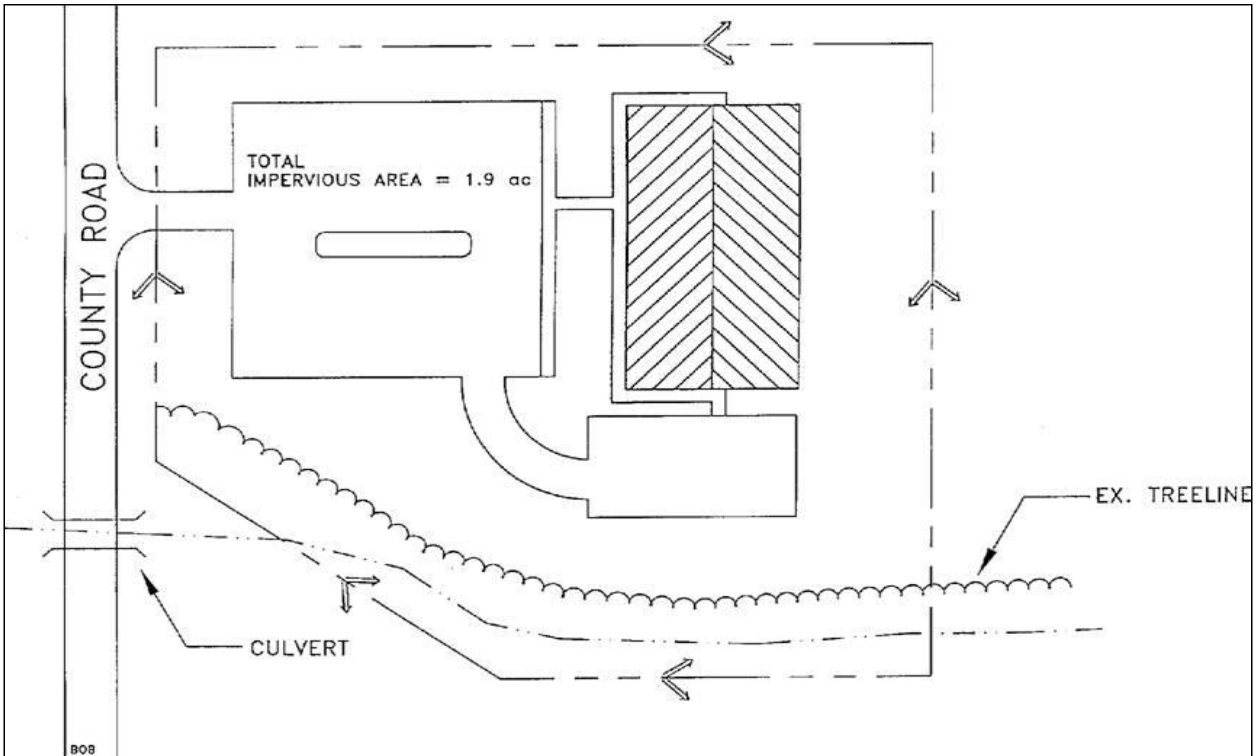
Appendix D-3

Sand Filter Design Example



Base Data	Hydrologic Data		
Site Area = Total Drainage Area (A) = 3.0 ac		Pre	Post
Impervious Area = 1.9 ac; or I = 1.9/3.0 = 63.3%	CN	57	83
Soils Types: “B”	t_c	.36	.15

Figure 1. Georgia Pines Community Center Site Plan

This example focuses on the design of a surface sand filter to meet the water quality treatment requirements of the site. Channel protection and overbank flood control is not addressed in this example other than quantification of preliminary storage volume and peak discharge requirements. It is assumed that the designer can refer to the previous pond example in order to extrapolate the necessary information to determine and design the required storage and outlet structures to meet these criteria. In general, the primary function of sand filters is to provide water quality treatment and not large storm attenuation. As such, flows in excess of the water quality volume are typically routed to bypass the facility. Where quantity control is required, the bypassed flows can be routed to conventional detention basins (or some other facility such as underground storage vaults).

NOTE: This real life example uses the overbank protection volume of Q_{p25}. However, Columbia County requires an overbank protection volume of Q_{p50}.

Computation of Preliminary Stormwater Storage Volumes and Peak Discharges

The layout of the Georgia Pines Community Center is shown in Figure 1.

Step 1 -- Compute runoff control volumes from the Unified Stormwater Sizing Criteria

Compute Water Quality Volume, WQ_v

- Compute Runoff Coefficient, R_v

  $R_v = 0.05 + (63.3)(0.009) = 0.62$

- Compute WQ_v

  $WQ_v = (1.2")(R_v)(A)/12$

  $= (1.2")(0.62)(3.0\ ac)(43{,}560\ ft^2/ac)(1\ ft/12\ in)$

  $= 8{,}102\ ft^3 = 0.186\ ac-ft$

Compute Stream Channel Protection Volume (Cp_v):

For stream channel protection, provide 24 hours of extended detention for the 1-year event.

- Develop Site Hydrologic and Hydrologic Input Parameters and perform Preliminary Hydrologic Calculations

  Per Figures 2 and 3. Note that any hydrologic models using SCS procedures, such as TR-20, HEC-HMS, or HEC-1, can be used to perform preliminary hydrologic calculations.

Condition	CN	Q_{1-year}	Q_{1-year}	$Q_{25-year}$	$Q_{100-year}$
		Inches	*cfs*	*cfs*	*cfs*
Pre-developed	57	0.5	0.6	6.0	9.0
Post-developed	83	1.9	5.5	17.0	22.0

- Utilize modified TR-55 approach to compute channel protection storage volume

  Initial abstraction (Ia) for CN of 83 is 0.41: (TR-55) **[Ia = (200 / CN – 2)]**

  $Ia/P = (0.41)/3.6\ inches = 0.11$

  T_c = 0.15 hours

  q_u = 590 csm/in

  Knowing q_u and T (extended detention time), find q_o/q_i for a Type II rainfall distribution.

  Peak outflow discharge/peak inflow discharge (q_o/q_i) = 0.03

  $Vs/Vr = 0.683 - 1.43(q_o/q_i) + 1.64(q_o/q_i)^2 - 0.804(q_o/q_i)^3$

  Where Vs equals channel protection storage (Cp_v) and Vr equals the volume of runoff in inches.

  Vs/Vr = 0.64

  Therefore, $Vs = Cp_v = 0.64(1.9")(1/12)(3\ ac) = 0.30\ ac-ft = 13{,}068\ ft^3$

- Define the average ED Release Rate

  The above volume, 0.30 ac-ft, is to be released over 24 hours.

  $(0.30\ ac-ft * 43{,}560\ ft^2/ac)/(24\ hrs * 3{,}600\ sec/hr) = 0.15\ cfs$

Determine Overbank Flood Protection Volume (Q_{p25}):

For a Q_{in} of 17 cfs, and an allowable Q_{out} of 6 cfs, the Vs necessary for 25-year control is 0.52 ac-ft or 22,677 ft^3, under a developed CN of 83. Note that 7.9 inches of rain fall during this event, with 5.9 inches of runoff.

Analyze for Safe Passage of 100 Year Design Storm (Q_f):

At final design, prove that discharge conveyance channel is adequate to convey the 100-year event and discharge to receiving waters, or handle it with a peak flow control structure, typically the same one used for the overbank flood protection control.

Table 1 Summary of General Design Information for Georgia Pines Community Center

Symbol	Control Volume	Volume Required (cubic feet)	Notes
WQ_v	Water Quality	8,102	
Cp_v	Channel Protection	13,068	
Q_{p25}	Overbank Flood Protection	22,677	
Q_f	Extreme Flood Protection	NA	Provide safe passage for the 100-year event in final design

Step 2-- Determine if the development site and conditions are appropriate for the use of a surface sand filter.

Site Specific Data

Existing ground elevation at the facility location is 22.0 feet, mean sea level. Soil boring observations reveal that the seasonally high water table is at 13.0 feet. Adjacent creek invert is at 12.0 feet.

Step 3 -- Confirm local design criteria and applicability

There are no additional local criteria that must be met for this design.

PEAK DISCHARGE SUMMARY				
JOB:	Georgia Pines Center			EWB
DRAINAGE AREA NAME:	Pre-Developed Conditions			3-Jan-00
COVER DESCRIPTION	SOIL NAME	GROUP A, B, C or D?	CN FROM TABLE 2.1.5-1	AREA (in acres)
meadow (good condition)		B	58	2.40
meadow (good condition)		B	55	0.60
		Area Subtotals:		3.00
Time of Concentration	Surface Cover	Manning 'n'	Flow Length	Slope
2-yr 24-hr Rainfall = 4.1 in	Cross Section	Wetted Per	Avg Velocity	Tt (Hrs)
Sheet Flow	dense grass	'n' = 0.24	150 Ft.	1.50%
				0.30 Hrs
Shallow Flow	unpaved		500 Ft.	2.00%
			2.28	0.06
Channel Flow				
Total Area in Acres =	3.00	Total Sheet	Total	Total Channel
Weighted CN =	57	Flow =	Flow =	Flow =
Time of Concentration =	0.36 Hrs	0.30 Hrs	0.06 Hrs	0.00 Hrs
Pond Factor =	1	RAINFALL TYPE		
STORM	Precipitation (P) inches	Runoff (Q)	Qp, Peak Discharge	Total Storm Volumes
1 Year	3.6 In.	0.5 In.	0.6 CFS	4,943 cu. ft.
2 Year	4.8 In.	1.0 In.	1.8 CFS	10,887 cu. ft.
5 Year	6.0 In.	1.7 In.	3 CFS	18,252 cu. ft.
10 Year	6.7 In.	2.1 In.	4 CFS	23,186 cu. ft.
25 Year	7.9 In.	2.9 In.	6 CFS	32,076 cu. ft.
50 Year	8.9 In.	3.6 In.	8 CFS	39,672 cu. ft.
100 Year	9.8 In.	4.4 In.	9 CFS	47,613 cu. ft.

Figure 2. Georgia Pines Community Center Pre-Developed Conditions

PEAK DISCHARGE SUMMARY				
JOB:	Georgia Pines Center			EWB
DRAINAGE AREA NAME:	Post-Development Conditions			3-Jan-00
COVER DESCRIPTION	SOIL NAME	GROUP A, B, C or D?	CN FROM TABLE 2.1.5-1	AREA (in acres)
open space (good condition)		B	61	0.50
woods (good condition)		B	55	0.60
impervious		B	98	1.90
			Area Subtotals:	3.00
Time of Concentration	Surface Cover	Manning 'n'	Flow Length	Slope
2-yr 24-hr Rainfall = 4.1 in	Cross Section	Wetted Per	Avg Velocity	Tt (Hrs)
Sheet Flow	short grass	'n' = 0.15	50 Ft.	1.50%
				0.09 Hrs
Shallow Flow	paved		600 Ft.	2.00%
			2.87 F. P. S.	0.06 Hrs
Channel Flow		'n'=0.024	50 Ft.	2.00%
Hydraulic Radius = 0.75	X-S	WP estimated	7.25 F. P. S.	0.00 Hrs
Total Area in Acres =	3.00	Total Sheet Flow = 0.09 Hrs	Total Flow = 0.06 Hrs	Total Channel Flow = 0.00 Hrs
Weighted CN =	83			
Time of Concentration =	0.15 Hrs			
Pond Factor =	1	RAINFALL TYPE		
STORM	Precipitation (P) inches	Runoff (Q)	Qp, Peak Discharge	Total Storm Volumes
1 Year	3.6 In.	1.9 In.	5.5 CFS	21,159 cu. ft.
2 Year	4.8 In.	3.0 In.	8.6 CFS	32,602 cu. ft.
5 Year	6.0 In.	4.1 In.	12 CFS	44,555 cu. ft.
10 Year	6.7 In.	4.8 In.	14 CFS	51,881 cu. ft.
25 Year	7.9 In.	5.9 In.	17 CFS	64,262 cu. ft.
50 Year	8.9 In.	6.8 In.	20 CFS	74,281 cu. ft.
100 Year	9.8 In.	7.7 In.	22 CFS	84,372 cu. ft.

Figure 3. Georgia Pines Community Center Post-Developed Conditions

Step 4 -- Compute WQ_v peak discharge (Q_{wq}) & Head

- Water Quality Volume:

  WQ_v previously determined to be 8,102 cubic feet.

- Determine available head (See Figure 4)

  Low point at parking lot is 23.5. Subtract 2' to pass Q_{25} discharge (21.5) and a half foot for channel to facility (21.0). Low point at stream invert is 12.0. Set outfall underdrain pipe 2' above stream invert and add 0.5' to this value for drain (14.5). Add to this value 8" for the gravel blanket over the underdrains, and 18" for the sand bed (16.67). The total available head is 21.0 - 16.67 or 4.33 feet. Therefore, the average depth, h_f, is $\mathbf{(h_f) = 4.33' / 2}$, and $\mathbf{h_f = 2.17'}$.

  The peak rate of discharge for the water quality design storm is needed for the sizing of off-line diversion structures, such as sand filters and grass channels. Conventional SCS methods have been found to underestimate the volume and rate of runoff for rainfall events less than 2". This discrepancy in estimating runoff and discharge rates can lead to situations where a significant amount of runoff by-passes the filtering treatment practice due to an inadequately sized diversion structure and leads to the design of undersized bypass channels.

  The following procedure can be used to estimate peak discharges for small storm events. It relies on the volume of runoff computed using the Small Storm Hydrology Method (Pitt, 1994) and utilizes the NRCS, TR-55 Graphical Peak Discharge Method (USDA, 1986). A brief description of the calculation procedure is presented below.

- Using the water quality volume (WQ_v), a corresponding Curve Number (CN) is computed utilizing the following equation:

  $$\mathbf{CN = 1000/[10 + 5P + 10Q - 10(Q^2 + 1.25\,QP)^{1/2}]}$$

  where P = rainfall, in inches (use 1.2" for the Water Quality Storm)
  and Q = runoff volume, in inches (equal to WQ_v / area)

- Once a CN is computed, the time of concentration (t_c) is computed

- Using the computed CN, t_c and drainage area (A), in acres; the peak discharge (Q_{wq}) for the Water Quality Storm is computed (based on the procedures identified in Section 2.1 (either Type II or Type III in the State of Georgia).

  - Read initial abstraction (I_a), compute I_a/P
  - Read the unit peak discharge (q_u) for appropriate t_c
  - Using the water quality volume (WQ_v), compute the water quality peak discharge (Q_{wq})

    $$\mathbf{Q_{wq} = q_u * A * WQ_v}$$

    where: Q_{wq} = the peak discharge, in cfs
    q_u = the unit peak discharge, in cfs/mi^2/inch
    A = drainage area, in square miles
    WQ_v = Water Quality Volume, in watershed inches

For this example, the steps are as follows:

Compute modified CN for 1.2" rainfall

P = 1.2"

$$\mathbf{Q = WQ_v / area = (8{,}102\ ft^3 / 3\ ac / 43{,}560\ ft^2 / ac * 12\ in / ft) = 0.74"}$$

$$\mathbf{CN = 1000 / [10 + 5P + 10Q - 10(Q^2 + 1.25 * Q * P)^{1/2}]}$$

$$\mathbf{= 1000 / [10 + 5 * 1.2 + 10 * 0.74 - 10(0.74^2 + 1.25 * 0.74 * 1.2)^{1/2}]}$$

$$\mathbf{= 95.01}$$

Use CN = 95

For CN = 95 and the T_c = 0.15 hours, compute the Q_p for a 1.2" storm. With the CN = 95, a 1.2" storm will produce 0.74" of runoff. I_a = 0.105, therefore $\mathbf{I_a / P = 0.105 / 1.2 = 0.088.}$ From Section 2.1, q_u = 625 csm/in, and therefore $\mathbf{Q_{wq} = (625\ csm/in)(3.0\ ac/640\ ac/\ sq\ mi)(0.74") = 2.2\ cfs.}$

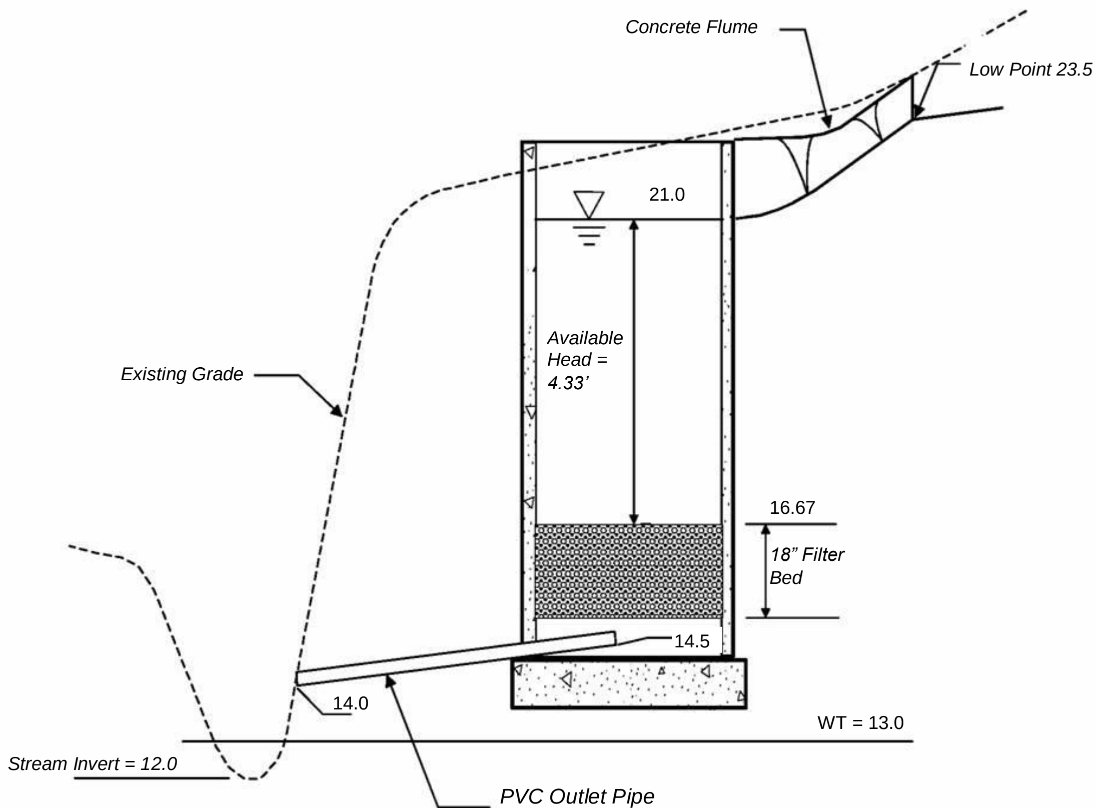


Figure 4. Available Head Diagram

Step 5 -- Size flow diversion structure (see Figure 5):

Size a low flow orifice to pass 2.2 cfs with 1.5' of head using the Orifice equation.

$$\mathbf{Q = CA(2gh)^{1/2};\ 2.2\ cfs == (0.6)(A)[(2)(32.2\ ft/s^2)(1.5')]^{1/2}}$$

$$\mathbf{A = 0.37\ sq\ ft = \pi d^2/4:\quad d = 0.7'\ or\ 8.5";}\ \underline{\text{use 9 inches}}$$

Size the 25-year overflow as follows: the 25-year wsel is set at 23.0. Use a concrete weir to pass the 25-year flow (17.0 cfs) into a grassed overflow channel using the Weir equation. Assume 2' of head to pass this event. Overflow channel should be designed to provide sufficient energy dissipation (e.g., riprap, plunge pool, etc.) so that there will be non-erosive velocities.

$$\mathbf{Q = CLH^{3/2}}$$

$$\mathbf{17 = 3.1\,(L)\,(2')^{1.5}}$$

L = 1.94'; use L = 2'-0" which sets flow diversion chamber dimension.

Weir wall elev. = 21.0. Set low flow invert at $\mathbf{21.0 - [1.5' +)0.5 * 9" * 1\ ft/12")] = 19.13.}$

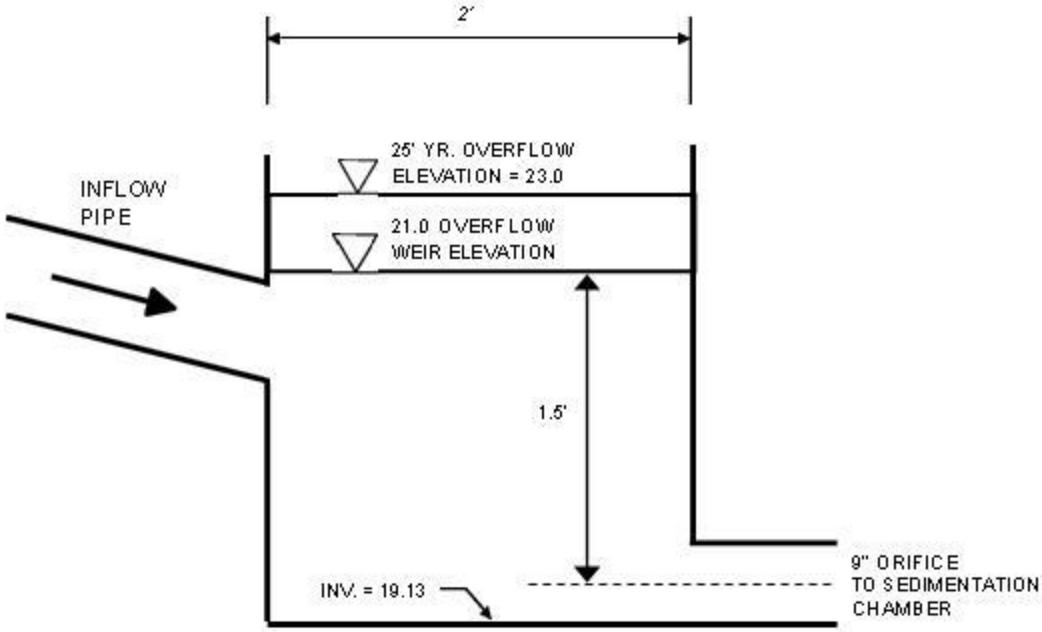


Figure 5. Flow Diversion Structure

Step 6 -- Size filtration bed chamber (see Figure 6):

From Darcy's Law: $\mathbf{Af = (WQ_v)(d_f)/[k(h_f + d_f)(t_f)]}$

where: d_f = 18"
k = 3.5 ft/day
h_f = 2.17'
t_f = 40 hours

$$\mathbf{A_f = (8{,}102\ ft^3)(1.5')/[3.5(2.17'+1.5')(40\ hr/24\ hr/day)]}$$

$\mathbf{A_f}$ = **567.7 sq ft**; using a 2:1 ratio, say filter is 17' by 34' (= 578 sq ft)

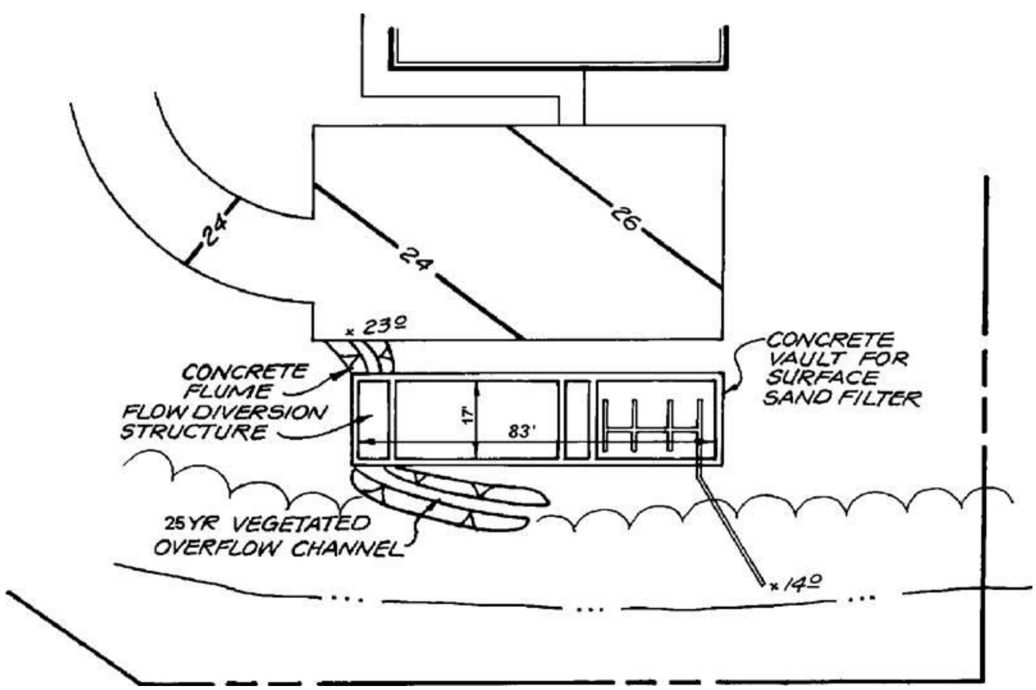


Figure 6. Surface Sand Filter Site Plan

Step 7 -- Size sedimentation chamber

From Camp-Hazen equation, for **I** < 75%: $A_s = 0.066\ (WQ_v)$

$$\mathbf{A_s = 0.066\ (8{,}102\ ft^3)\ or\ \underline{535\ sq\ ft}}$$

Given a width of 17 feet, the length will be 535'/17' or 31.5 feet (use 17' * 32')

Step 8 -- Compute V_{min}

$$\mathbf{V_{min} = \tfrac{3}{4}(WQ_v)\ or\ 0.75\ (8{,}102\ ft^3) = \underline{6{,}077\ ft^3}}$$

Step 9 -- Compute storage volumes within entire facility and sedimentation chamber orifice size:

Volume within filter bed (V_f): $\mathbf{V_f = A_f (d_f)(n)}$; n = 0.4 for sand

$$\mathbf{V_f = (578\ sq\ ft)(1.5')(0.4) = \underline{347\ ft^3}}$$

Temporary storage above filter bed ($V_{f\text{-}temp}$): $\mathbf{V_{f-temp} = 2h_f A_f}$

$$\mathbf{V_{f-temp} = 2\ (2.17')\ (578\ sq\ ft) = \underline{2{,}509\ ft^3}}$$

Compute remaining volume for sedimentation chamber (V_s):

$$\mathbf{V_s = V_{min} - [V_f + V_{f-temp}]\ or\ 6{,}077 - [347 + 2509] = \underline{3{,}221\,ft^3}}$$

Compute height in sedimentation chamber (h_s): $\mathbf{h_s = V_s / A_s}$

(3,221 cubic ft) / (17' x 32') = 5.9' which is larger than the head available (4.33'); increase the size of the settling chamber, using 4.33' as the design height;

(3,221 cubic ft) / 4.33' = 744 sq ft; 744'/17' yields a length of 43.8 feet (say 44')

New sedimentation chamber dimensions are 17' by 44'

With adequate preparation of the bottom of the settling chamber (rototil earth, place gravel, then surge stone), the bottom can infiltrate water into the substrate. The runoff will enter the groundwater directly without treatment. The stone will eventually clog without protection from settling solids, so use a removable geotextile to facilitate maintenance. Note that there is 2.17' of freeboard between bottom of recharge filter and water table.

Provide perforated standpipe with orifice sized to release volume (within sedimentation basin) over a 24 hr period (see Figure 7). Average release rate equals $\mathbf{3{,}221\,ft^3 / 24\ hr = 0.04\ cfs}$.

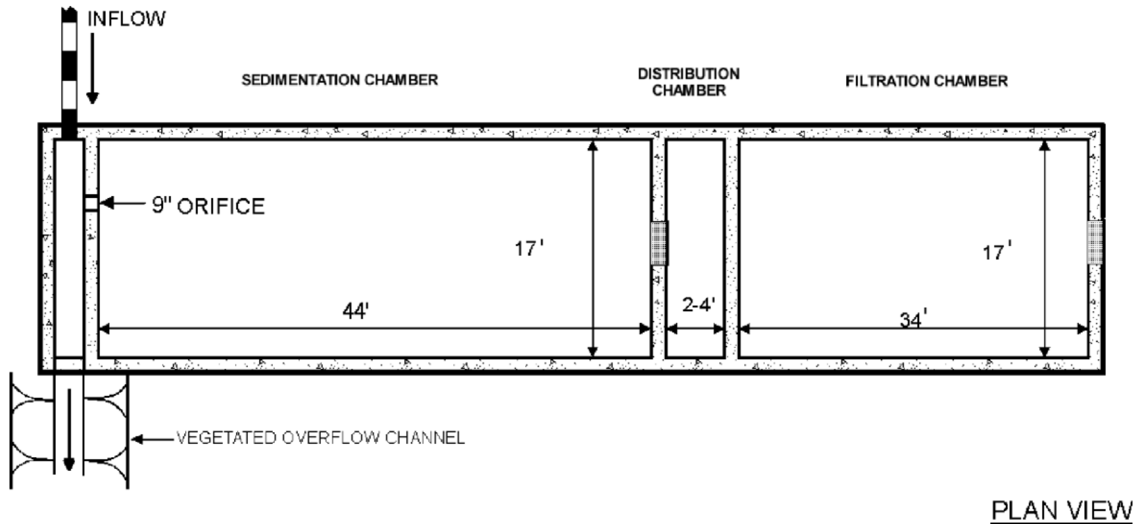
Equivalent orifice size can be calculated using orifice equation:

$\mathbf{Q = CA(2gh)^{1/2}}$**,** where h is average head, or $\mathbf{4.33' / 2 = 2.17'}$**.**

$$\mathbf{0.04\ cfs = 0.6 * A * (2 * 32.2\ ft/s^2 * 2.17\ ft)^{1/2}}$$

$\mathbf{A = 0.005\ ft^2 = \pi D^2 / 4:}$ therefore equivalent orifice diameter equals 1".

Recommended design is to cap stand pipe with low flow orifice sized for 24-hour detention. Over-perforate pipe by a safety factor of 10 to account for clogging. Note that the size and number of perforations will depend on the release rate needed to achieve 24-hour detention. A multiple orifice stage-discharge relation needs to be developed for the proposed perforation configuration. Stand pipe should discharge into a flow distribution chamber prior to filter bed. Distribution chamber should be between 2 and 4 feet in length and same width as filter bed. Flow distribution to the filter bed can be achieved either with a weir or multiple orifices at constant elevation. See Figure 8 for stand pipe details.



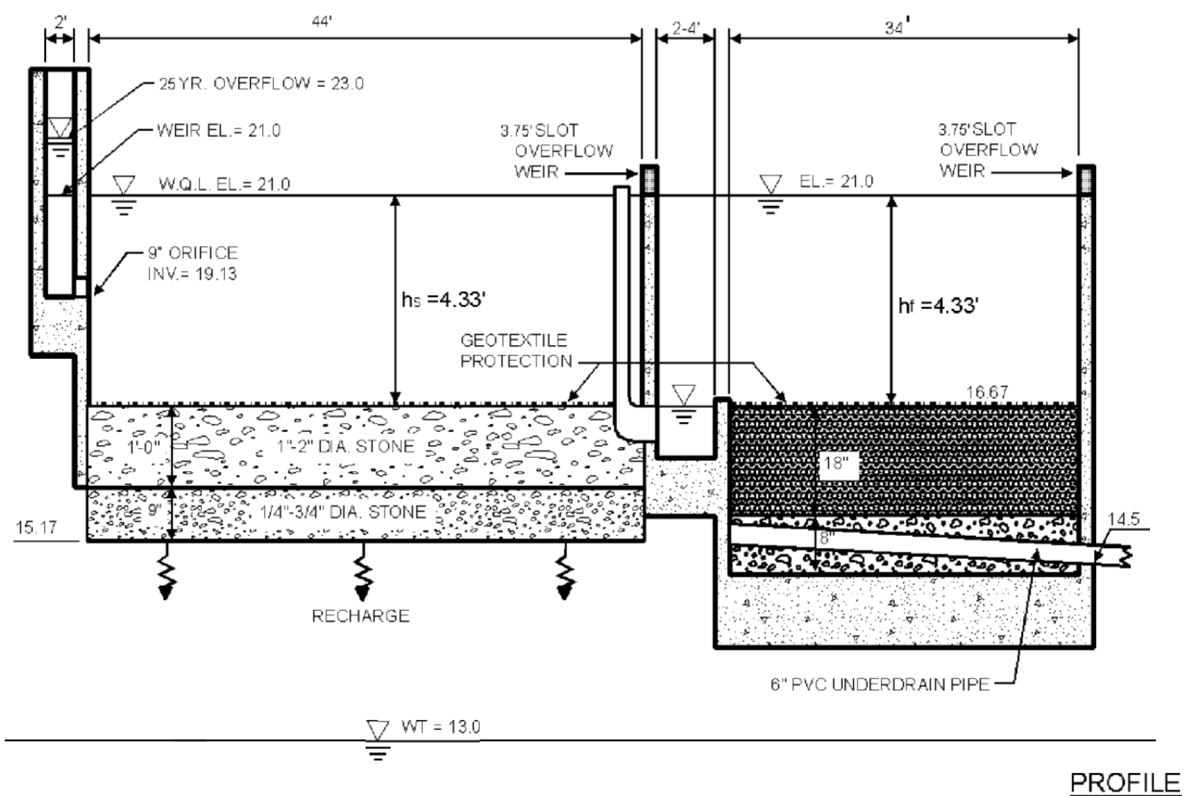


Figure 7. Plan and Profile of Surface Sand Filter

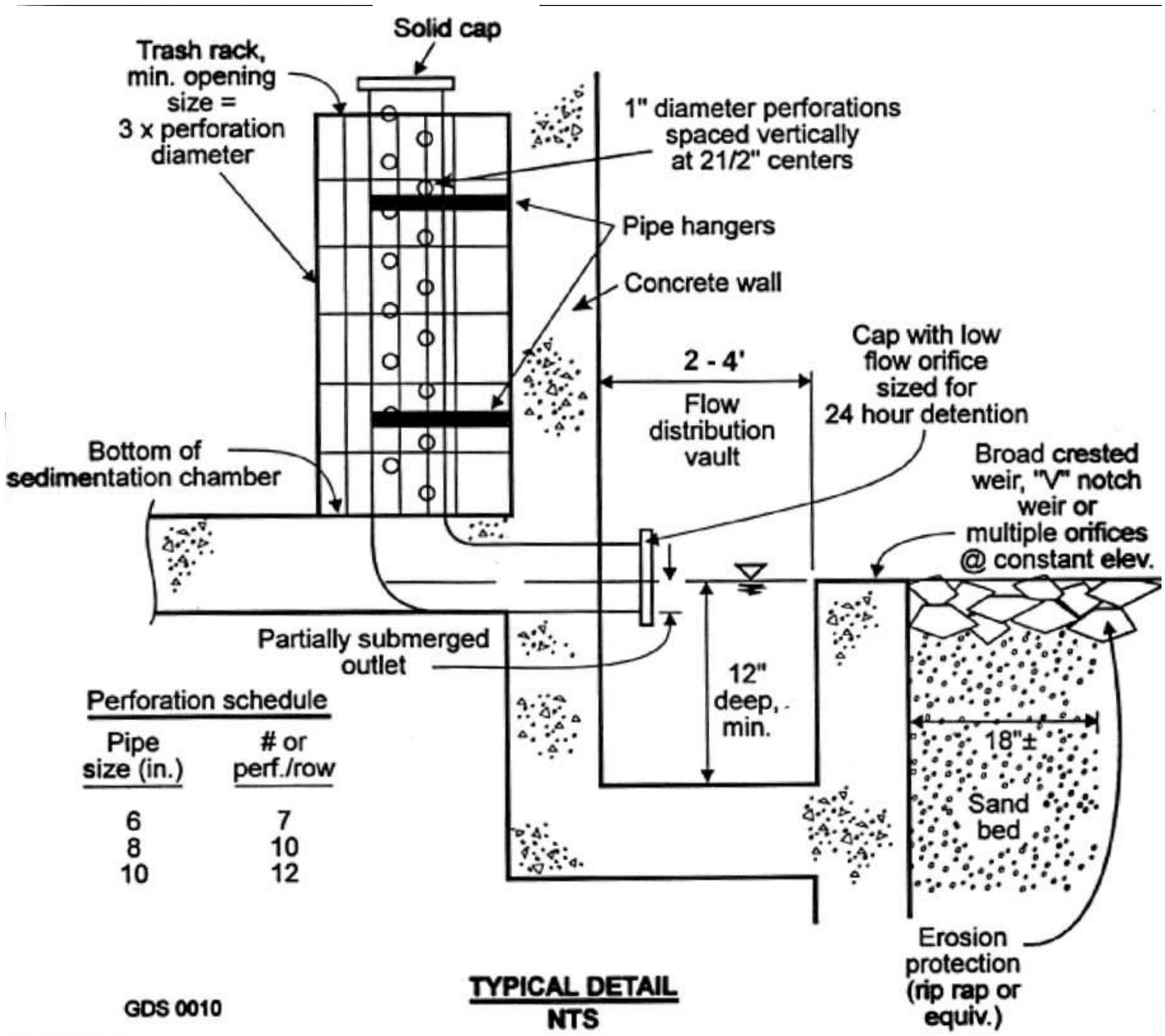


Figure 8. Perforated Stand Pipe Detail

Step 10 -- Design inlets, pretreatment facilities, underdrain system, and outlet structures.

Step 11 -- Compute overflow weir sizes

Assume overflow that needs to be handled is equivalent to the 9" orifice discharge under a head of 3.5 ft (i.e., the head in the diversion chamber associated with the 25-year peak discharge).

$$\mathbf{Q = CA(2gh)^{1/2}}$$

$$\mathbf{Q = 0.6\ (0.44\ ft^2)[(2)(32.2\ ft/s^2)(3.5\ ft)]^{1/2}}$$

$$\mathbf{Q = 3.96\ cfs,\ say\ 4.0\ cfs}$$

For the overflow from the sediment chamber to the filter bed, size to pass 4 cfs.

Weir equation: $\mathbf{Q = CLH^{3/2}}$, assume a maximum allowable head of 0.5'

$$\mathbf{4.0 = 3.1 * L * (0.5\ ft)^{3/2}}$$

$$\mathbf{L = 3.65\ ft,\ \underline{Use\ L = 3.75\ ft.}}$$

Similarly, for the overflow from the filtration chamber to the outlet of the facility, size to pass 4.0 cfs.

Weir equation: $\mathbf{Q = CLH^{3/2}}$, assume a maximum allowable head of 0.5'

$$\mathbf{4.0 = 3.1 * L * (0.5\ ft)^{3/2}}$$

$$\mathbf{L = 3.65\ ft,\ \underline{Use\ L = 3.75\ ft.}}$$

Adequate outlet protection and energy dissipation (e.g., riprap, plunge pool, etc.) should be provided for the downstream overflow channel.