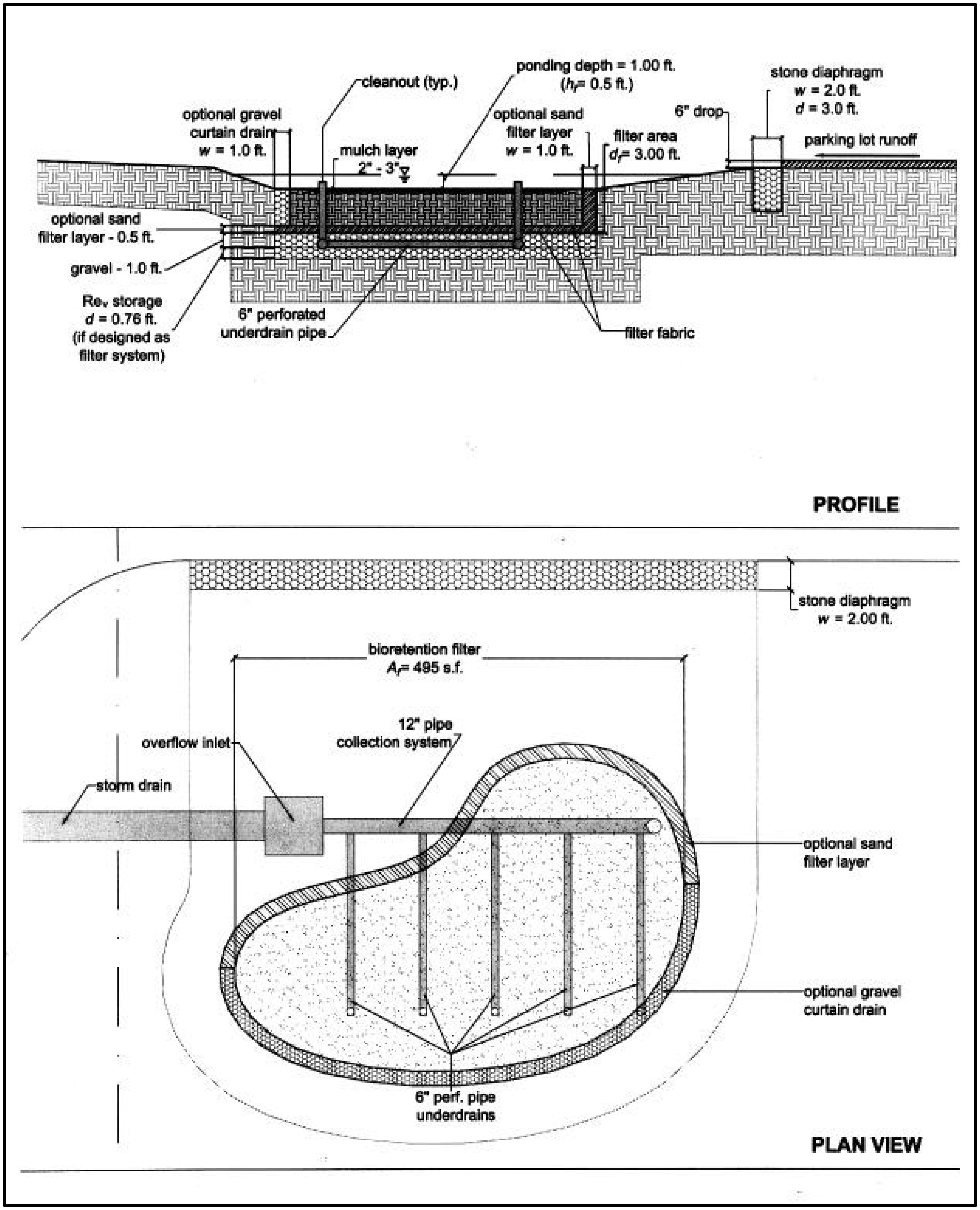
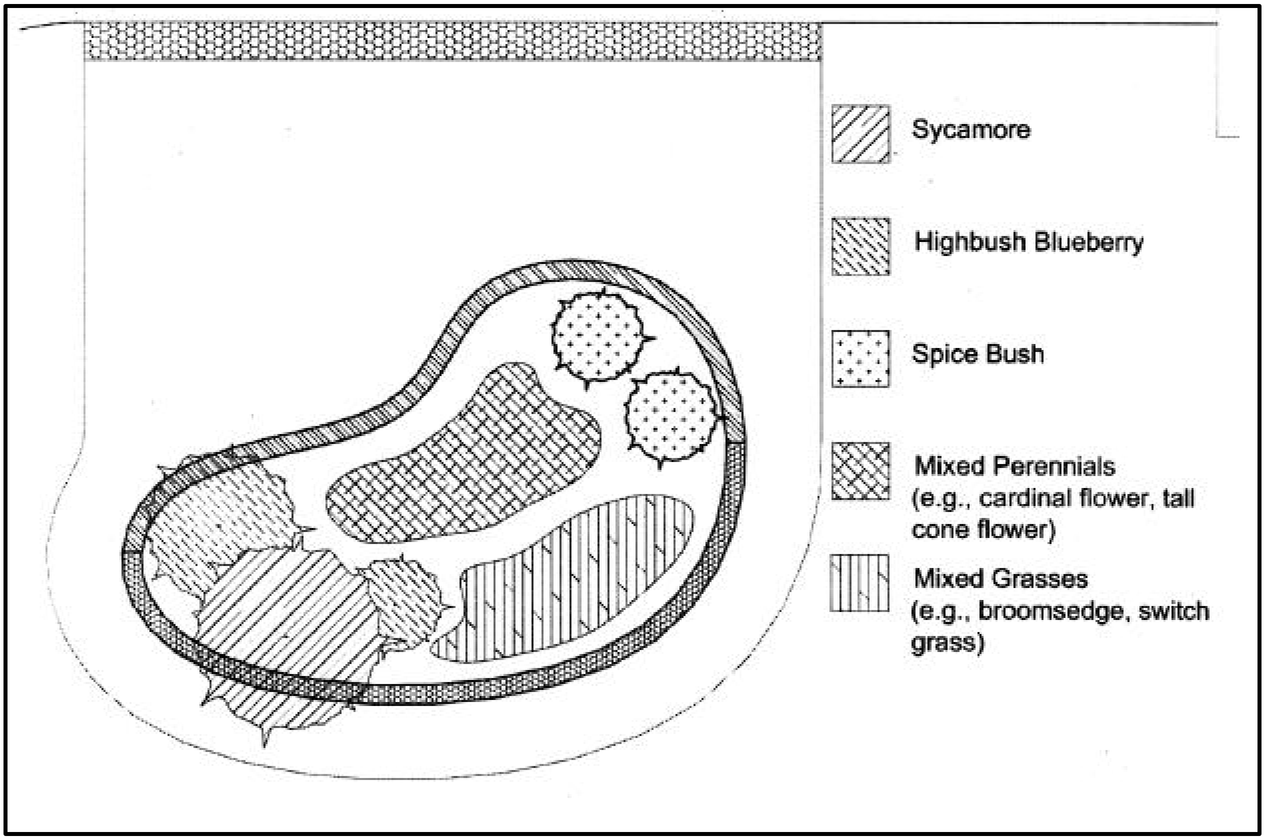


Figure C.2.11 Bioretention Planting Plan



C.2.4.2 Infiltration Trench (I-1) for DA-2

Pretreatment

The pretreatment requirements for an infiltration trench are as follows:

The pretreatment volume (Vp) for the infiltration trench shall be at least 25% of the computed WQ_v:

$$
\begin{aligned}
V_p &= (0.25)(WQ_v) \\
&= (0.25)(694.8 \text{ cf.}) \\
&= 173.7 \text{ cf.}
\end{aligned}
$$

Using a width (w) of 8.0 ft. and a length (l) of 11.0 ft., the required depth for the sedimentation chamber = 173.7 cf. /(8.0 ft.)(11.0 ft.) = 1.97 ft.; **Use a sedimentation chamber 8.0 ft by 11.0 ft. by 2.0 ft.**

Additionally, each infiltration trench shall have at least three of the following measures to prevent clogging and maintain the long-term integrity of the trench:

1. grass channel;
2. grass filter strip (minimum 20 ft.);

3. bottom sand layer
4. upper sand layer (minimum 6”) with filter fabric at sand/gravel interface; and
5. use washed bank run gravel as aggregate.

This design will use a bottom sand layer, upper sand layer, and washed bank run gravel.

Treatment

The treatment requirements for an infiltration trench are as follows:

The practice shall be designed to exfiltrate the entire WQv less the pretreatment volume through the floor of the practice. The design volume (V_w) = WQv-V_p = 521.1 cf.

Infiltration practices are designed using the methodology in Appendix D.13.

The maximum allowable depth (d_{max}) of an infiltration trench is

$$d_{\max} = f \times \frac{T_s}{n}$$

where:

f is the infiltration rate, for this design $f = 0.52$ inches/hour
T_s is the maximum allowable storage of 48 hours
n is the porosity of the stone reservoir, use 0.4

Therefore, d_{max} = $0.52 \text{ inches/hour} \times (48 \text{ hours}/0.4) = 62.4$ inches (5.2 ft). **Use a trench depth (d_t) = 5.0 ft.**

Using equation D.13.3, the area of the infiltration trench (A_t) is:

$$A_t = \frac{V_w}{nd_t + fT}$$ where the time to fill the trench (T) is 2.0 hours.

$$A_t = \frac{521.1\, cf.}{(0.4 \times 5.0) + (0.52 \text{ inches/hour} \times 2.0\, hours \times 1\, ft / 12\, in)} = 249.7\, sf.$$

Use an infiltration trench 7.5 ft. by 35.0 ft. by 5.0 ft.

Groundwater Recharge (Re_v)

Infiltration trenches automatically meet the Re_v storage requirement; no additional storage is required.

Overflow

As the infiltration trench will be located "off-line" from the main conveyance system, a flow splitter will be required to divert the WQv into the filter. Use the flow splitter design from the pocket sand filter above.

Design details for the infiltration trench are shown in Figures C.2.12.

C.2.5 BMP Design Option 3

The third option consists of the bioretention area (F-6) previously designed for DA-1 and a dry swale (O-1) for DA-2. In the dry swale design, Re_v storage will be provided below the swale's underdrain system. As a result, the entire WQv must be considered in the design of the dry swale. A plan view of Option 3 is shown in Figure C.2.13.

<u>C.2.5.1 Dry Swale (O-1) for DA-2</u>

Pretreatment

The pretreatment requirements for a dry swale are as follows:

> Pretreatment storage of 0.1 inch of runoff from impervious area shall be provided. This is equivalent to 10% of WQv. Therefore, $V_P = (10\%)(WQv) =$ 69.5 cf. **Use a forebay or sedimentation chamber sized to store 62.5 cf.**

Figure C.2.12 Infiltration Trench Details

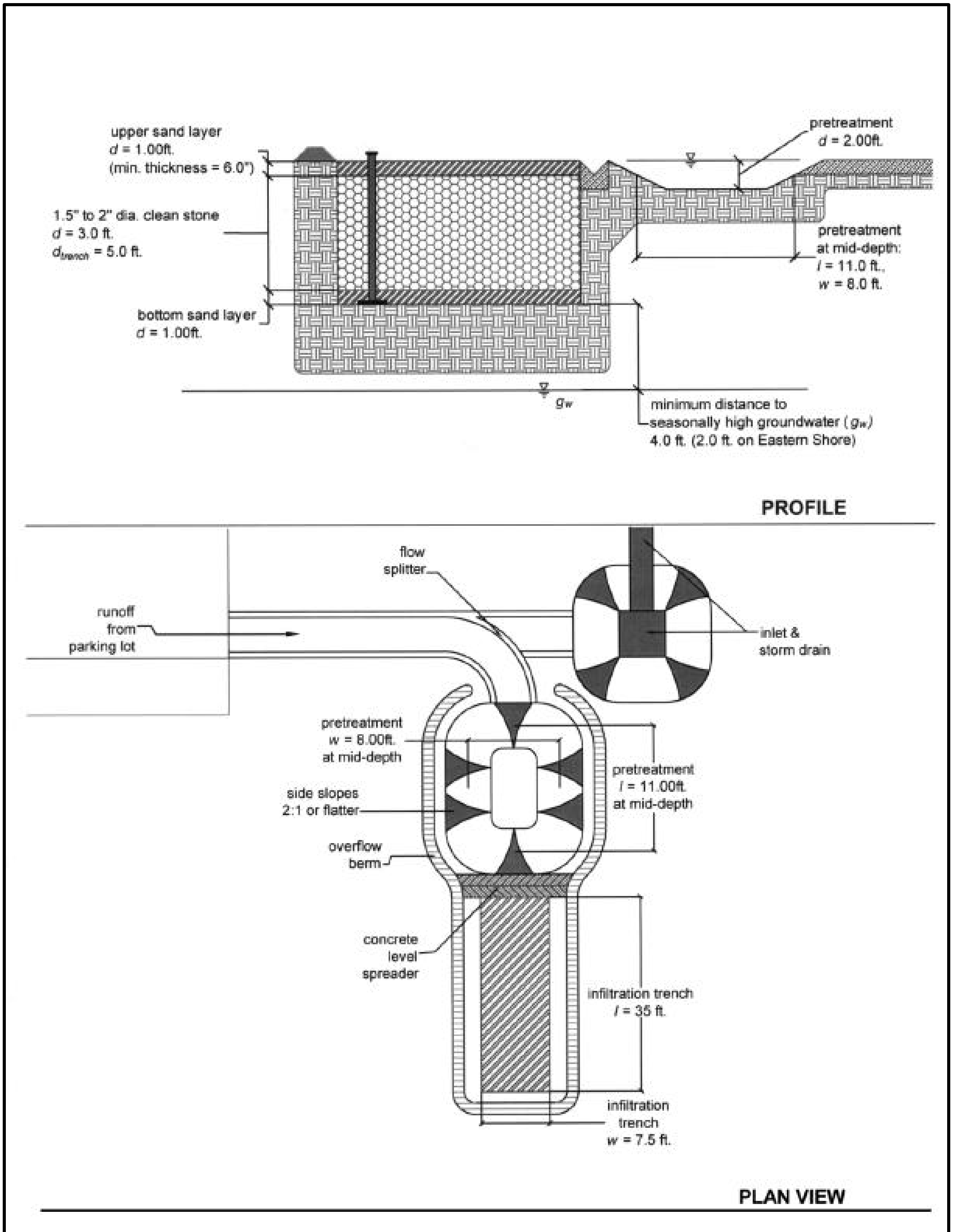
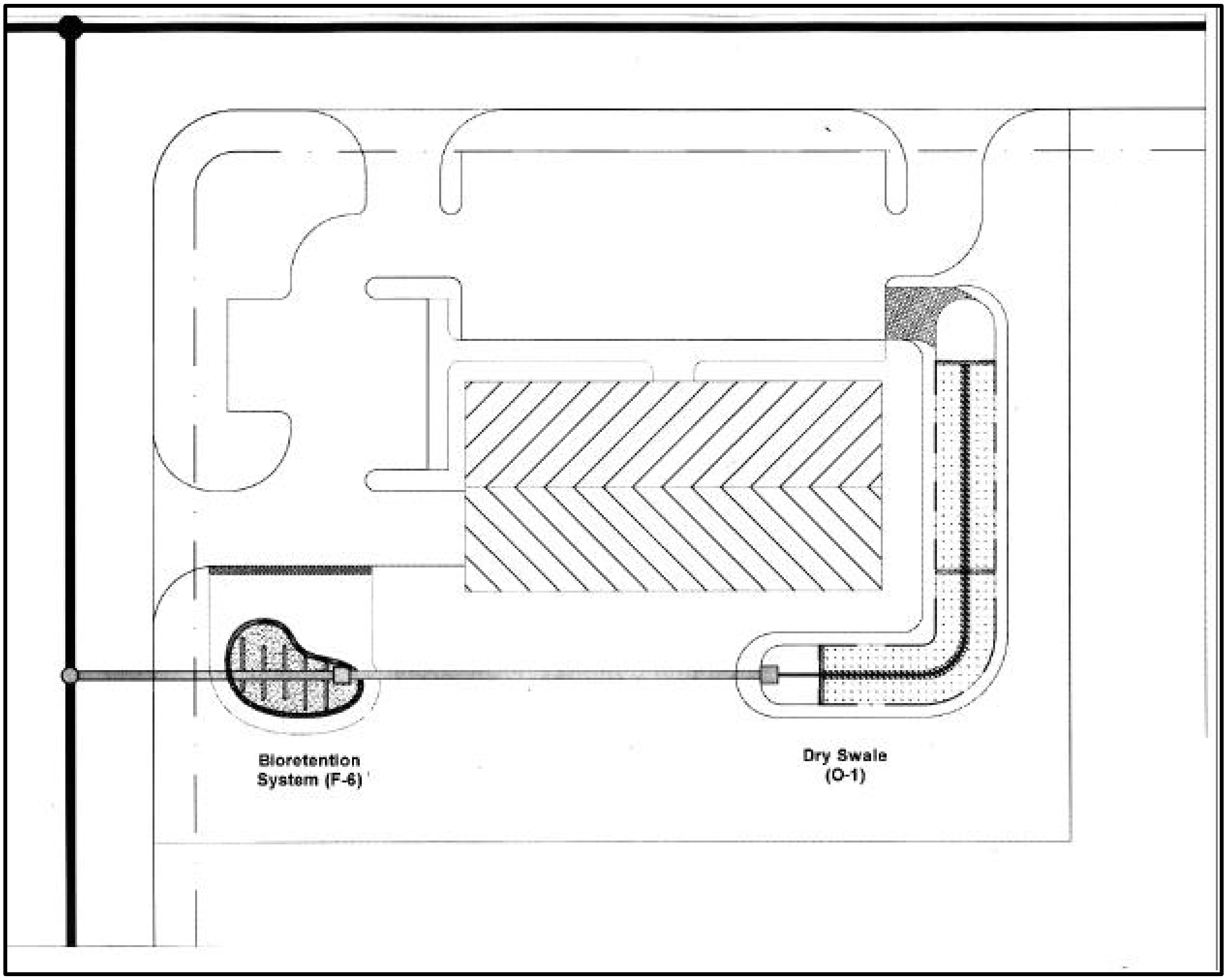


Figure C.2.13 Design Option 3 – Plan View



Treatment

The treatment requirements for the dry swale are as follows:

> Dry swales shall be designed to temporarily store the WQv for a maximum 48-hour period. An underdrain system shall provided to ensure the maximum ponding time is not exceeded.
>
> Dry swales shall have a maximum longitudinal slope (*s*) of 4.0%. **For this design, s = 3.0%.**
>
> Channel side slopes (*z*:1) should be no steeper than 2:1. **For this design, side slopes shall be 4:1 (z = 4).**

Dry swales shall have a bottom width (w_b) no narrower than 2.0 ft. and no wider than 8.0 ft. (if wider than 8.0 ft., a meandering drainage pattern shall be established).

Maximum ponding depths (d_{mid}, d_{end}) of 1.0 ft. at the channel mid-point and 1.5 ft. at the downstream end shall be maintained. *Use d_{mid} = 0.75 ft. and d_{max} = 1.5 ft.*

Due to the length (100 ft.) and grade (3.0%) of the channel, the channel will be divided into two contiguous channels separated by a check dam to achieve d_{mid} and d_{end} requirements. *Use three check dams located at the entrance, mid-point and end of the swale.*

With three check dams, there will be two ponding areas of equal storage. Using $d_{mid} = 0.75'$, and setting the total length of the swale to 100 ft., the treatment volume of the swale is:

$$WQ_v - V_p = w \times l \times d_{mid}$$

By rearranging this equation and solving for the width of storage surface (w):

$$w = \frac{WQ_v - V_p}{l \times d_{mid}} = \frac{694.8\ cf. - 69.5\ cf.}{100\ ft. \times 0.75\ ft.} = 8.3\ ft.$$

Using $w = 8.3$ ft. and 4:1 side slopes, $w_b = w\text{-}(2 \times z \times d_{mid}) = 8.3\text{-}(2 \times 4 \times 0.75) = 2.3$ ft. **Use a dry swale with bottom dimensions of 2.3 ft. by 100 ft. with 4:1 side slopes.**

Groundwater Recharge (Re_v)

Re_v storage will be provided within a stone-filled reservoir below the dry swale underdrain system. Using the swale dimensions (2.3 ft by 100 ft.), the reservoir depth (d) needed to store the Re_v volume ($V = 180.6$ cf.) is:

$$d = \frac{V}{l \times w \times n} \quad \text{where } n \text{ is the porosity of stone; use } n = 0.4$$

Therefore, $d = 180.6\text{ cf.}/(100\text{ ft.} \times 2.3\text{ ft.} \times 0.4) = 1.96$ ft. **Use a stone-filled reservoir 2.3 ft. by 100.0 ft. by 2.0 ft.**

Overflow (*Q_{10}* Conveyance)

A dry swale is required to safely convey the 10-year design storm with minimum freeboard of 3 inches. Check the design to ensure that the 10-year storm is conveyed non-erosively and that the minimum freeboard is provided. For DA-2, the 10-year peak flow (Q_{10}) $= 2.0$ cfs. At d_{max}, the width (w_{max}) $= w + (2 \times z \times d_{mid}) = 14.3$ ft. Using a trapezoidal channel with a bottom width $= 14.3$ ft., 4:1 side slopes, and a longitudinal slope (s) $= 3.0\%$, the depth (d) and velocity (v) of flow can be calculated using the Manning equation:

$$v = \frac{1.49}{n} r^{2/3} s^{1/2}$$

where: n is the roughness coefficient of the channel lining, use 0.025
r is the hydraulic radius of the channel; at $d = 0.10$ ft., r is very nearly 0.10

Therefore, at $d = 0.1$ ft.:

$$v = \frac{1.49}{0.025}(0.10)^{2/3}(0.03)^{1/2} = 2.2 \ fps$$

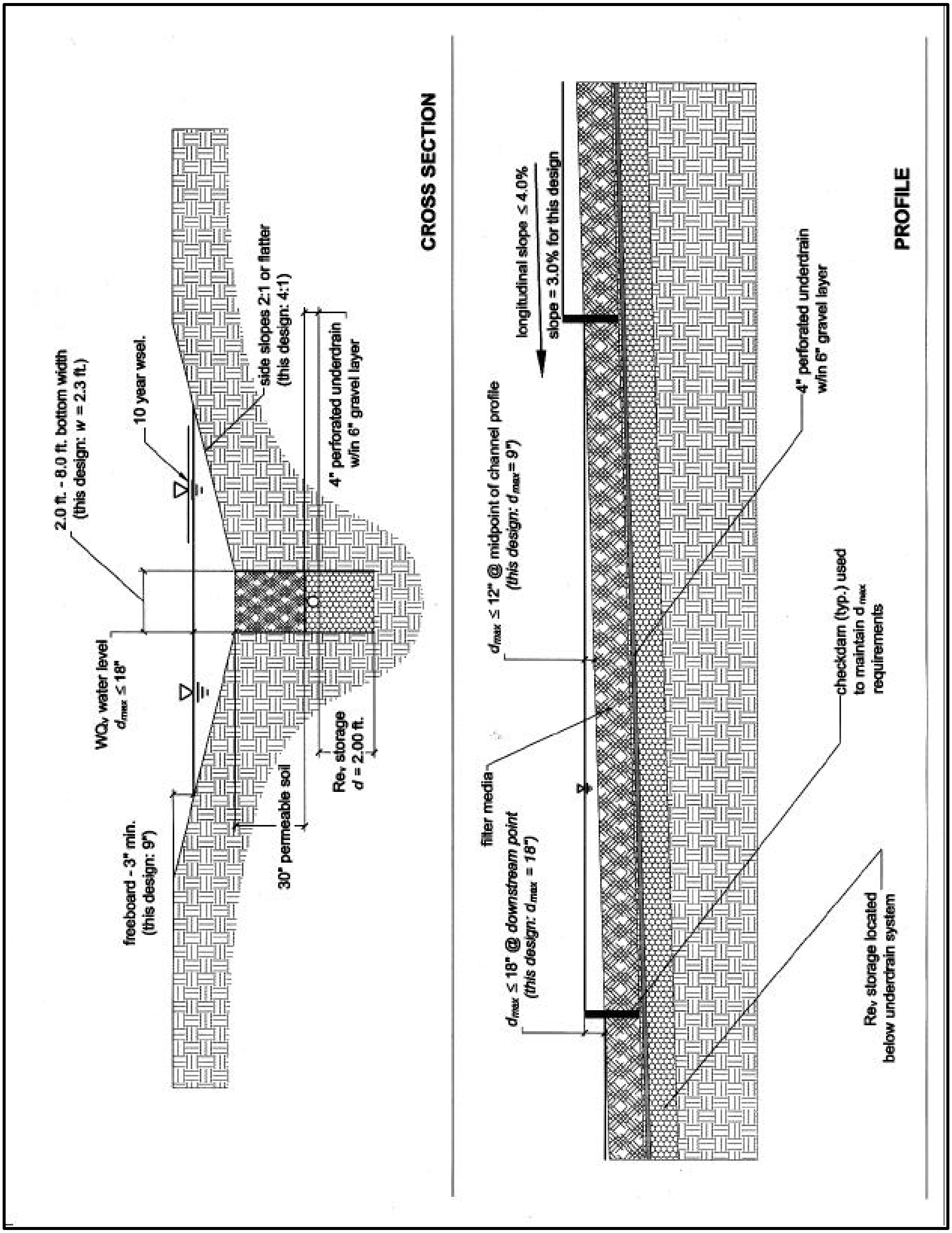
The cross-sectional area of the channel (A) needed to safely pass Q_{10} can be calculated using $A = Q_{10}/v = 2.0$ cfs / 2.2 fps $= 0.91$ sf. At $d = 0.1$ ft., $A = 1.4$ sf. **The proposed design will safely convey the 10-year storm.**

The minimum depth of the channel (d_c) may be determined by adding the required depths:

$$d_c = d_{max} + d_{10\ yr.\ storm} + d_{freeboard}$$
$$= 1.5 \text{ ft.} + 0.1 \text{ ft.} + 0.25 \text{ ft.}$$
$$= 1.85 \text{ ft.}$$ **Use channel depth (d_c) = 2.0 ft.**

Design details for the dry swale are shown in Figures C.2.14.

C.2.14 Dry Swale Design Details



d. Determine United States Department of Agriculture (USDA) or Unified Soil Classification (USC) System textures at the proposed bottom and 4 feet below the bottom of the best management practice (BMP);

e. Determine depth to bedrock (if within 4 feet of proposed bottom);

f. The soil description should include all soil horizons; and

g. The location of the test pit or boring shall correspond to the BMP location; test pit/soil boring stakes are to be left in the field for inspection purposes and shall be clearly labeled as such.

Infiltration Testing Requirements (field testing required)

a. Install casing (solid 5 inch diameter, 30” length) to 24” below proposed BMP bottom (see Figure D.1.1).

b. Remove any smeared soiled surfaces and provide a natural soil interface into which water may percolate. Remove all loose material from the casing. Upon the tester’s discretion, a two (2) inch layer of coarse sand or fine gravel may be placed to protect the bottom from scouring and sediment. Fill casing with *clean* water to a depth of 24” and allow to pre-soak for twenty-four hours.

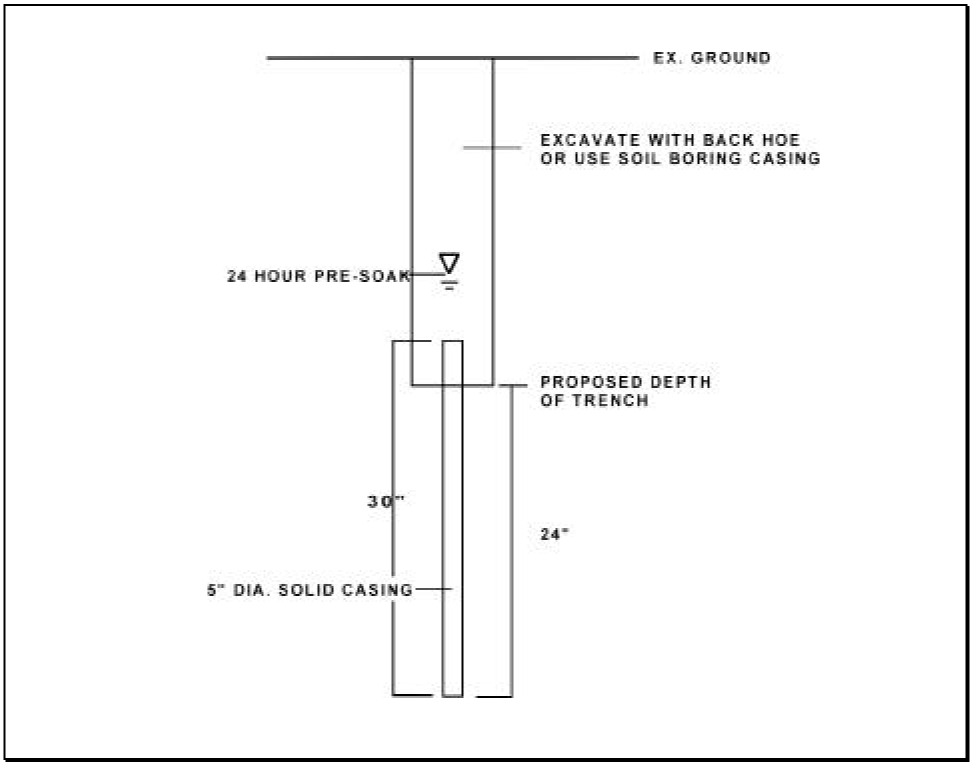
c. Twenty-four hours later, refill casing with another 24” of clean water and monitor water level (measured drop from the top of the casing) for 1 hour. Repeat this procedure (filling the casing each time) three additional times, for a total of four observations. Upon the tester’s discretion, the final field rate may either be the average of the four observations, or the value of the last observation. The final rate shall be reported in inches per hour.

d. May be done through a boring or open excavation.

e. The location of the test shall correspond to the BMP location.

f. Upon completion of the testing, the casings shall be immediately pulled, and the test pit shall be back-filled.

Figure D.1.1 Infiltration Testing Requirements



Laboratory Testing

Use grain-size sieve analysis and hydrometer tests (where appropriate) to determine USDA soils classification and textural analysis. Visual field inspection by a qualified professional may also be used, provided it is documented. The use of lab testing to establish infiltration rates is prohibited.

Bioretention Testing

All areas tested for application of F-6 facilities shall be back-filled with a suitable sandy loam planting media. The borrow source of this media, which may be the same or different from the bioretention area location itself, must be tested as follows:

If the borrow area is undisturbed soil one test is required per 200 square feet of borrow area. The test consists of "grab" samples at one foot depth intervals to the bottom of the borrow area. All samples at the testing location are then mixed, and the resulting sample is then lab-tested to meet the following criteria: