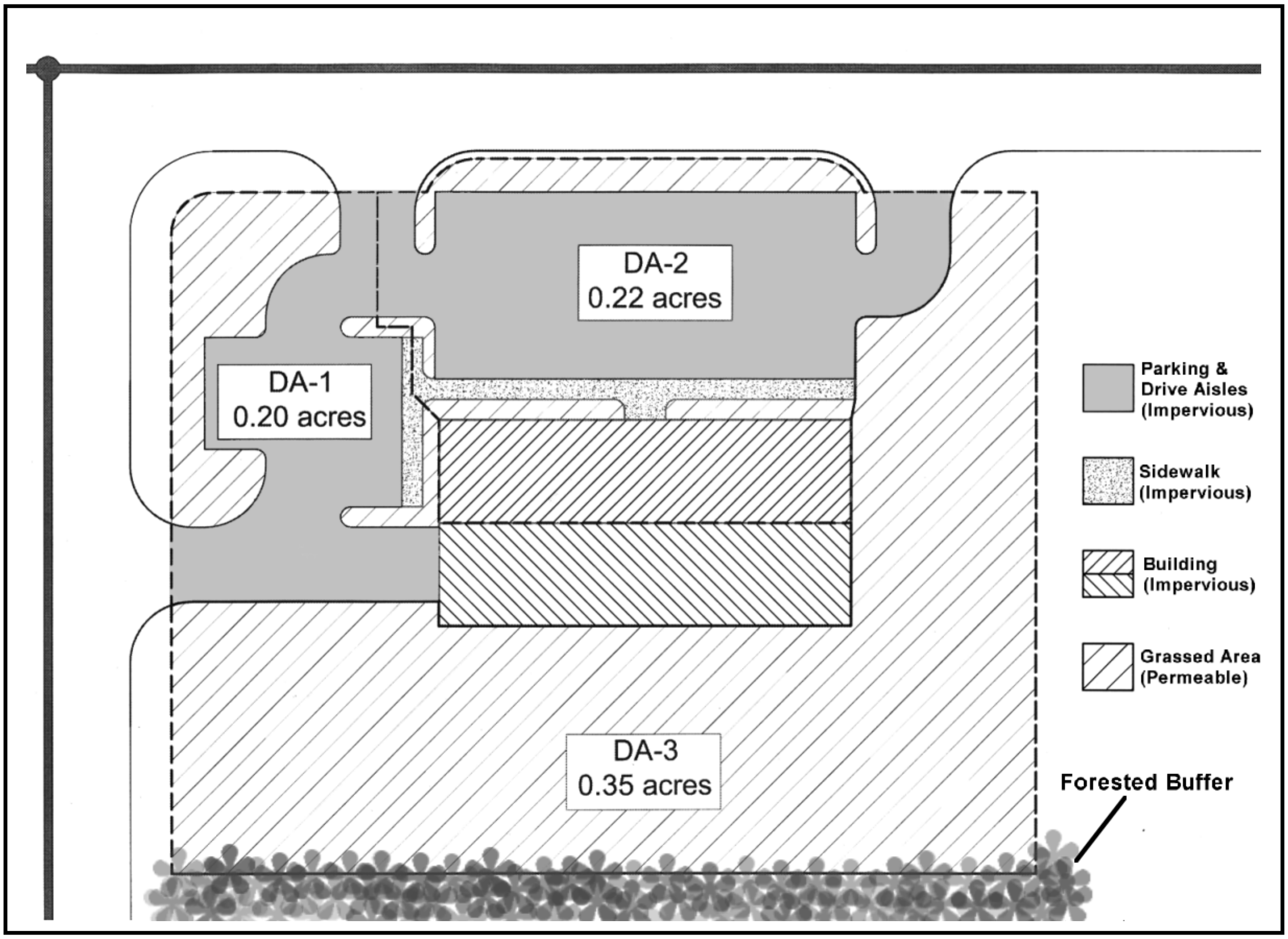
Appendix C.2

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_f 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 (cfs/acre/in)	1.578	1.578	1.578	1.578	1.578	1.578	1.578
Pond and Swamp Factor 0.0% Ponds Used	1.00	1.00	1.00	1.00	1.00	1.00	1.00
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 (cfs/acre/in)	1.578	1.578	1.578	1.578	1.578	1.578	1.578
Pond and Swamp Factor 0.0% Ponds Used	1.00	1.00	1.00	1.00	1.00	1.00	1.00
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)

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

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:

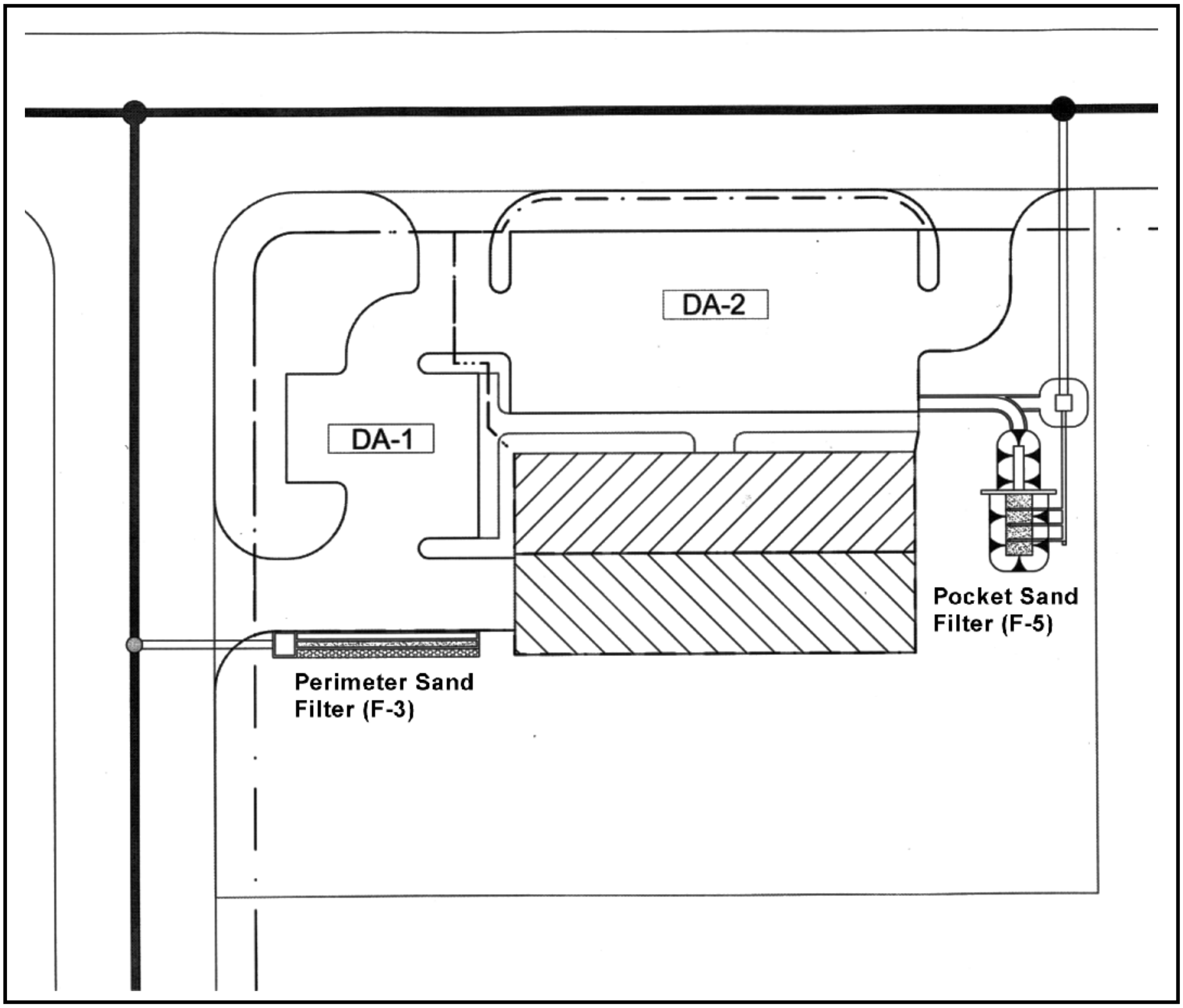
$$A_s = \frac{Q_o}{W} \times E' \qquad \text{(see Section 3.4.3 for terms)}$$

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:

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

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}: $V_{temp} = V_p + V_{treatment}$

$= 139.4 + [(1.0)(1.5)(45) + (1.0)(1.5)(45)(0.4)] = 236.5$ 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. × 2.0 ft. × 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 Q_{10}.**

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:

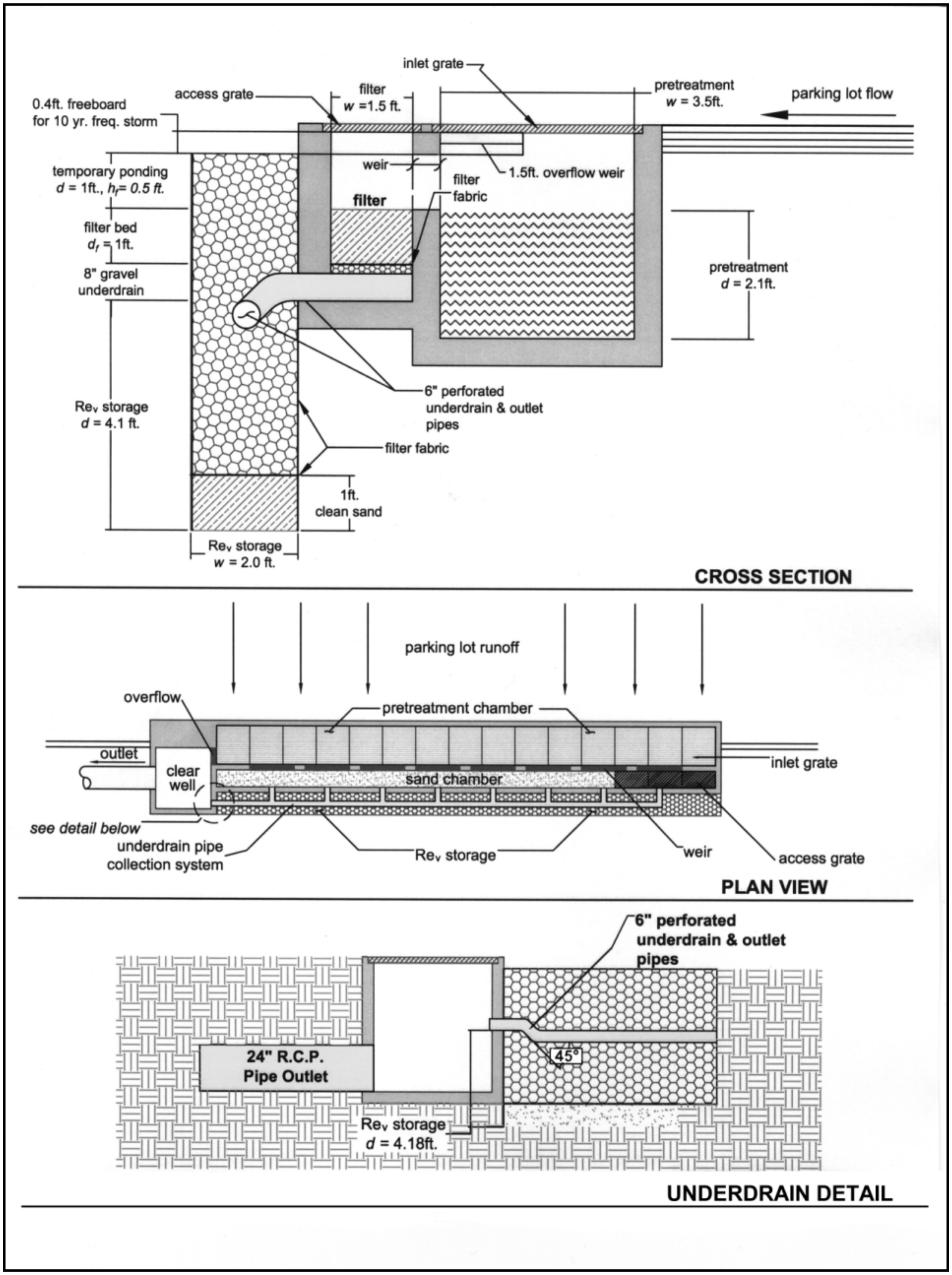
$$A_s = \frac{Q_o}{W} \times E'$$

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:

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

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').

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

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

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

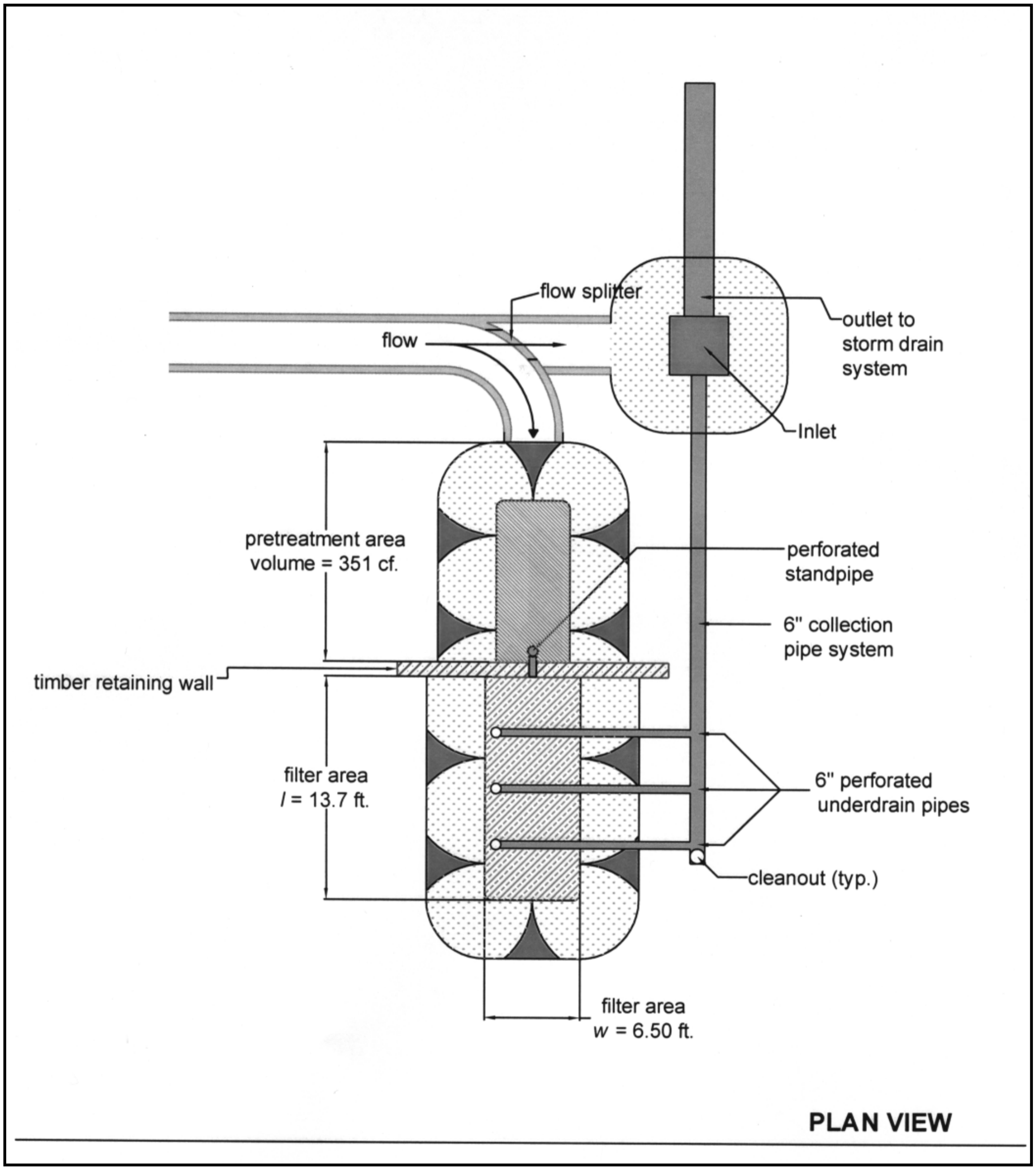
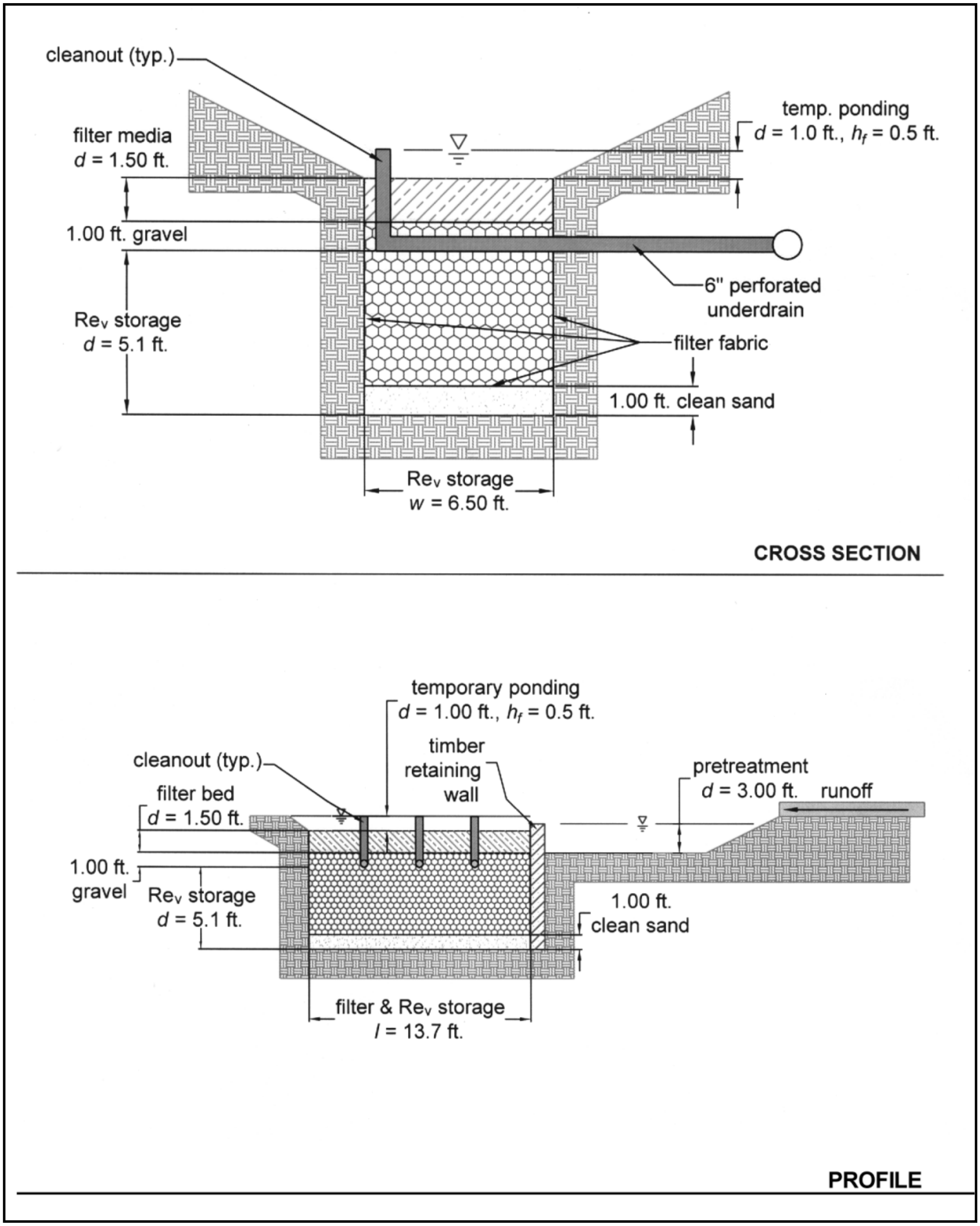


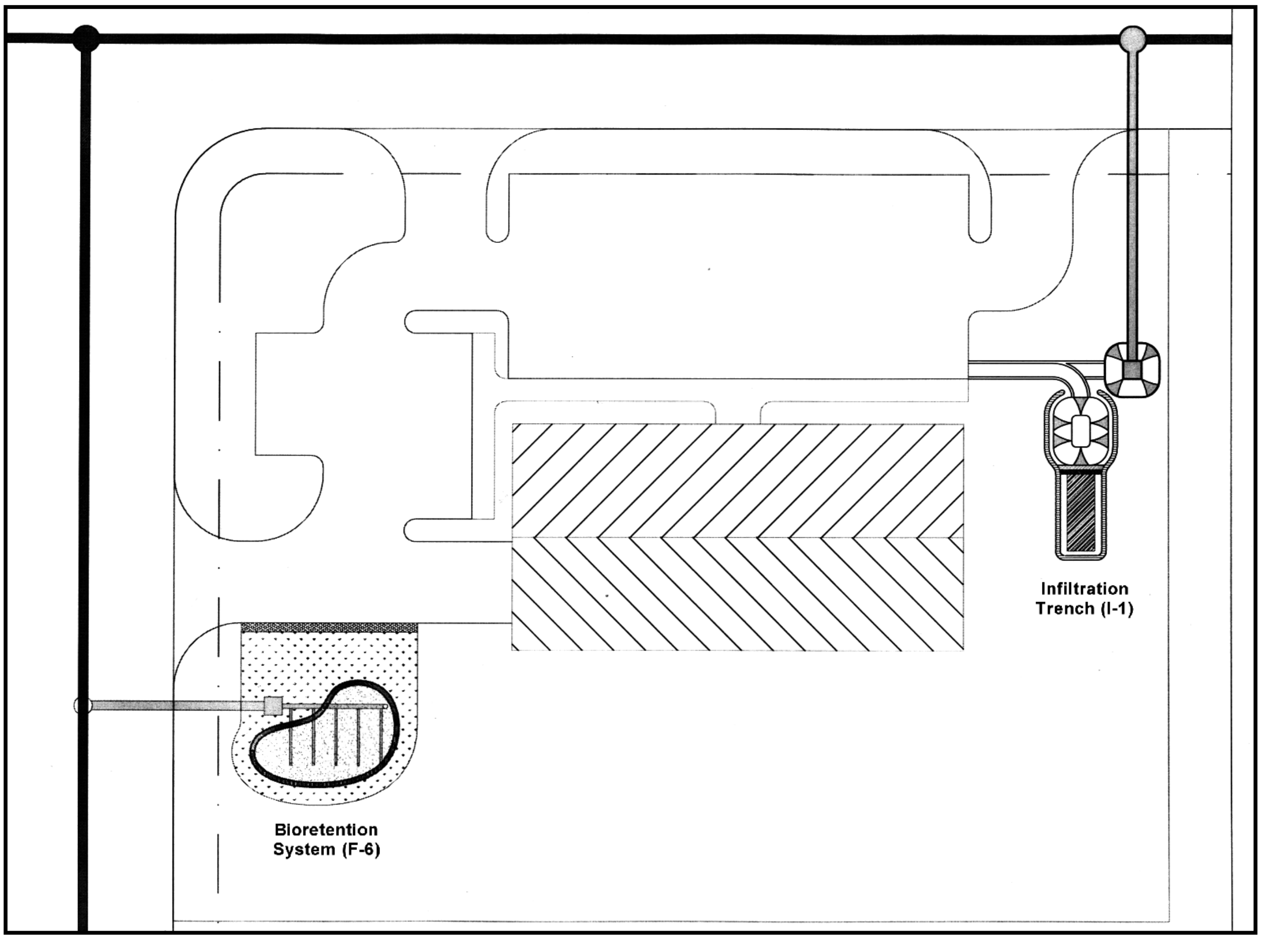
Figure C.2.8 Pocket Sand Filter Design Details



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:

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

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:

$$d = \frac{V}{A_r \times n}$$ where n 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

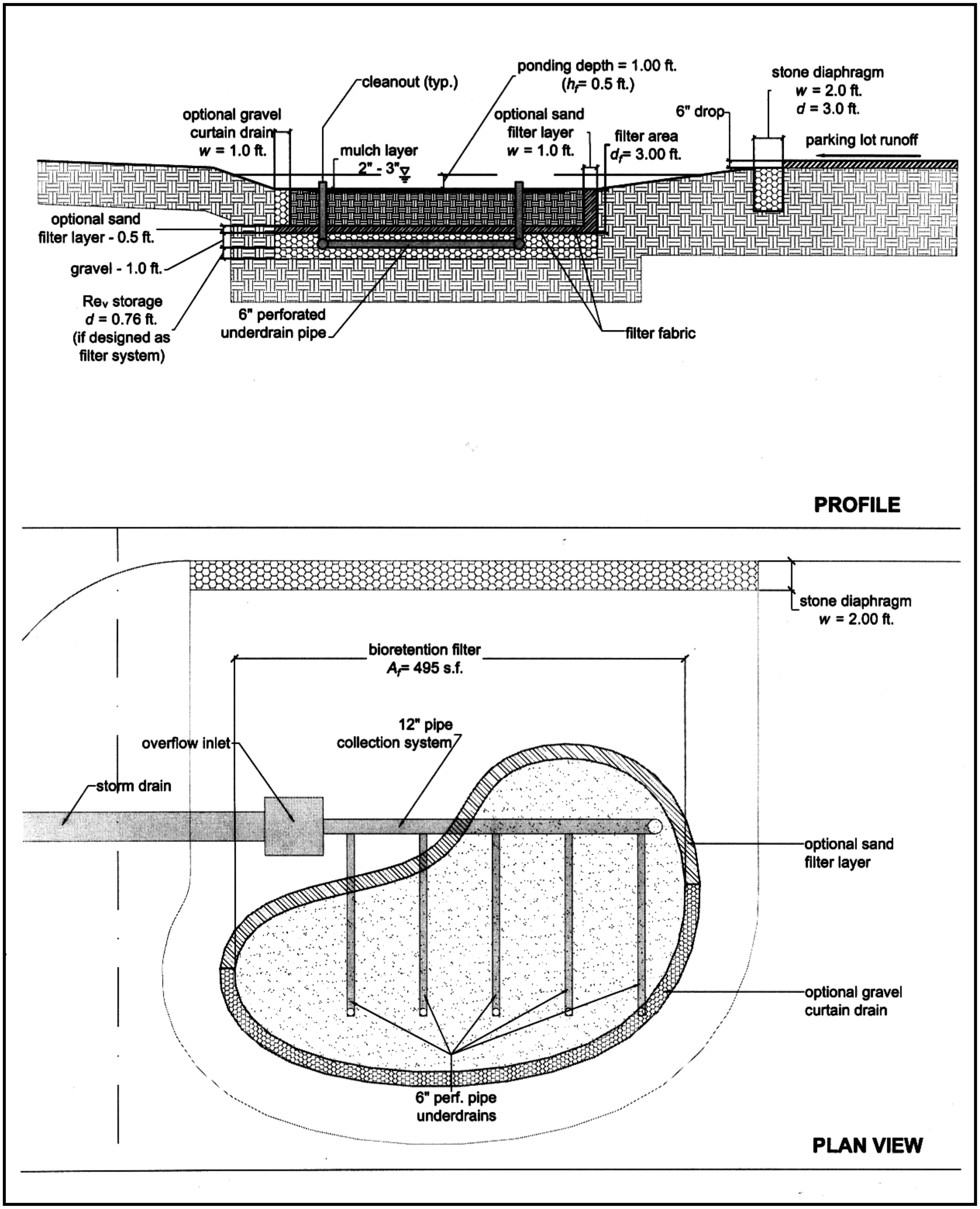
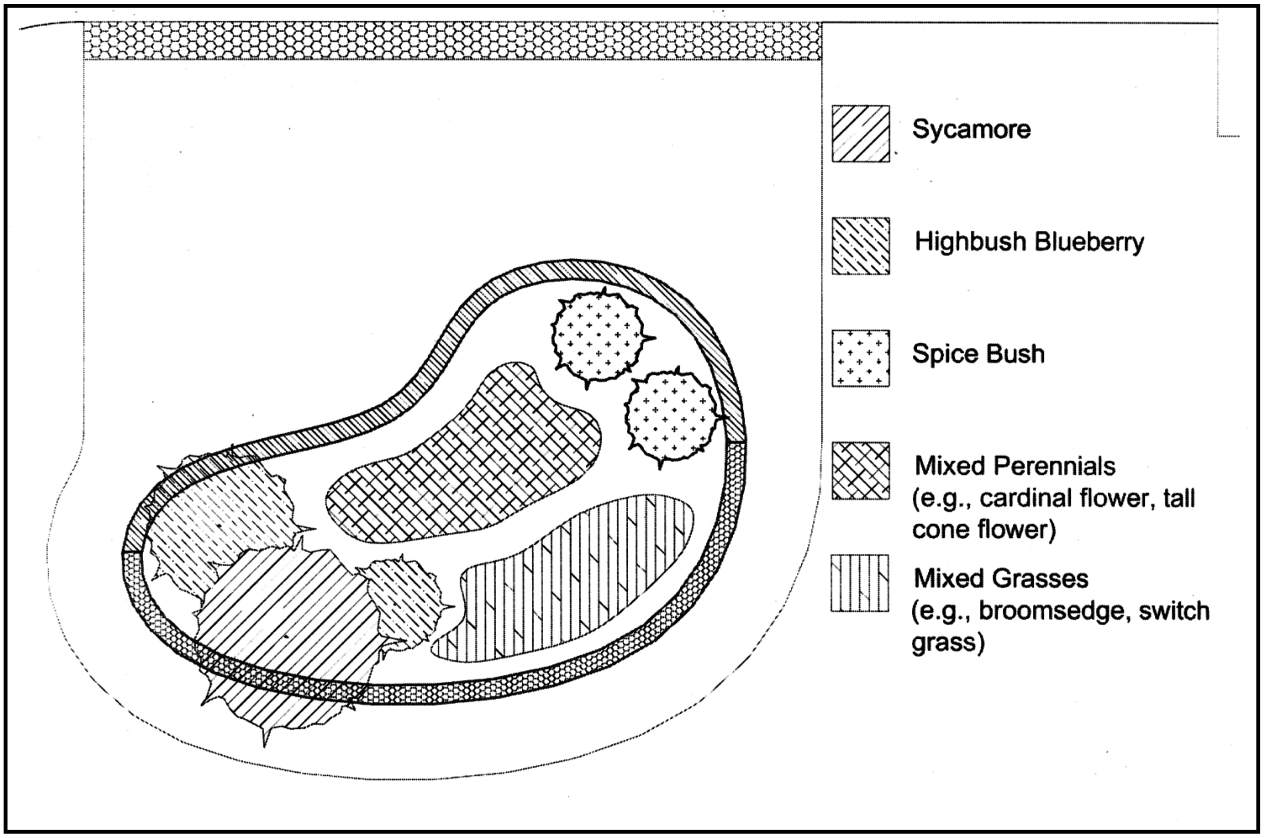


Figure C.2.11 Bioretention Planting Plan



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:

$$V_p = (0.25)(WQ_v)$$
$$= (0.25)(694.8 \text{ cf.})$$
$$= 173.7 \text{ cf.}$$

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 $f = 0.52$ $^{\text{inches}}/_{\text{hour}}$
T_s is the maximum allowable storage of 48 hours
n is the porosity of the stone reservoir, use 0.4

Therefore, d_{max} = $0.52^{\text{inches}}/_{\text{hour}} \times (48 \text{ hours}/0.4) = 62.4$ 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.

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

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

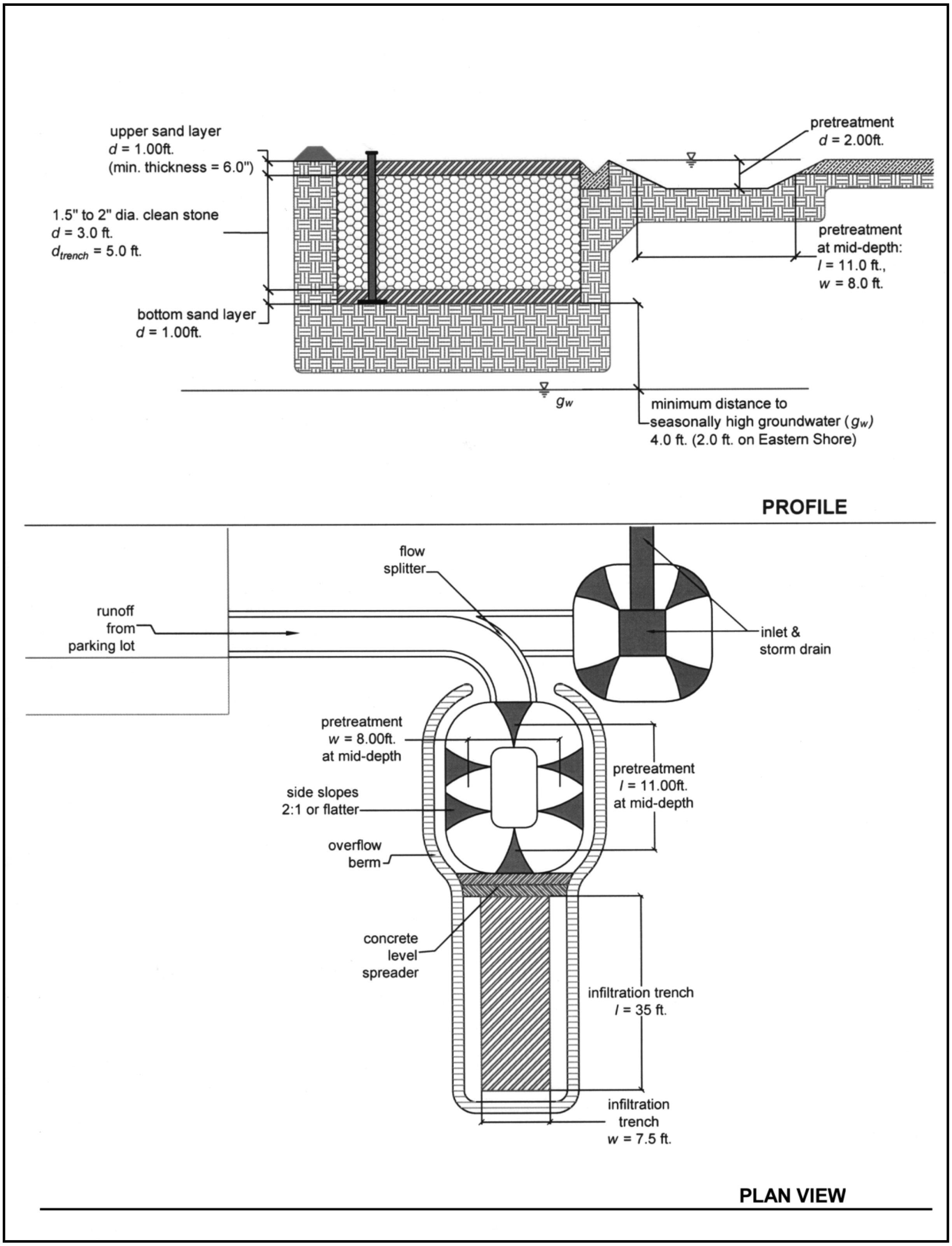
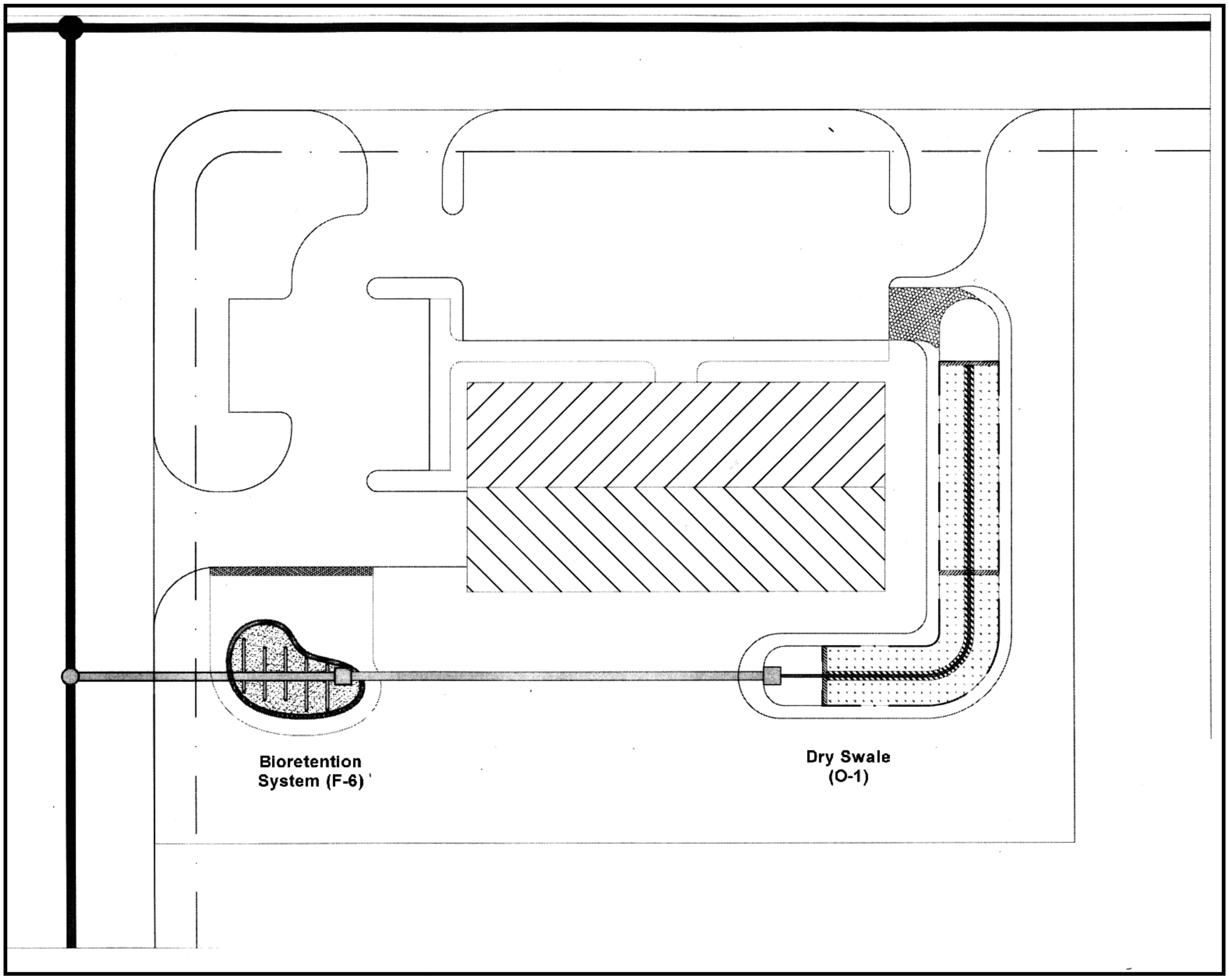


Figure C.2.13 Design Option 3 – Plan View



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-($2 \times z \times d_{mid}$)= 8.3-($2 \times 4 \times 0.75$)=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:

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

Therefore, d=180.6 cf./(100 ft.$\times$2.3 ft.$\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:

$$\begin{aligned} d_c &= d_{max} + d_{10\,yr.\,storm} + d_{freeboard} \\ &= 1.5 \text{ ft.} + 0.1 \text{ ft.} + 0.25 \text{ ft.} \\ &= 1.85 \text{ ft.} \end{aligned}$$

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