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Development of Best Management Practice Design Guidance for Roadway Applications in Nebraska

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DEVELOPMENT OF BEST MANAGEMENT PRACTICE DESIGN GUIDANCE

FOR ROADWAY APPLICATIONS IN NEBRASKA

By

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Runoff from roadways carries pollutants which may be detrimental to aquatic ecosystems. The primary pollutants of concern for roadway runoff are solids and heavy metals, particularly cadmium, copper, and zinc. Roadway runoff falls under the legislation of the Clean Water Act (CWA) via the National Pollutant Discharge Elimination System (NPDES). CWA regulates discharge of nonpoint source pollutants, such as roadway runoff, by issuing permits to public entities which manage Municipal Separate Storm Sewer Systems (MS4s). Part of the Nebraska Department of Roads (NDOR) permitting requirement is to create a design guide for Best Management Practices (BMPs) tailored to remediate roadway runoff in Nebraska, which this document is intended to fulfill.

BMPs which are most applicable to treating roadway runoff are those which can remove 80% of the total solid load in the runoff, reduce metal concentrations to below acute toxicity levels, have low maintenance burden, are cost effective, do not pose a safety hazard to motorists, can be implemented within the right-of-way, do not negatively impact the road subgrade, and are aesthetically pleasing. The BMPs which best fit these criteria are vegetated filter strips, vegetated swales, bioretention, sand filters, and horizontal filter trenches. In this study fact sheets and design guides have been compiled for each of these BMPs. The fact sheet provides background on the BMP including cost considerations, siting constraints, and predicted maintenance requirements. The design guide provides the process for sizing the BMP, design criteria the BMP must meet, and a design example which goes through the design process for a hypothetical application.

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Section 1 Introduction

1.1 History of Stormwater Regulation

As water quality regulations have developed, a greater focus has been put on remediating pollutants associated with stormwater runoff. The main piece of surface water quality legislation is the Clean Water Act (CWA). The CWA was originally passed in 1972 as the Federal Water Pollution Control Amendments of 1972, and became known as the CWA after 1977 amendments were made. The goal of the CWA was to "Restore and maintain the chemical, physical, and biological integrity of the nation's waters" (CWA 1977a). The original CWA was implemented to regulate discharges into navigable waters from discrete point sources. Point sources were considered to be any "pipe, ditch, channel, tunnel, conduit, well, discrete fissure, container, rolling stock, concentrated animal feeding operation, or vessel or float craft" (Clean Water Act 1977a). Regulations were based on effluent limitations which were enforced through permitting in the National Pollutant Discharge Elimination System (NPDES). Permitting for a facility included conditions for effluents limitation, monitoring, operation and maintenance, upset and bypass provisions, record keeping, and inspections.

Although stormwater runoff, known as non-point source pollution, is conveyed through measures which are considered point sources (i.e., ditches, pipes, or channels), they were not regulated until the 1987 amendments to the CWA which included them into the NPDES (CWA 1977b). Runoff under NPDES is regulated for construction and post-construction considerations. Highway construction and operation permits are regulated under NPDES due to the build-up of pollutants associated with automobiles and wear of the driving surface. However, not all highway systems currently require permitting. Permits are only required where the roadway discharges into a Municipal Separate Storm Sewer System (MS4), which have discrete outfalls to receiving waters and are located in urban areas (NDOR 2010). Storm Water Management Plans are developed for MS4 permits which feature the following 6 minimum Best Management

Practice (BMP) programs: public education and outreach, public participation and involvement, illicit discharge detection and elimination, construction site runoff control, post-construction site runoff control, and pollution prevention and good house-keeping (CWA 1977a). The permitting does not include effluent limitations, but does stipulate that the above 6 minimum BMP programs should be instituted to remediate runoff to the maximum extent practicable (CWA 1977a). BMPs can be broadly categorized as structural or non-structural. Structural BMPs actively remove pollutants from runoff while non-structural are generally related to source control.

1.2 Pollutants Discharging from Roadways

Knowing which pollutants are present in runoff from the roadway is essential to remedial efforts. Table 1 shows the primary constituents of runoff from Interstate 80 near the 108th street crossing in Omaha, Nebraska which had been sampled between 2009 and 2011 for the Nebraska Department of Roads (NDOR) (Torres 2010). Many of the contaminants are innocuous in themselves or in such low concentrations that they will not impact the ecosystem.

Table 1: Roadway pollutants (Torres 2010)

Calcium	**Total Kjeldahl Nitrogen**	**Total Phosphorus**	**Lead**
Magnesium	**Total Dissolved Solids**	**Nitrate**	**Mercury**
Potassium	**Total Suspended Solids**	**Nitrite**	**Nickel**
Sodium	**Total Solids**	**Phosphate**	**Oil and Grease**
Cadmium	**Volatile Dissolved Solids**	**Sulfate**	**COD**
Chromium	**Volatile Suspended Solids**	**Zinc**	**Soluble Phosphate**
Copper	**Total Volatile Solids**	**Silica**	**Chloride**
Iron	**Alkalinity as CaCO3**	**Bromide**	**Fluoride**

Pollutants of concern found in the Nebraska study are metals, total solids, dissolved solids (in the form of sodium), and diesel and gasoline constituents (Torres 2010). Although nutrients such as nitrogen and phosphorous were found they were not at high enough concentrations to adversely impact receiving waters (Torres 2010). Sampling found high concentrations of total extractable hydrocarbons (TEH). The TEH are generally compounds with molar weights consistent with gasoline or diesel. There is no toxicity data available for the TEH, so it is not generally classified as a chemical of environmental concern (Torres 2010).

It would be ideal to establish primary pollutants for each site where BMPs are being considered. However, this is not always feasible. The data collected during the NDOR study will, therefore, be considered to be characteristic of runoff contamination across the state. Metals and solids will be the pollutants the BMPs will be designed to remediate.

Section 2 Objectives

The objective of this work is to assemble a set of design guides of Best Management Practices (BMPs) tailored to treating runoff from roadways. This will be accomplished by identifying which BMPs are applicable to roadside scenarios, compiling fact sheets on the applicable BMPs, and establishing the design processes for the selected BMPs. The fact sheets are to be consulted in order to determine which BMP is best for site-specific conditions. Then the design guide for that BMP will be used to ensure the selected BMP will function properly. This work has been developed to comply with requirements for a NPDES permit for the MS4 servicing highways in Nebraska.

Section 3 Literature Review

3.1 Historical Perspective

The forerunner of BMPs came with the Soil Conservation Act of 1935. This Act was enacted to counter the soil erosion of the dust bowl era and spawned a Soil Conservation District movement (Ice 2004). Although this Act did not directly regulate discharges to water, it did begin legislation directed towards protecting environmental resources.

In 1949 the *Yearbook in Agriculture* published the article "Watersheds and How to Care for Them." This article stressed the importance of maintaining the land and streams, which would allow them to continue to be usable. It called for the implementation of better land practices to protect receiving waters and prevent erosion, much like BMPs are used today (Ice 2004).

The watershed approach from 1949 can be seen mirrored in modern Total Maximum Daily Load (TMDL) requirements. TMDLs are the acceptable loading of a given compound in a water-body which is considered safe for the intend use of the water-body. Non-point source pollution has been ruled in the case of *Pronsolino v Nastri* to be considered in the TMDL (Ice 2004). This ruling furthered the need for BMPs on a watershed scale.

One of the first BMPs to be developed and rigorously studied was the surface sand filter by the city of Austin, Texas (Landphair et al. 2000).

3.2 Expected Quality of Runoff From Roadways

Pollutant concentrations can vary widely. Table 2 shows a comparison of observed pollutant concentrations coming from roadways. These results show a significant variation on a site-to-site basis. Runoff from roadways is generally low in nitrogen and phosphorous concentrations, but may contain excessive amounts of solids, metals, or oil and grease.

Table 2: Comparison of contaminant concentrations in roadway runoff (Torres 2010)

Analyte		Wu et al. 1998			Kayhanian et al. 2007			Barret et al. 1998			Driscoll et al. 1990		
		Monitoring Site I			Range	Median	Mean	35th Street			National Highway Runoff Report		
		Range	Median	Mean				Range	Median	Mean	Range	Median	Mean
Cu	(µg/L)	9.0–52	15	24.2	1.1–130	14.9	10.2	2.0–120	34	38	5–155	52	39
Cd	(µg/L)	<DL	<DL	<DL	0.2–8.4	0.24	0.13	-	-	-	-	-	-
Cr	(µg/L)	5.0–20	6.5	8.1	1.0–23	3.3	2.2	-	-	-	-	-	-
Pb	(µg/L)	7.0–56	15	21	1.0–480	7.6	1.2	7–440	50	99	11–1457	525	234
Fe	(µg/L)	-	-	-	32–3310	378	150	300–10000	2606	3537	-	-	-
Ni	(µg/L)	9.0–17	9	8.1	1.1–40	4.9	3.4	-	-	-	-	-	-
Zn	(µg/L)	-	-	-	3–1017	68.8	40.4	34–590	208	237	40–2892	368	217
TDS	(mg/L)	61–577	107	157	3.7–1800	87.3	60.3	-	-		-	-	-
TSS	(mg/L)	32–771	215	283	1–2988	112.7	59.1	33–914	131	202	9–406	143	93
COD	(mg/L)	4–177	48	70	-	-	-	18–464	126	149	41–291	103	84
NO_3^-+ NO_2^-	(mg/L)	0.08–13.37	0.38	2.25	0.01–4.8	1.07	0.6	0.0–3.66	1.03	1.25	0.19–3.32	0.84	0.66
TKN	(mg/L)	0.76–.45	1	1.42	0.1–17.7	2.06	1.4	-	-	-	0.38–3.51	1.79	1.48
Ortho P	(mg/L)	0.01–0.74	0.08	0.15	0.01–2.4	0.11	0.06	-	-	-	-	-	-
Total P	(mg/L)	0.04–1.54	0.2	0.43	0.03–4.69	0.29	0.18	0.07–1.09	0.33	0.42	-	-	-
O&G	(mg/L)	1.0–11	3.3	4.4	1–20	6.6	6	0.8–35.1	4.1	6.5	-	-	-

3.2.1 Sources of Pollutants

Roadway pollutants are associated with wear and maintenance of the roadway surface, vehicle operation, and atmospheric loading. Wear of the surface creates particulates which are then washed off the road by rainfall. Vehicles can deposit heavy metals, oil/grease, poly-aromatic hydrocarbons (PAHs), petroleum hydrocarbons, benzene, toluene, ethyl benzene, xylene (BTEX), as well as debris from careless drivers throwing trash out as they drive (MSSC 2008; Nixon & Saphores 2007).

Vehicles also discharge contaminants into the atmosphere which then settle back onto roads. These pollutants include heavy metals, dust, and PAHs (Barrett et al. 1995). Atmospheric loading is also shown to deposit nutrients, accounting for as much as 90% of the nitrogen loading (Wu et al. 1998).

Deicing considerations must be taken in winter months in order to keep roadways running efficiently (MSSC 2005). Deicing salts add high levels of sodium and chloride to runoff as well as adding suspended solids. These loads can be reduced by employing more benign salts, such as Calcium Magnesium Acetate and Potassium Acetate, which will have less negative environmental impacts (FHA 1997a). Vegetated systems adjacent to roadways may also be negatively affected by road salt (Barrett et al. 1995). When the roadside vegetative cover decreases it promotes channelization of runoff causing erosion, which adds to particulate loading (FHA 2002a). Sand added for traction during the winter also contributes to particulate loading (MSSC 2005).

3.2.2 Factors Affecting Pollutant Loads

Pollutant concentrations and constituents vary with season, time between runoff events, road usage, and within individual events. Average daily traffic has been found to result in higher concentrations of some pollutants in urban area and higher concentrations of other pollutants in rural (Kayhanian et al. 2003). Urban areas have been shown to have high metals and solids but low nutrient loads (Flint and Davis 2007) while rural areas may have higher nutrient loads from agricultural practices. This finding implies there are factors besides traffic volume affecting pollutant concentration.

Required roadway maintenance can cause pollutant fluctuation with the seasons (Barrett et al. 1995). Table 3 lists necessary maintenance practices which may impact receiving waters (Kramme et al. 1985). The potential for these activities to adversely affect water quality increases with proximity to the receiving water (Barrett et al. 1995).

Table 3: Maintenance activity which may contribute to highway contamination (Kramme et al. 1985)

Activities with Probable Impact	Activities with Possible Impact
Repairing slopes, slips, and slides	Full depth repairs
Cleaning ditches, channels and drainage structures	Surface treatments
Repairing drainage structures	Blading and repairing unpaved berms and /or ditches
Bridge painting	Bridge surface cleaning
Subsurface repair	Bridge deck repairs
Chemical vegetation control	Mowing
	Planting or care of shrubs, plants, and trees
	Seeding, sodding, and fertilizing
	Application of abrasives
	Care of rest areas
	Washing and cleaning maintenance equipment
	Bulk storage of motor fuels
	Disposal of used lubricating oil

There also may be concentration differences between wet and dry periods due to time between runoff events (Lee et al. 2004). Periods with little rain allow pollutants to build-up on roadways creating higher loads when the accumulated pollutants are subsequently washed away.

Although traffic volume, antecedent dry period, rainfall intensity, and rainfall depth have been demonstrated to affect pollutant loading and concentration, that is not always the case. Multiple studies have shown weak correlations to these factors (Desta et al. 2007; Torres 2010). Site-specific sampling is required to get an accurate prediction for contamination loads.

However, this may not be cost effective, and the wide variation within an individual site, as shown in Table 2, often still leaves significant uncertainty.

3.2.3 First Flush

Pollutants tend to be washed from the surface of the roadway by the initial runoff in a phenomenon known as the first flush. If the majority of pollutants are contained within the first small portion of rainfall, BMPs only need to be sized to accommodate that volume. The first flush can be described by the first percentage of a storm which runs off or as the first depth of runoff, regardless of total event precipitation. Using the first percentage from roadways method has yielded inconclusive or unsatisfactory results for pollutant loading (Hallberg & Renman 2008; Flint & Davis 2007). Therefore, basing the first flush will be based on an initial runoff depth. The first flush has been observed to remove 81–86% of contaminants in the first 0.5 inches and 89–96% of pollutants in the first 0.75 inches (Flint & Davis 2007).

Early spring rain events and snow melt may also cause a seasonal first flush phenomenon (Sansalone et al. 1995; Stenstrom & Kayhanian 2005). For example, a spike in pollutants during spring may be due to the washing away of pollutants which have built up on roadways during the winter such as deicing agents, and sand and gravel applied to the roadway (MSSC 2005).

Section 4 Methods

4.1 BMP Selection Criteria

The following criteria, which are based on guidance from NDOR, were considered when determining which BMPs were most applicable for roadside applications:

- Pollutants to be remediated: 80% removal TSS (MSSC 2008; KCDENR 2009), heavy metal (Torres 2010), total extractable hydrocarbons (gasoline and diesel) (Torres 2010)
- Low maintenance
- Cost Effective
- No permanent pools
- Implement BMP within existing right of way
- Infiltration should not be primary removal mechanism near roadway
- Peak flow reduction
- Aesthetics
    - Green infrastructure

4.2 BMP Selection Process

Many BMPs were considered for this manual based on the selection criteria. The BMPs which were selected for this manual are vegetated filter strips, vegetated swales, bioretention, sand filters, and horizontal filter trenches.

- Vegetated filter strips and vegetated swales have shown adequate pollutant removal while providing low construction and maintenance costs. They also have high retrofit potential within the right-of-way and provide pleasing aesthetics of vegetation near roadways (UDFC 2010; CEI & NHDES 2008).

- Bioretention is also an effective pollutant removal BMP while providing positive aesthetics (UDFCD 2010; CEI & NHDES 2008) and is flexible enough to be located within the right-of-way or in urban areas (SEMCOG 2008).
- Sand filters were selected based on their track record of successful application in storm water management (Landphair et al. 2000) and their ability to be used in urban areas where land availability limits other BMPs.
- Horizontal filter trenches are a BMP which is being developed. They have been selected for this design guide because they are a relatively simple BMP which will fit within the right-of-way and will not require a significant amount of maintenance.

After evaluating the criteria, some common BMPs which were not deemed suitable for roadside applications are detention facilities, retention ponds, permanent wetlands and infiltration facilities. These were not further evaluated in this work.

- Detention ponds were not included due to limited solids removal compared to other BMPS (CEI & NHDES 2008; EPA 2006a) and space constraints within the right-of-way.
- Retention ponds and permanent wetlands were not considered due to the inherent danger of locating standing water near roadways.
- Inclusion of infiltration facilities would have been redundant because design variations to the horizontal filter trench enable it to act as an infiltration trench, and variations to bioretention allow it to perform as an infiltration basin.
- Ultra-urban BMPs, such as inlet inserts and hydrodynamic separators, were not considered in this report due to their general ineffectiveness as stand-alone BMPs in regard to removal of dissolved solids and metals (EPA 2006e; FHA 2002). These products also tend to be expensive compared to the selected BMPs, particularly in regards to treatment attained (UNH 2005). There are also generally high maintenance burdens to avoid a drop off in performance (EPA 2006e; UNH 2005; FHA 2002c).

4.3 Hydrology

Water Quality Flow and Volume:

First Flush:

The first flush is the initial runoff which comes off the roadway. This value can be defined by percentage of pollutant load, percentage of total runoff, or as a static value of runoff depth (e.g., 0.5 inches or 0.75 inches). The latter definition is a simple yet effective means to quantify the first flush. The first 0.5 inches has been shown to contain 81–86% of pollutants and is commonly used to define the water quality volume (WQV) that requires treatment (Flint and Allen 2010). Therefore, a depth of 0.5 inches is used to represent the first flush and to determine the WQV runoff throughout this document.

Calculating the Design Precipitation

To determine the rainfall that produces the first flush, or WQV, back calculations using the National Resource Conservation Service (NRCS) method and 0.5 inches of runoff are performed using Equation 4-1 (NRCS 1986):

$$\mathbf{Q = \frac{(P-0.2S)^2}{(P+0.8S)}} \tag{4-1}$$

Where:

Q: Depth of runoff over the watershed (in or cm)

P: Precipitation (in or cm)

S: Potential maximum retention of water by the soil (in or cm)

Potential maximum retention is a function of the Curve Number (CN) calculated with Equation 4-2:

$$\mathbf{CN = \frac{1000}{10+S}} \tag{4-2}$$

Table 4 shows the curve numbers for various land uses and hydrologic soil groups. Table 5 defines these soil groups.

Table 4: Curve Numbers for Various Land Uses and Conditions

Description of Land Use	**Hydrologic Soil Group**			
	A	**B**	**C**	**D**
Paved parking lots, roofs, driveways	98	98	98	98
Streets and Roads:				
Paved with curbs and storm sewers	98	98	98	98
Gravel	76	85	89	91
Dirt	72	82	87	89
Cultivated (Agricultural Crop) Land:				
Without conservation treatment (no terraces)	72	81	88	91
With conservation treatment (terraces, contours)	62	71	78	81
Pasture or Range Land:				
Poor (<50% ground cover or heavily grazed)	68	79	86	89
Good (50–75% ground cover; not heavily grazed)	39	61	74	80
Meadow (grass, no grazing, mowed for hay)	30	58	71	78
Brush (good, >75% ground cover)	30	48	65	73
Woods and Forests:				
Poor (small trees/brush destroyed by over-grazing or burning)	45	66	77	83
Fair (grazing but not burned; some brush)	36	60	73	79
Good (no grazing; brush covers ground)	30	55	70	77
Open Spaces (lawns, parks, golf courses, cemeteries, etc.):				
Fair (grass covers 50–75% of area)	49	69	79	84
Good (grass covers >75% of area)	39	61	74	80
Commercial and Business Districts (85% impervious)	89	92	94	95
Industrial Districts (72% impervious)	81	88	91	93
Residential Areas:				
1/8 Acre lots, about 65% impervious	77	85	90	92
1/4 Acre lots, about 38% impervious	61	75	83	87
1/2 Acre lots, about 25% impervious	54	70	80	85
1 Acre lots, about 20% impervious	51	68	79	84

(NRCS 1986)

Table 5: Hydrologic Soil Groups

Group	Minimum Infiltration Rate (in/hr)	Texture
A	0.3–0.45	Sand, loamy sand, or sandy loam
B	0.15–0.3	Silt loam or loam
C	0.05–0.15	Sandy clay loam
D	0–0.05	Clay loam, silty clay loam, sandy clay, silty clay, or clay

(Gupta 2008)

For impervious surfaces, such as pavement, a CN of 98 is assigned resulting in S = 0.2 from Equation 4-2.

$$98 = \frac{1000}{10 + S}$$

Equation 4-1 is then used, in accordance with Nebraska Department of Roads (NDOR) guidance, with the calculated S value and a known Q of 0.5 inches (WQV) in the NRCS equation, the design precipitation is determined to be approximately 0.75 inches. The 0.75 inch event will produce 0.55 inches of runoff. This 0.75 inch storm is then used for BMP designs.

$$0.5 = \frac{(P - 0.2 * 0.2)^2}{(P + 0.8 * 0.2)}$$

Peak Flow-Rate Calculations

Separate peak flow calculations are performed for WQV peak flow rate, which is used to size the BMP treatment processes, and peak flow for the 10-year storm, which is used to design for potential scouring.

Peak Water Quality Flow Rates

Peak flows have been calculated and displayed in Table 6 for impervious surfaces, such as pavement, up to 5 acres. For pervious areas or areas larger than 5 acres, peak flow rates are determined by using the 0.75 inch design storm with a type II NRCS 24-hour distribution and Equation 4-3 (NRCS 1986).

$$\boldsymbol{q_p = q_u A_m Q F_p} \quad (4\text{-}3)$$

Where:

q_p: Peak discharge (cfs)

q_u: Unit peak discharge (cfs/mi^2/in) (Figure 1 or Table 7)

A_m: Drainage area (mi^2)

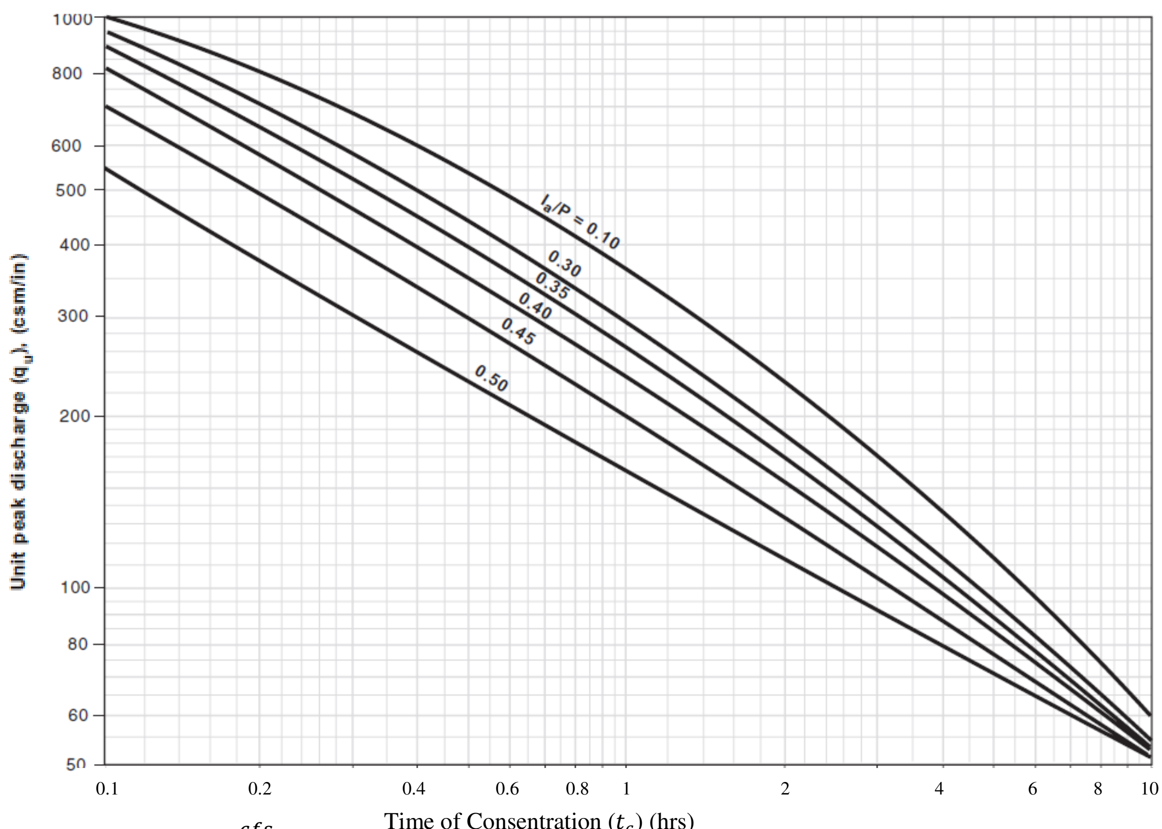
Q: Runoff depth corresponding to 24-hr rainfall (in) (Table 8 for WQV)

F_p: Pond or swamp adjustment factor (1.0 for Nebraska)

Table 6: Peak water quality flows and water quality volumes for impervious watersheds up to 5 acres

Drainage Area (ac)	Peak Discharge [a] (cfs)	WQV^b (ft^3)	Drainage Area (ac)	Peak Discharge [a] (cfs)	WQV^b (ft^3)
0.1	0.095	181.5	1.25	1.184	2268.75
0.2	0.189	363	1.5	1.421	2722.5
0.3	0.284	544.5	1.75	1.657	3176.25
0.4	0.379	726	2	1.894	3630
0.5	0.474	907.5	2.5	2.368	4537.5
0.6	0.568	1089	3	2.841	5445
0.7	0.663	1270.5	3.5	3.315	6352.5
0.8	0.758	1452	4	3.788	7260
0.9	0.852	1633.5	4.5	4.262	8167.5
1	0.947	1815	5	4.735	9075

a) Calculated with Equation 4-3 and the 0.75 inch design storm
b) Calculated with Equation 4-4 and 0.5 inches of runoff



*csm/in = $\frac{cfs}{mi^2 * in}$

Figure 1: Unit peak discharge for Type II distribution (NRCS 1986)

Table 7: Unit Peak discharge (qu) and initial abstraction (Ia) for various curve numbers (CN), for the water quality volume (WQV), and scour check (10 yr) storms (NRCS 1986)

CN	I_a[a] (in)	$\frac{I_a}{0.75\ in}$[b] (WQV)	$\frac{I_a}{5\ in}$[d] (10 yr)	qu[c] (10 yr) $\left(\frac{cfs}{mi^2 * in}\right)$
40	3.00	4.00	0.60	550
41	2.88	3.84	0.58	550
42	2.76	3.68	0.55	550
43	2.65	3.53	0.53	550
44	2.55	3.39	0.51	550
45	2.44	3.26	0.49	580
46	2.35	3.13	0.47	610
47	2.26	3.01	0.45	700
48	2.17	2.89	0.43	720
49	2.08	2.78	0.42	800
50	2.00	2.67	0.40	815
51	1.92	2.56	0.38	840
52	1.85	2.46	0.37	880
53	1.77	2.37	0.35	900
54	1.70	2.27	0.34	910
55	1.64	2.18	0.33	925
56	1.57	2.09	0.31	935
57	1.51	2.01	0.30	950
58	1.45	1.93	0.29	950
59	1.39	1.85	0.28	960

CN	I_a[a] (in)	$\frac{I_a}{0.75\ in}$[b] (WQV)	qu[c] (WQV) $\left(\frac{cfs}{mi^2 * in}\right)$	$\frac{I_a}{5\ in}$[d] (10 yr)	qu[c] (10 yr) $\left(\frac{cfs}{mi^2 * in}\right)$
60	1.33	1.78	-	0.27	965
61	1.28	1.71	-	0.26	965
62	1.23	1.63	-	0.25	965
63	1.18	1.57	-	0.24	970
64	1.13	1.50	-	0.23	970
65	1.08	1.44	-	0.22	970
66	1.03	1.37	-	0.21	975
67	0.99	1.31	-	0.20	975
68	0.94	1.25	-	0.19	975
69	0.90	1.20	-	0.18	975
70	0.86	1.14	-	0.17	980
71	0.08	0.11	-	0.02	980
72	0.78	1.04	-	0.16	980
73	0.74	0.99	-	0.15	985
74	0.70	0.94	550.00	0.14	985
75	0.67	0.89	550.00	0.13	990
76	0.63	0.84	550.00	0.13	990
77	0.60	0.80	550.00	0.12	995
78	0.56	0.75	550.00	0.11	995
79	0.53	0.71	550.00	0.11	995

CN	I_a[a] (in)	$\frac{I_a}{0.75\ in}$[b] (WQV)	qu[c] (WQV) $\left(\frac{cfs}{mi^2 * in}\right)$	$\frac{I_a}{5\ in}$[d] (10 yr)	qu[c] (10 yr) $\left(\frac{cfs}{mi^2 * in}\right)$
80	0.50	0.67	550	0.10	1000
81	0.47	0.63	550	0.09	1000
82	0.44	0.59	550	0.09	1000
83	0.41	0.55	550	0.08	1000
84	0.38	0.51	550	0.08	1000
85	0.35	0.47	600	0.07	1000
86	0.33	0.43	720	0.07	1000
87	0.30	0.40	795	0.06	1000
88	0.27	0.36	880	0.05	1000
89	0.25	0.33	910	0.05	1000
90	0.22	0.30	955	0.04	1000
91	0.20	0.26	965	0.04	1000
92	0.17	0.23	970	0.03	1000
93	0.15	0.20	975	0.03	1000
94	0.13	0.17	980	0.03	1000
95	0.11	0.14	985	0.02	1000
96	0.08	0.11	990	0.02	1000
97	0.06	0.08	1000	0.01	1000
98	0.04	0.05	1100	0.01	1100

a) Initial abstraction is a function of the CN and was found in TR-55 (NRCS 1986)

b) Initial abstraction to precipitation ratio for WQV (0.75 inch) rainfall

c) Determined from Figure 1 with a t_c= 5 min and the corresponding $\frac{I_a}{P}$ value

d) Initial abstraction to precipitation ratio for 10-year (5 inch) rainfall

Table 8: Curve numbers with their associated runoff depths for 0.75 inch rainfall (WQV)

CN	Q^a (in)	CN	Q^a (in)
≤73	0	86	0.088
74	0.001	87	0.105
75	0.002	88	0.124
76	0.004	89	0.145
77	0.007	90	0.170
78	0.011	91	0.198
79	0.017	92	0.230
80	0.023	93	0.266
81	0.030	94	0.307
82	0.039	95	0.355
83	0.049	96	0.410
84	0.060	97	0.475
85	0.073	98	0.551
a) Calculated using Equation 4-1			

The unit peak discharge (q_u) is a function of the time of concentration, the $\frac{I_a}{P}$ ratio, and the rainfall distribution type. The time of concentration is dependent on watershed characteristics and is defined as the time it takes for water to move from the hydraulically most distant point in the watershed to the outlet. The $\frac{I_a}{P}$ ratio is determined by dividing the initial abstraction (I_a), which can be found in Table 7, by the total precipitation (0.75 inches for the design storm). The entire state of Nebraska falls within the type II rainfall distribution.

For runoff from impervious areas and rainfall depth of 0.75 inches the $\frac{I_a}{P}$ ratio is ~0.055. This value, along with time of concentration, is then used to determine q_u from Figure 1. Using a conservative 5 minute time of concentration, q_u is found to be approximately 1100 $\frac{cfs}{mi^2 * in}$. This value was extrapolated from Figure 1.

The accuracy of this method will be reduced for values of $\frac{I_a}{P}$ outside of the range shown on Figure 1. If the values fall outside of this range use the tabular hydrograph method as stated in the TR-55 manual (NRCS 1986). There are also several software packages which are equipped to perform these calculations for complicated basins.

When considering a watershed with both impervious and pervious ground cover, the area can either be considered completely impervious, or a weighted flow may be calculated. Assuming total imperviousness would result in larger than actual flows and, therefore, oversized BMPs. For this reason the weighted flow method is recommended.

When using the weighted flow method, consider the impervious and pervious sections of the watershed individually and sum the resulting peak flows from each section. This method differs from the weighted curve number method by taking into account the runoff which flows directly from the impervious area to the BMP without first encountering the pervious area. The weighted flow method results in larger flows which are more realistic in many roadway scenarios.

Peak Ten Year Flow Rates

The peak ten year (scouring) flow rate will be used for scour checks in coordination with storm sewer sizing for expressways (NDOR 1996) and is calculated using Equation 4-3. The Equation 4-3 variables associated with the 10-year storm can be found in these locations:

- q_u can be found on Table 7 or Figure 1
- Q can be found on Table 9
- F_P is 1.0 for Nebraska
- A_m is site-specific

When calculating the 10-year scour flow a 5 inch rainfall will be used. The 5 inch rainfall represents the highest peak precipitation in the state of Nebraska for the 10-year storm. Using the largest rainfall event will result in adequate or conservative sizing across the state. Similarly to the WQV calculations, the weighted flow method should be used.

Table 9: Curve numbers with their associated runoff depths for 5 inch rainfall (10 year storm)

CN	Q^a (in)	CN	Q^a (in)	CN	Q^a (in)	CN	Q^a (in)
31	0.01	48	0.59	65	1.65	82	3.08
32	0.03	49	0.64	66	1.73	83	3.17
33	0.04	50	0.69	67	1.80	84	3.27
34	0.06	51	0.75	68	1.88	85	3.37
35	0.08	52	0.80	69	1.96	86	3.47
36	0.11	53	0.86	70	2.04	87	3.57
37	0.14	54	0.92	71	2.12	88	3.67
38	0.17	55	0.98	72	2.20	89	3.77
39	0.20	56	1.04	73	2.28	90	3.88
40	0.24	57	1.10	74	2.36	91	3.98
41	0.27	58	1.17	75	2.45	92	4.09
42	0.31	59	1.23	76	2.54	93	4.20
43	0.35	60	1.30	77	2.62	94	4.31
44	0.40	61	1.37	78	2.71	95	4.42
45	0.44	62	1.44	79	2.80	96	4.53
46	0.49	63	1.51	80	2.89	97	4.65
47	0.54	64	1.58	81	2.99	98	4.76
a) Calculated using Equation 4-1							

Calculating the Water Quality Volume:

Water quality volumes for impervious surfaces, such as pavement, up to 5 acres have been calculated and displayed in Table 6. For pervious areas or areas larger than 5 acres, use the following methodology.

The water quality volume is found by multiplying the new development area (e.g., newly constructed roadway) by 0.5 inches (Equation 4-4). This volume will then be incorporated into the BMP design.

$$\boldsymbol{WQV_{New\ Dev} = 0.5in * \frac{Area\ of\ New\ Roadway\ (ft^2)}{12\frac{in}{ft}}} \quad (4\text{-}4)$$

Calculating Run-On Volume

Run-on (WQV_{Run-On}) is water from surfaces (impervious or pervious), other than the new development area, that is co-mingled with water from the new development area. Because run-on co-mingles with the $WQV_{New\ Dev}$, it must be treated in the BMP.

Run-on volume from pervious surfaces during the 0.75 inch rainfall event will result in less than 0.5 inches of runoff. Table 8 shows the runoff depth from a 0.75 inch rainfall for areas with various curve numbers.

For areas that contribute run-on that will co-mingle with the $WQV_{New\ Dev}$, the run-on volume (WQV_{Run-On}) can be calculated by using Equation 4-5:

$$\mathbf{WQV_{Run-On} = Q\ from\ Table\ 8 * \frac{Contributing\ area(ft^2)}{12\frac{in}{ft}}} \quad (4\text{-}5)$$

The total water quality volume (WQV_{Total}) is the sum of the runoff from new impervious areas ($WQV_{New\ Dev}$) and run-on (WQV_{Run-on}) as shown in Equation 4-6.

$$\mathbf{WQV_{Total} = WQV_{New\ Dev} + WQV_{Run-on}} \quad (4\text{-}6)$$

This volume is the minimum amount of water to be treated.

In-line BMPs need to be designed to either handle the flow of larger storms, or they need to be able to bypass larger flows. For offline BMPs the WQV from the new roadway to be treated must be routed through the BMP.

Design Example

The urban highway in Figure 2 is being redeveloped. The redeveloped highway contributes 3.3 acres which will contribute to the water quality volume and peak flows. There are vegetated areas north and south of the highway that account for 4 acres of extra drainage (run-on) to the system.

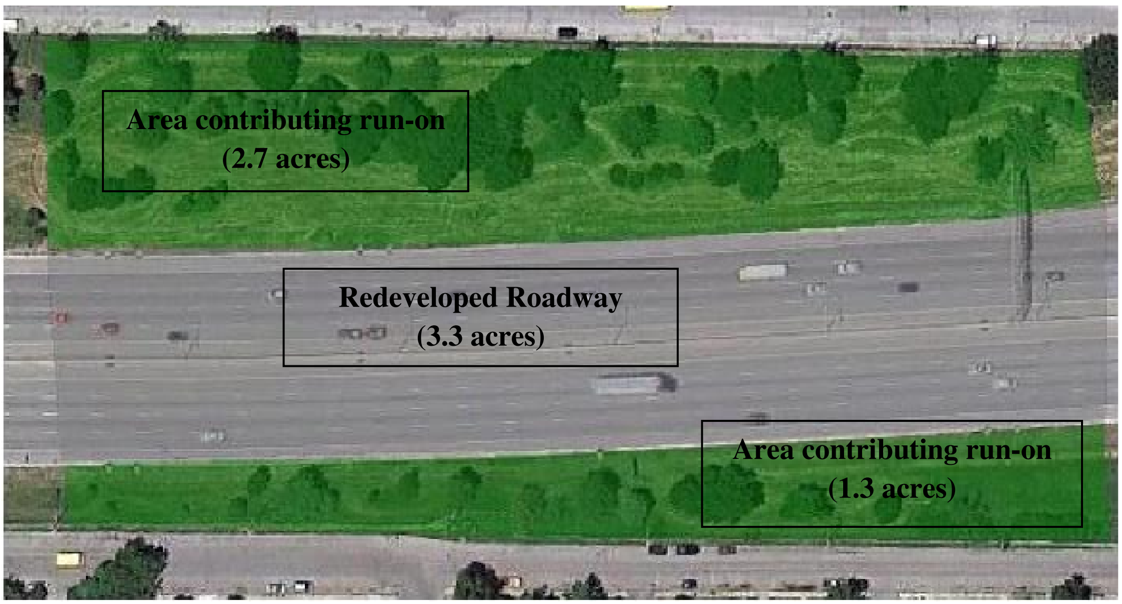


Figure 2: Plan view of redeveloped highway

Calculating Peak Water Quality Flow Rates

The WQV and peak WQV flow rate are calculated using precipitation of 0.75 inches which corresponds to 0.5 inches of runoff from impervious surfaces. The peak water quality flow rate will be found by summing the peak flow coming off of the 3 sub-basins within the system. The individual peak flow rates are found using Equation 4-3.

The curve number for each section is given in Table 4. The CN is used to determine the runoff depth (Q) from Table 8. The redeveloped roadway is paved, so it has a curve number of 98 according to Table 4. The vegetated sections are considered open space with grass cover of greater than 75% in soil type C, as described by Table 5, so they each have a curve number of 74. Table 8 shows that a curve number of 98 produces a runoff, Q, of 0.551 inches for the design 0.75

inch rainfall, and a curve number of 74 produces a runoff depth, Q, of 0.001 inches for the design 0.75 inch rainfall.

The unit peak discharge (q_u) can be found on Table 7, or it can be determined by using the initial abstraction (I_a) in the ratio of initial abstraction (I_A) to precipitation (P) $\left(\frac{I_a}{P}\right)$, found on Table 7, along with Figure 1 and an assumed time of concentration (t_c). A curve number of 98 results in a q_u of 1100 $\frac{cfs}{mi^2 * in}$, and a curve number of 74 results in a q_u of 550 $\frac{cfs}{mi^2 * in}$ for the WQV rainfall.

The swamp adjustment factor F_p is assumed to be 1 for the state of Nebraska. Equation 4-3 ($q_p = q_u A_m Q F_p$) is then solved for each area, these values are given for impervious areas in Table 6 but must be calculated for the pervious areas.

Redeveloped roadway:

$$q_p = 1100 \frac{cfs}{mi^2 * in} * 0.005\ mi^2 * 0.551\ in * 1 = 3.03 \text{ cfs}$$

2.7 acre vegetated area:

$$q_p = 550 \frac{cfs}{mi^2 * in} * 0.004 mi^2 * 0.001\ in * 1 = 0.0022 \text{ cfs}$$

1.3 acre vegetated area:

$$q_p = 550 \frac{cfs}{mi^2 * in} * 0.002\ mi^2 * 0.001\ in * 1 = 0\ .0011 \text{ cfs}$$

The flows from each area are then summed to find the peak WQV flow of the drainage area.

$$3.03\ cfs + 0.0022\ cfs + 0.0011\ cfs = 3.03\ cfs$$

The peak WQV flow is found to be 3.03 cfs.

Table 10 summarizes calculations for peak water quality flow.

Table 10: Calculations for peak WQV flow

(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)
A (acres)	P (in)	CN	$t_c{}^a$ (min)	I_a (in)	$\frac{I_a}{P}{}^a$	$q_u{}^b$ $\left(\frac{cfs}{mi^2 * in}\right)$	Q^c (in)	Area (A) (mi^2)	F_p	$q_p{}^d$ (cfs)
3.3	0.75	98	5	0.04	0.053	1100	0.551	0.005	1	3.031
2.7	0.75	74	5	0.7	0.933	550	0.001	0.004	1	0.002
1.3	0.75	74	5	0.7	0.933	550	0.001	0.002	1	0.001
a) Use with Figure 1 to find q_u b) Found with Figure 1 or Table 7 c) Found in Table 8 d) (7)*(8)*(9)*(10)										q_p Total (cfs) 3.03

Calculating Peak Flow Rates for Scour Evaluation

The peak flow rate is used to evaluate the need for scour protection in flow-through BMPs. It is found by summing the peak flow coming from the 3 sub-basins within the system from the 10-year (5 inch) storm. The individual peak flow rates are found using Equation 4-3 ($q_p = q_u A_m Q F_p$).

The curve number (Table 4) for each section is used to find the runoff depth (Q) from Table 9. The redeveloped roadway is paved, so it has a curve number of 98 and a Q of 4.76 inches. The vegetated sections are considered open space with grass cover of greater than 75% in soil type C, as described by Table 5, so they each have a curve number of 74 and a Q of 2.36 inches.

The unit peak discharge (q_u) can be found on Table 7 or can be determined by using the initial abstraction (I_a) in the ratio of initial abstraction (I_a) to precipitation (P) $\left(\frac{I_a}{5\ in}\right)$, found on Table 7, along with Figure 1 and an assumed time of concentration (t_c). A curve number of 98 results in a q_u of 1100 $\frac{cfs}{mi^2 * in}$, and a curve number of 74 results in a q_u of 985 $\frac{cfs}{mi^2 * in}$ for the 10-year rainfall.

The swamp adjustment factor F_p is assumed to be 1 for the state of Nebraska.

Equation 4-3 ($q_p = q_u A_m Q F_p$) is then solved for each area.

Redeveloped roadway:

$$q_p = 1100 \frac{cfs}{mi^2 * in} * 0.005\ mi^2 * 4.76\ in * 1 = 26.2 \text{ cfs}$$

2.7 acre vegetated area:

$$q_p = 985 \frac{cfs}{mi^2 * in} * 0.004 mi^2 * 2.36\ in * 1 = 9.3 \text{ cfs}$$

1.3 acre vegetated area:

$$q_p = 985 \frac{cfs}{mi^2 * in} * 0.002\ mi^2 * 2.36\ in * 1 = 4.6 \text{ cfs}$$

The flows from each area are then summed to find the peak flow of the drainage area.

$$26.2\ cfs + 9.3\ cfs + 4.6\ cfs = 40.1\ cfs$$

The peak flow is found to be 40.1 cfs.

Table 11 summarizes the calculations for peak scour flow.

Table 11: Calculations for peak scour flow

(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)
A (acres)	P (in)	CN	${t_c}^a$ (min)	I_a (in)	$\frac{I_a}{P}^a$	${q_u}^b$ $\left(\frac{cfs}{mi^2 * in}\right)$	Q^c (in)	Area (A) (mi^2)	F_p	${q_p}^d$ (cfs)
3.3	5	98	5	0.04	0.008	1100	4.76	0.005	1	26.180
2.7	5	74	5	0.7	0.140	985	2.36	0.004	1	9.298
1.3	5	74	5	0.7	0.140	985	2.36	0.002	1	4.649
a) Use with to find q_u b) Find with or Table 7 c) Found in Table 9										q_p Total (cfs)
d) (7)*(8)*(9)*(10)										40.1

Calculating Water Quality Volume:

The total water quality volume (WQV) is the sum of the WQV from new development (e.g. pavement) ($WQV_{New\ Dev}$) and the volume of run-on which co-mingles with the $WQV_{New\ Dev}$ (WQV_{Run-On}). $WQV_{New\ Dev}$ is found by multiplying the newly constructed or redeveloped area by 0.5 inches using Equation 4-4.

$$WQV_{New\ Dev} = 0.5in * \frac{Area\ of\ New\ Roadway\ (ft^2)}{12\frac{in}{ft}}$$

For this example, the area of newly developed pavement is 3.3 acres (143,747 ft^2). Thus the $WQV_{New\ Dev}$ is:

$$WQV_{New\ Dev} = 0.5in * \frac{143{,}747\ ft^2}{12\frac{in}{ft}} = 5{,}989 ft^3$$

In order to calculate the run-on volume (WQV_{Run-On}) the depth of runoff (Q) from the 0.75 inch storm must be found for the associated curve numbers of the contributing areas determined by Table 8. This value is incorporated into Equation 4-5 to find the WQV_{Run-On}.

$$WQV_{Run-On} = Q\ from\ \text{Table } \mathbf{8} * \frac{Contributing\ area(ft^2)}{12\frac{in}{ft}}$$

The WQV_{Run-On} for the 2.7 acre vegetated area is:

$$WQV_{Run-On} = 0.001\ inches * \frac{117{,}611 ft^2}{12\frac{in}{ft}} = 9.8\ \ ft^3$$

The WQV_{Run-On} for the 1.3 acre vegetated area is:

$$WQV_{Run-On} = 0.001\ inches * \frac{56{,}628 ft^2}{12\frac{in}{ft}} = \ 4.7\ ft^3$$

Equation 4-6 is then used to find the total volume requiring treatment.

$$5{,}989 ft^3 + 9.8\ \ ft^3 + 4.7\ ft^3 = 6{,}004\ ft^3$$

Section 5 Results and Discussion

5.1 Fact Sheets

The fact sheets provide the design engineer the background on each BMP which will be used to determine the applicability of a specific BMP or determine which BMP is best for site-specific conditions. Each fact sheet typically includes the following:

- Description: Provides a basic description of the BMP.
- Pollutant removal potential: Shows pollutant removal based on multiple studies.
- Initial costs: Provides projected capital costs and costs observed during case studies.
- Maintenance costs: Provides estimates and case study results of maintenance costs as well as required maintenance hours.
- Siting constraints: Identifies applicable locations and conditions for the BMP.
- Maintenance and operation considerations: Identifies ways to prevent and repair potential problems with the BMP.

5.1.1 Vegetated Filter Strip

Figure 3: Roadside vegetated filter strip (TWG 2008)

Description:

Vegetated filter strips, also known as vegetated buffers or grass filter strips, are sloped vegetated surfaces which are intended to treat runoff from adjacent impervious areas. These areas must have sufficient vegetative cover and minimal slope perpendicular to flow (cross slope) to facilitate treatment. Treatment of runoff is accomplished primarily through filtration, biological processes associated with the vegetation, and infiltration.

The primary requirement with vegetated filter strips is maintaining sheet flow. If runoff is allowed to channelize there are two primary drawbacks. The first drawback is the formation of rills, which can occur when concentrated flows locally erode surface soils. This eroded material then adds to the solids load of the runoff. The second problem comes from short-circuiting associated with rill formation. Rills allow runoff to bypass the vegetation where treatment occurs. Vegetated filter strips become largely ineffective if channelization is allowed to occur.

One way to maintain sheet flow is through the use of a level spreader. Level spreaders are used to slow and evenly distribute runoff. Roadside level spreaders include gravel filled trenches, earthen berms, rip-rap, or treated lumber which have minimal cross slope. It is recommended to use level spreaders at the top of the buffer.

Pollutant Removal Potential:

Vegetated filter strips primarily remediate runoff through filtration, biological processes, and infiltration. High solids removal has been shown in the first 13 ft (4 m) of the strip (Barrett 2005), and it plateaus after 33 ft (10 m) (Zhang et al. 2010). The slope should not exceed 15% to keep velocities low and pollutant removal high. Removal of solids peaks at 10% slope, though has been shown to be effective at steeper slopes (Zhang et al. 2010). Table 12 shows observed pollutant removals by vegetated filter strips.

Table 12: Pollutant removal potential for vegetated filter strips

	Zhang et al.[1] 2010	Li et al. 2008	Caltrans 2004	Winston and Hunt [3] 2010	Barrett and Walsh [4] 1998
	Removal %	Removal %	Removal %	Removal %	Removal %
Pollutant					
Total Suspended Solids (TSS)	86	35.7	83	68	85
Total Nitrogen (TN)	68.3	4.7	44	13	48
Total Phosphorous (TP)	71.9	-121	-76	12	45
Total Metals (TM)	-	49.7	89.3[2]	-	63[5]

1) Results of a literature review
2) Average of Cu, Pb, & Zn
3) Average of 6 sites
4) Load reduction of existing infrastructure
5) Average of Zn, PB, & Fe

Cost Considerations:

Initial Cost:

The small amount of design and infrastructure associated with vegetated filter strips makes them a relatively inexpensive BMP. The construction costs of vegetated buffers include grading, vegetating the strip, and installation of a level spreader. The cost of grass installation has been estimated at \$13,000 per acre for seeding and \$30,000 per acre for sod as of 2006 (EPA

2006d). Level spreader costs range from \$5–\$20 per foot as of 2006 (DEPBWM 2006), and grading costs vary with site size and conditions.

Another major expense is the availability of the land required to place this BMP. The large foot print can make vegetated filter strips impractical in urban areas where acquiring the necessary land is expensive. However, it has been shown that existing vegetation along roadways can act as vegetated filters (Barrett 2005). Sites which are already acting in this capacity require very little initial capital.

Maintenance Costs:

Maintenance costs are also low with vegetated filter strips. Annual maintenance costs have been estimated at \$350 per acre of filter strip based on a report from 1991 (EPA 2006d). A study (CalTrans 2004) demonstrated that the majority of maintenance overlapped with general roadside maintenance. A related study was performed which showed that the pollutant removal effectiveness of existing roadside vegetation, which had only regular maintenance, compared favorably with filter strips designed for water quality improvement (CalTrans 2003).

Siting Constraints:

Vegetated filter strips are applicable for use in most areas, and are effective as pretreatment BMPs in a treatment train. Runoff from small areas such as parking lots or roadways is a good candidate for treatment by vegetated strips. However, the relatively large spatial requirement of filter strips is a major restricting factor.

Although filter strips are suited for most climates, they may need some climate-specific considerations. For example, in cold or seasonal climates vegetation should be selected that is salt tolerant, especially when adjacent to roadways. In more arid regions lack of rainfall may require irrigation to maintain acceptable vegetated cover which may make vegetated filter strips cost-prohibitive.

The large size requirement creates the potential for the required width of the buffer to extend beyond the standard right of way. Intrusion on neighboring properties causes an increase in cost which may limit the practicality of filter strips. Other BMPs, with a smaller footprint, may be better suited for densely developed areas.

Another constraining factor is the requirement for minimal slope perpendicular to flow. This is of particular concern for some roadway applications because the land adjacent to roads generally has a similar topography. Highways which have vertical curves of more than 2% will likely not be able to effectively accommodate a vegetated filter strip. Other forms of vegetated filtration such as vegetated swales may be considered in these areas. See the Vegetated Swale Fact Sheet of this work to determine their applicability.

There must be safe access to all parts of the filter strip. Due to the nature of this BMP, maintenance vehicle access at the top of the slope should be sufficient for the majority of maintenance activities. Any necessary vehicle traffic on the strip should occur when the ground is dry, and vehicles should travel horizontally across the strip as much as possible. The ruts formed decrease vegetated cover which can reduce the performance of this BMP, and if the ruts are created running down-hill they will promote channelization.

Maintenance and Operation Considerations:

It is very likely that much of the cost of operation and maintenance will overlap with general vegetation maintenance along roadways (Barrett 2005). The primary focuses of maintenance are maintaining healthy vegetation, removing litter and detritus, and the preservation of sheet flow throughout the length and width of the filter strip. Maintaining healthy vegetation consists of keeping vegetated cover above 80%. This should be done, as much as possible, without the use of pesticides or herbicides which can contribute to contaminants in the runoff.

In a retrofit study, the California Department of Transportation (Caltrans 2004) found that 105 hrs/yr were required to maintain an effective filter strip serving 4.9 acres. 67 of these hours were spent mowing and removing woody vegetation, which are standard roadside maintenance activities.

Table 13 shows potential maintenance and operation requirements of vegetated filter strips which could be observed during inspection and suggested corrective procedures.

Table 13: Operations and maintenance considerations and suggested corrective procedures

Inspection Frequency	Problem	Suggested Corrective Procedure
Annually	Level spreader is not distributing runoff evenly across strip due unevenness or clogging.	Level the flow spreader and clean out clogs (NCDENR 2007).
	Substantial channelization or rilling.	Regrade and reseed the strip (DEPBWM 2006).
Semi-Annually	Burrowing animals cause vegetated cover to drop below 80%.	Take applicable action which will vary with pest type.
	Sediment accumulation of 3 or more inches near outlet or enough to cover vegetation within the strip.	Remove sediment, re-level, and replant where applicable.
Regularly/ As Needed	Grass becomes unacceptably tall.	Maintain grass length from 2”–6”. Clippings should be removed if nutrients are pollutants of concern. Mowing should be performed across the slope when it is dry so rutting caused by tires will not promote channelization (DEPBWM 2006).
	Weeds or unwanted vegetation begin to dominate strip.	Weeds should be removed by hand ideally, otherwise a herbicide which is not toxic to recommended vegetation should be used (NCDENR 2007).
	Bare areas form within strip.	Remulch and reseed bare areas.
	Rills of less than 8” wide form.	Fill rills with gravel which will soon be overtaken by grass (SEMCOG 2008).
	Litter and detritus build up.	Remove litter which is aesthetically unpleasant, negatively affects performance of the strip, or is itself harmful to the environment (CalTrans 2004).
	Not enough rainfall to sustain vegetation.	Irrigation may be necessary to maintain adequate cover. It is suggested that grasses be selected which are drought tolerant and will not require irrigation.
	Standing water beyond 48 hrs of isolated storm event.	Repair grade where runoff pools and take any necessary vector control measures (SEMCOG 2008).

5.1.2 Vegetated Swale

Figure 4: Roadside Swale (CalTrans 2012)

Description:

Vegetated swales are open channels which have vegetative (usually grass) linings that provide water quality benefits while conveying stormwater runoff. Swales rely on maintaining low flow velocities to promote sedimentation, filtration through the vegetation, and infiltration. The low velocity also decreases peak runoff rates from impervious drainage areas. The vegetated channels also have more aesthetic appeal than rock or concrete lined channels.

Swales can be enhanced with check dams to reduce flow velocity and to create temporary ponding which promote sedimentation and infiltration. Check dams can improve the functionality of the BMP as well as increase the life span of vegetated swales(Landphair et al. 2000) .

Pollutant Removal Potential:

Vegetated swales have shown good removal for solids and metals and moderate removal for nutrients, such as phosphorous and nitrogen (UDFCD 2010). Pollutant removals by vegetated swales, as reported by several researchers, are presented in Table 14.

Table 14: Pollutant removal potential for vegetated swales

	Landphair [1] et al. 2000	MSSC 2005	CalTrans 2004[3]	DEPBWM 2006	Clar et al. 2004
Pollutant	Removal %	Removal %	Removal %	Removal %	Removal %
Total Suspended Solids (TSS)	81–98	85	76	50	83
Total Nitrogen (TN)	40–99	35	67	50	25
Total Phosphorous (TP)	18–99	50	1	20	29
Total Metals (TM)	78.5[2]	80	85[4]		59[5]
Hydrocarbons (oil and grease)	-	80	-		75
1) Average of 6 sites 2) Zn: 60–99; Pb: 50–99 3) Average of 6 sites		4) Average of Pb, Cu, and Zn 5) Average of Pb, Cu, and Zn			

Cost Considerations:

Initial Cost:

Initial capital costs for vegetated swales are generally low. Existing infrastructure should be used as much as possible to keep costs low. In many cases it is possible to meet municipal separate storm sewer discharge permit requirements as specified in the Clean Water Act Section 401 by adding check dams to existing drainage measures (Landphair et al. 2000). Construction costs can result from swale size, grading, clearing, grubbing, or plant establishment. The EPA has predicted swale construction costs to range from \$0.25–\$0.50 per ft^2 (\$2.75–\$5.50 per m^2) based on a report from 1997(EPA 2006b).

The Pennsylvania Stormwater BMP Manual reported costs of \$8.50 to \$50 per linear foot (\$28 to \$165 per meter) in 2006 (DEPBWM 2006). The Michigan LID Manual predicts costs ranging from \$4.50 to \$8.50 per linear foot (\$15 to \$28 per meter) for seeding and \$15 to \$20 per linear foot (\$50 to \$66 per meter) for sodding as of 2008 (SEMCOG 2008). These values compare favorably to capital costs for underground pipes (\$2 per foot per inch of diameter) and curb and gutter systems (\$13–\$15 per foot) (SEMCOG 2008).

Another method of cost estimation is based on cost per volume treated. Cost per volume can range from \$0.50 per ft^3 (\$18 per m^3) (CH2MHILL 2008) to \$1.50 per ft^3 (\$52 per m^3) (CalTrans 2004). The cost of the swale per volume treated can vary based on the size of the contributing watershed and the scope of the construction project. Although these values are good for estimation, larger drainage areas have been shown to have lower costs per volume treated (CalTrans 2004), so a linear relationship may not be reliable. Construction costs can also be mitigated by constructing the swale in conjunction with other construction activities within a larger project (Lampe et al. 2005).

Maintenance Costs:

Vegetated swales are considered to have a low life cycle cost when compared to other BMPs (UDFCD 2010). Annual maintenance costs for swales are expected to be 5–7% of the construction costs (CH2MHILL 2008). This estimate fits with a 2004 study which projects \$2,736 of annual maintenance for a swale serving 6 acres (CalTrans 2004).

Siting Constraints:

Vegetated swales are useful along roadways, parking lots, and as components of treatment trains (KCDNRP 2009). Their linear nature and combination of drainage and water quality benefits make them ideal for use along roadways (KCDNRP 2009). Existing drainage areas within the right-of-way, such as ditches and medians, are often compatible with the use of vegetated swales. Existing drainage infrastructure (e.g., ditches) may already be functioning as a vegetated swale, but any retrofit project requires confirmation with the constraints laid out in the

Vegetated Swale Design Guide section of this work. When using swales along roadways, they can effectively replace the curb and gutter system (UDFCD 2010).

Contributing drainage area also limits the applicability of what vegetated swales are best suited for. Ideally, swales will not treat more than 5 acres (SEMCOG 2008). However, guidance of up to 10 acres has been given (Clar et al. 2004). If treating more than 5 acres, less than 5 acres of the contributing area should be impervious (KCDNRP 2009).

If vegetated swales have a gentle slope (i.e., < 1%) they should not be used where the seasonal high watertable, or bedrock is within 2 feet (0.61 m) of the bottom of the swale. Building the swale with inadequate drainage considerations could result in dewatering problems which can lead to mosquito breeding grounds (SEMCOG 2008). Dewatering is also a concern with NRCS type D (i.e., clay) soils (Landphair et al. 2000). Swales may still be used in type D soils, but an adequate slope (i.e., greater than 1%) must be maintained throughout the course of the swale to facilitate drainage. When considering swales for urban or residential applications, the number of driveways crossing the swale must be considered. Driveways crossing the swale require culverts to pass flows. Culverts can reduce pollutant removal by vegetated swales (Clar et al. 2004).

Maintenance and Operation Considerations:

Maintenance of vegetated swales overlaps significantly with normal vegetated roadside maintenance (Landphair et al. 2000). These maintenance considerations are focused on supporting healthy grass, removing trash, mowing, and keeping woody vegetation down. Additional considerations for water quality swales include sediment removal, preventing and fixing erosion, providing even distribution of flow across the channel, and maintaining check dams (if present). A study found that vegetated swales, when designed properly, should require approximately 50 hours of maintenance annually for a swale serving 6 acres (CalTrans 2004).

Table 15: Operations and maintenance considerations and suggested corrective procedures

Inspection Frequency	Problem	Suggested Corrective Procedure
Annually	Sediment inhibits grass growth in more than 10% of the swale length or inhibits even spread of runoff	Remove sediment by hand or with flat shovel and reseed with same mix as soon as possible (KCDNRP 2009)
	Substantial channelization or rilling.	Regrade and reseed the swale (KCDNRP 2009)
Semi-Annually	Burrowing animals cause vegetated cover to drop below 80%.	Take applicable action which will vary with pest type.
	Sediment accumulation of 3 or more inches near outlet or enough to cover vegetation within the strip.	Remove sediment, re-level, and replant where applicable (Clar et al. 2004, CalTrans 2004)
	Check dam gets clogged with debris or sediment	Remove sediment or debris and reseed with same mix as soon as possible (Landphair et al. 2000)
Regularly/ As Needed	Grass becomes unacceptably tall.	Maintain grass length from 3−4 in (FHA 1997b). Clippings should be removed if nutrients are concern pollutants (Clar et al. 2004).
	Weeds or unwanted vegetation begin to dominate strip.	Weeds should be removed without using tactics which adversely affect recommended vegetation (CalTrans 2004).
	Rills of less than 8” wide form.	Fill, compact, and reseed eroded area with same seed mix (Clar et al. 2004)
	Litter and detritus build up.	Remove litter which is aesthetically unpleasant, negatively affects performance of the swale, or is itself harmful to the environment (FHA 1997b).
	Not enough rainfall to sustain vegetation.	Irrigation may be necessary to maintain adequate cover (SEMCOG 2008). It is suggested that grasses be selected which are drought tolerant and will not require irrigation.
	Standing water beyond 48 hrs of isolated storm event.	Repair grade where runoff pools and take any necessary vector control measures (MSSC 2005).

5.1.3 Bioretention Cell

Figure 5 Highway median bioretention in Delaware (DelDOT 2012)

Description:

Bioretention BMPs are highly customizable and flexible vegetated soil filters that are designed to retain and treat the water quality volume (WQV) and filter it through an engineered soil mix. Remediation is accomplished through filtration, plant uptake, and potentially, infiltration. The soil mix must allow the retained runoff to drain in 24 to 48 hours while performing remediation functions and supporting the vegetation in the system (MDEP 2009).

The vegetation can be very diverse in bioretention; however, using grass as the only vegetation can produce excellent water quality results (Davis et al. 2009). Trees should not be used near roadways due to safety concerns. If vegetation is properly selected and maintained bioretention cells can be very beneficial aesthetically along with their environmental benefits. Vegetation selection and planting strategies are discussed in the Bioretention Design Guide.

Bioretention BMPs can be designed as either infiltration or filtration facilities. Infiltration is encouraged if it does not threaten surrounding buildings or roadways. Infiltrating the WQV contributes to ground water recharge as well as decreasing runoff which could contribute to stream channel erosion. In situations where infiltration is not desirable, an under-drain is used to discharge treated runoff. Under-drain systems are ideal for areas with impermeable soils or in highly developed areas. Figure 5 shows a bioretention facility in a roadway median, and Figure 6 shows a plan and section view of a potential bioretention layout.

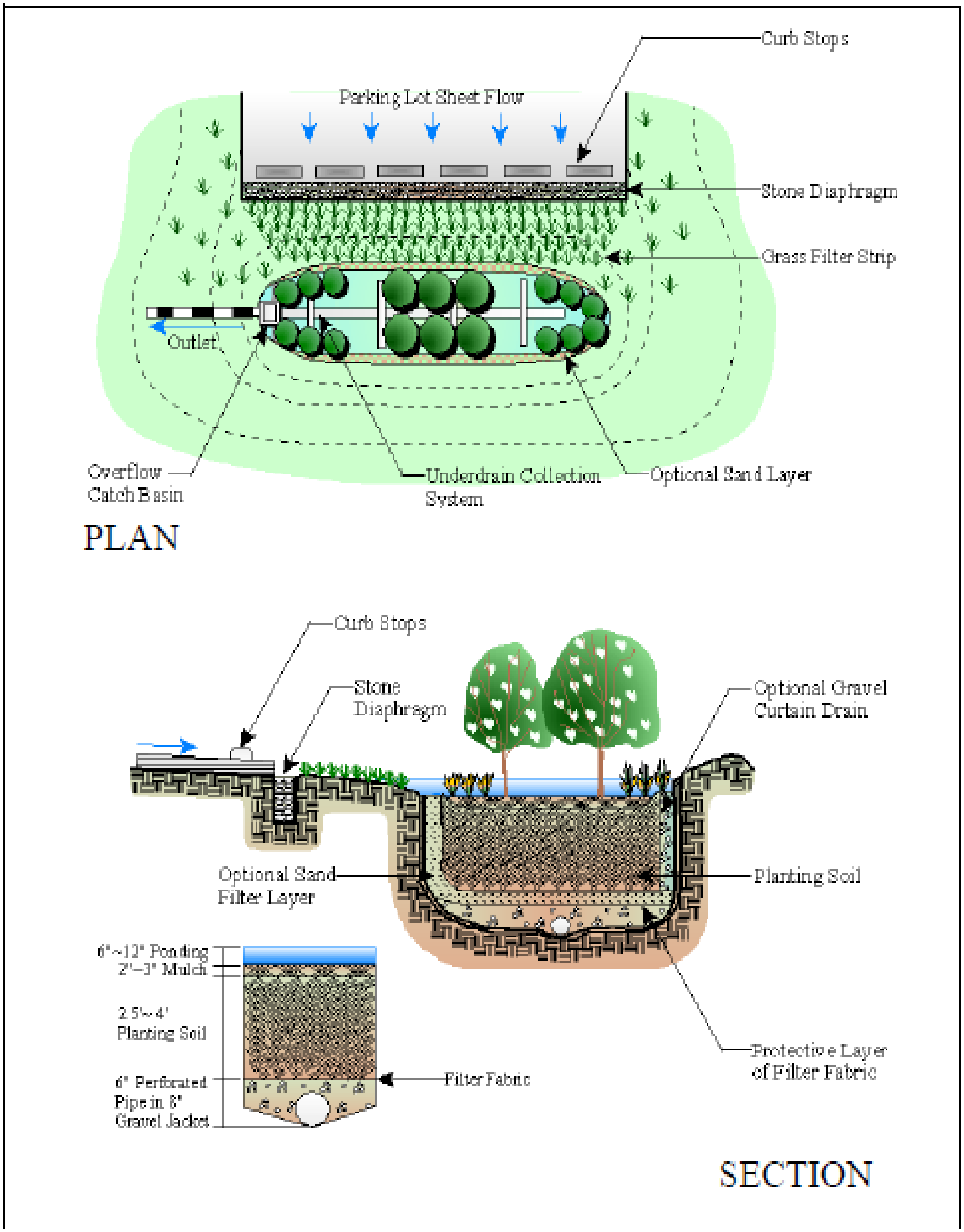


Figure 6: Bioretention facility plan and section view (Landphair et al 2000)

Pollutant Removal Potential:

Pollutant removal is primarily achieved through filtration and uptake from plants and microbials. Solid removal is high but has been shown to plateau at 10 $\frac{\text{mg}}{\text{L}}$ regardless of initial concentration (Lampe et al. 2005). Table 16 shows the pollutant removal from several studies.

Table 16: Pollutant removal potential for bioretention

Pollutant	Li and Davis[1] 2009 Removal %	Atchison et al. 2006 Removal %	MSSC 2008 Removal %	Davis et al.[2] 2009 Removal %	Passeport et al.[7] 2009 Removal %
Total Suspended Solids (TSS)	96	90	85	54–99[3]	-
	99				
Total Nitrogen (TN)	-3	65–75	45	32–97[4]	56
	97				47
Total Phosphorous (TP)	-36	80	50	-240–79[5]	53
	100				68
Total Metals (TM)	75	95	95	57–99[6]	
	99				
Fecal Coliforms	95	90	35	-	95
	100				85

1) 2 sites
2) Average of multiple studies
3) 5 studies
4) 5 studies
5) 7 studies
6) 5 studies for Zinc
7) 2 grass only sites

Cost Considerations:

Initial Cost:

Initial capital costs for bioretention facilities are considered low to moderate (WSDOT 2010). The city of Bellingham, Washington installed rain gardens in place of in-ground storage and saved 75–80% on construction costs (LeCroix et al. 2004). Bioretention facilities installed in 2004 cost \$12,800 to treat 4400 ft^3, which is equivalent to the WQV for 2.4 acres, and \$5,600 to treat 2300 ft^3, which is equivalent to the WQV for 1.3 acres (LeCroix et al. 2004). These costs were supported by the EPA who projected new construction of bioretention in commercial areas to be $12,357 and retrofits in commercial areas to be $12,355 for drainage areas no greater than 1 acre in 2004 (Clar et al. 2004). The precise initial capital requirement is site-specific and related to availability of materials, size of contributing drainage area, and necessity of under-drains.

Maintenance Costs:

The average expected maintenance cost for bioretention facilities was estimated to be $1,000 annually in 2004 (Lampe et al. 2004). Maintenance will need to be more rigorous, and therefore more costly, until plants can be established.

Maintenance costs can be tempered through community involvement. Because bioretention facilities are aesthetically pleasing, the public may be more prone to embrace and support their use. Community groups or business associations might be willing to participate in maintaining these BMPs. However, inspections and some maintenance activities would still be required.

Siting Constraints:

The flexibility of bioretention allows it to fit into most water treatment scenarios. Bioretention systems are very diverse and can be altered to site-specific conditions. The primary differentiation between types of bioretention systems are those which infiltrate the runoff and those which do not.

Infiltrating runoff benefits groundwater recharge as well as protects streams from erosion caused by high peak flows. However, infiltration is not always acceptable. The following instances do not allow for infiltration:

- The seasonal high ground water level is within 3 ft (0.9 m) of the bottom of the system (MSSC 2005)
- Treating a pollutant hot spot (i.e., gas station) where groundwater contamination is possible

- Inadequately drained subgrades (hydraulic conductivity ≤ 0.50 $^{in}/_{hr}$ (1.3 $^{cm}/_{hr}$))
- Potential interference with foundations/infiltration into basements
- Infiltration interferes with the subgrade of roadways

For applications that do not permit infiltration, under-drains can be used. Bioretention facilities with under-drains can be used in a wide variety of situations, and can be easily integrated into an urban landscape. When incorporating an under-drain, nearby structures must still be considered. If the bioretention cell is located adjacent to a building, roadway, or sidewalk, a concrete vault should be employed to prevent possibly harmful infiltration.

Maintenance and Operation Considerations:

Maintenance on bioretention BMPs focuses on keeping the plants healthy and preventing clogging of the filter media. Increased maintenance for these BMPs is required during the vegetation establishment period. Vegetation will require watering in times of little rainfall. Watering should be done weekly for the first 2–3 months and bi-weekly during summer months (Hartsig 2009). Table 17 shows expected maintenance and corrective procedures for the BMP.

Table 17: Operations and maintenance considerations and suggested corrective procedures for bioretention cells

Inspection Frequency	Problem	Suggested Corrective Procedure
Annually	Mulch layer thins	Evenly place mulch to a depth of 2–3 inches (5.1–7.6 cm) (Davis et al. 2009)
	Substantial rill formation	Fill rills with washed pea gravel and reconsider pretreatment to better attenuate flow velocity (DEPBWM 2006).
Semi-Annually	Burrowing animals cause vegetated cover to drop below 80%	Take applicable action which will vary with pest type.
	Sediment accumulation in fore-bay (if used)	Remove sediment and dispose of off-site (Clar et al. 2004).
Regularly/As Needed	Undesirable vegetation grows	All weeds and woody vegetation should be removed as soon as possible (SEMCOG 2008).
	Litter and detritus build up.	Remove and discard trash (MSSC 2008).
	Not enough rainfall to sustain vegetation.	Irrigation may be necessary to maintain adequate cover. It is suggested that vegetation be selected which is drought tolerant and will not require irrigation. Watering may be required to establish plants (LeCroix et al. 2004).
	Standing water beyond 48 hrs of isolated storm event.	Tilling the top layer should be done initially. If problems persist, remove filter media and replace with a better draining mix (NCDENR 2007).
	Vegetation becomes overgrown	Prune vegetation according to vegetation-specific requirements (Davis et al. 2009).
	Under drain clogs	Clean out pipes and dispose of sediment off-site (SEMCOG 2008).
	Under drain is damaged	Replace damaged pipe
	Vegetation is dead or diseased	Replace plants. If the plant species seems unsuited for this application select another species (Le Croix et al. 2004).

5.1.4 Basin Sand Filter

Figure 7: Sand filter for treatment of highway runoff (CalTrans 2004)

Description:

Basin sand filters are flow-through BMPs which temporarily detain the water quality volume (WQV) and filter it through sand. Treatment is accomplished primarily through filtration and secondly through sedimentation which occurs in a sedimentation chamber before the runoff is introduced to the filter media. Systems are typically designed for the sedimentation chamber to drain in 24 hours and the entire WQV to pass through the filter in 40 hours.

Sand filters are well suited to treat the first flush, but to avoid over-loading they should be designed so that flows in excess of the WQV bypass the system. They should not be used as in-line BMPs. Therefore, flow splitters should be employed upstream of the filter to prevent flows in excess of the WQV from entering the system.

Pollutant Removal Potential:

Sand filters have been shown to be very effective at removing sediment and metals from stormwater runoff. However, the moderate removal of nutrients provided by the sand filter prevents it from being a stand-alone BMP if discharging into nutrient impaired waterways. Observed removal rates are presented in Table 18.

Table 18: Pollutant removal potential for filter

	SEMCOG 2008[1]	MSSC 2005	CalTrans 2004	NCDENR 2007	Young et al. 1996
Pollutant	Removal %	Removal %	Removal %	Removal %	Removal %
Total Suspended Solids (TSS)	80–92	75–85	90	85	70–86
Total Nitrogen (TN)	30–47	0–35	32	35	31–47
Total Phosphorous (TP)	41–66	0–50	39	45	50–65
Total Metals (TM)	-	45–85	72[2]	-	78–84[4]
Hydrocarbons (oil and grease)	-	80	28[3]	-	-
1) 18 studies 2) Average of Pb, Cu, Zn			3) Average of TPH as oil and diesel 4) Average of Pb and Zn		

Cost Considerations:

Initial Cost:

Sand filters have relatively high construction costs. High costs are due in large part to construction costs for the concrete vaults which house many filters. These costs can be tempered by substituting earthen barriers or prefabricated vaults (SEMCOG 2008). Cost estimates have projected the treatment costs to be $16,000 per impervious contributing acre for filters less than 2 acres in 2002 (FHA 2002b). The cost-benefit of using prefabricated vaults is shown by a study which found costs of approximately $10,000 to treat 0.8 acres in 2008 (SEMCOG 2008).

Contributing watershed size is a major factor in the cost-effectiveness of sand filters. Watersheds greater than 10 acres are suggested to provide the greatest treatment value (Landphair et al. 2000). In 2002 the Federal Highway Administration projected initial filter costs to be $16,000 per impervious contributing acre when treating 2 acres or less and $3,400 per impervious contributing acre for watersheds greater than 5 acres (FHA 2002b).

Construction costs vary widely between studies. In a 2004 retrofit study, construction costs at 5 sites ranged from approximately $200,000 to approximately $315,000. The treated area in these sites ranged from 0.74 to 2.7 acres (CalTrans 2004). These wide ranges make it difficult to project construction costs based on area treated. Site-specific factors, such as excavation requirements can have effects on construction costs, and should be closely assessed when projecting facility costs.

Maintenance Costs:

A 2004 retrofit study projects that 43 hours will be spent servicing filters annually (CalTrans 2004), which corresponds with approximately $2,900 maintenance costs (CalTrans 2004). This budget is projected for years in which the filter media needs to be replaced. Since media rehabilitation is not an annual expense, maintenance costs will be lower on the off years.

Siting Constraints:

Applicable locations for sand filters include highway medians or within the roadway setbacks (Hubert et al. 2006). When being deployed near roadways, some safety concerns must be addressed. Sand filters or their components can act as fixed object hazards. Impact concerns can be mitigated by minimizing facility heights, employing appropriate setbacks, traffic barriers, and designing the structures to crumple when struck (Hubert et al. 2006).

Roadways and other transportation infrastructure, such as fueling and maintenance stations or park and rides are also ideal contributing watersheds because sand filters perform the best when treating runoff from highly impervious areas (MSSC 2005, DEPBWM 2006, CalTrans 2004). Sand filters may also be designed to occupy limited open space within right-of-ways or in an urban street setting where vegetated BMPs are impractical (Hubert et al. 2006). Although sand filters are adaptable for urban settings, industrial settings may be the most applicable due to a lack of aesthetic appeal compared to bioretention (MSSC 2005).

In order to facilitate gravity flow, and to avoid using pumps, there must be at least 3 ft (1 m) of elevation difference between the inlet of the system and the discharge point (Hubert et al. 2006, CalTrans 2004). The bottom of the facility should be at least 2 ft (0.61 m) above the high groundwater table to prevent possible facility damage and flooding of the underdrain (Hubert et al. 2006, CalTrans 2004). In areas where achieving sufficient heads causes interaction with the groundwater, the facility must be designed with sufficient mass to avoid buoyancy effects (Hubert et al. 2006). Leaching of groundwater into the system can be mitigated by lining the areas beneath the groundwater table with impervious geotextiles or using a concrete vault to house the filter.

Maintenance and Operation Considerations:

Sand filters should be inspected after the first storm of each year to ensure proper drainage and system functions (KCDNRP 2009). Inspections of contributing area should also be performed. If the contributing area is unstable or erosive the maintenance for the sand filter will be more intensive (Hubert et al. 2006). Removal of the top 2–5 inches (50–125 mm) of filter media is generally required every 3–5 years for properly designed filters (Landphair et al. 2000, MSSC 2005). The maintenance burden will be lower for contributing drainage areas with higher impervious areas, as there are typically fewer fines in the runoff (FHA 2002b). Table 19 shows potential operations and maintenance issues along with suggested procedures to correct them.

Table 19: Operations and maintenance considerations and suggested corrective procedures

Inspection Frequency	Problem	Suggested Corrective Procedure
Annually	Filter bed is not draining in design time	Manually manipulate surface, if this is inadequate remove top 2–5 inches (50–125 mm) and replace (if removal drops media depth under 18 inches (460 mm)). (MSSC 2005)
	Substantial channelization or rilling.	Fill any rills with sand and ensure level spreader is not clogged or damaged (KCDNRP 2009). If level spreader is in working order add erosion protection. (NCDENR 2007)
	Flow spreader is clogged or damaged	For clogs remove and dispose of sediment. For damage make necessary repairs or replace depending on severity. (NCDENR 2007)
Semi-Annually	Surface of media has hardened	Rake to break up surface. (Huber et al. 2006, SEMCOG 2008)
	Deterioration, spalling, or cracking of concrete	Patch damaged area. (Huber et al. 2006, MSSC 2005)
	6 inches (150 mm) or more of sediment built up in sedimentation chamber	Remove sediment. (MSSC 2005, Landphair et al. 2000)
Regularly/As Needed	Underdrains are clogged	Flush out underdrains (NCDENR 2007)
	Litter and detritus build up.	Remove litter and detritus. (NCDENR 2007)
	Contributing area is erosive	Stabilize contributing area. (Hubert et al. 2006)
	Flow diversion structure (if used) is clogged or damaged	For clogs remove and dispose of sediment. For damage make necessary repairs or replace depending on severity. (NCDENR 2007)
	Runoff is short circuiting the filter	Check clean out pipes and ensure there are no leaks in the filter or sediment chambers.

5.1.5 Horizontal Filter Trench

Description:

Horizontal filter trenches are sloped pea gravel-filled trenches which intercept runoff, pass it through the gravel filter media, and discharge it from the downstream end. Cobbles are used as armoring on top of the gravel-filled trench to prevent higher flows from washing away the pea gravel as well as slowing flows. The primary treatment processes in horizontal filter trenches is filtration, but infiltration can also be substantial depending on the characteristics of native soils.

The cobble armoring may not be sufficient for scour protection if flow velocities become too high. Therefore, stone check dams may be employed to slow the runoff. Check dams for horizontal filter trenches should not be earthen due to the potential for fines to migrate into and clog the filter. Rip-rap check dams function to slow runoff while not damaging the filter.

To ensure the filter trench is draining properly observation wells should be installed along the length of the trench. Observation wells will typically be 1–2 inch PVC pipe with perforations at the base. The PVC should be wrapped in filter fabric and capped to prevent clogging or contamination from outside sources. Figure 8 shows an observation well. Observation wells should be located at a minimum of 50 foot intervals for the length of the filter trench.

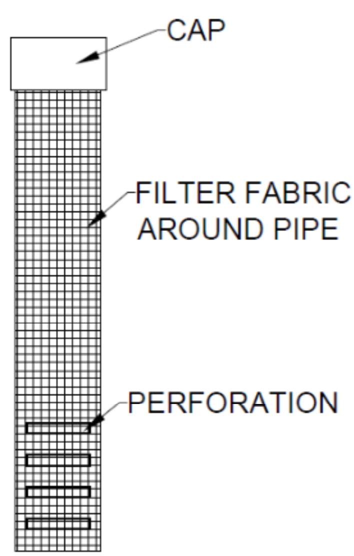


Figure 8: Observation Well

Pollutant Removal Potential

The horizontal filter trench is a BMP which is currently being developed for the Nebraska Department of Roads (NDOR), so there have not been opportunities to study pollutant removal potential. Horizontal filter trenches are expected to show high removal of solids, metals, and particulate phosphorous while nitrogen removal is expected to be low.

Cost Considerations:

Initial Cost:

Construction materials associated with horizontal filter trenches are well known, and accurate cost assessments can be made by contacting local vendors. Costs of materials in filter trenches include filter media, cobble armoring, geotextile, and PVC for the observation wells. Besides material costs, site preparation must be considered in cost assessments. The major costs of site preparation are excavation and stabilizing the contributing area.

The materials, processes, and designs required for construction of horizontal filter trenches are very similar to those required for construction of infiltration trenches, so reasonable cost estimates for the filter trench construction can be drawn from construction costs of infiltration trenches. Observations during a 2004 retrofit study indicated construction costs of nearly $150,000, or $21 per cubic foot treated (CalTrans 2004). The EPA estimated a lower cost of $5 per cubic foot of runoff treated in 2006 (EPA 2006c). Costs will vary with availability of aggregate.

Maintenance Costs:

Maintenance costs for horizontal filter trenches will also be similar to those for infiltration trenches. A 2004 retrofit study predicts 27 hours will be required annually for maintenance with costs of approximately $2,600 for a 4.9 acre contributing area (CalTrans 2004). Trench refurbishing costs are expected to be higher than initial construction costs.

Siting Constraints:

Horizontal filters are ideally located in long, narrow spaces with moderate slopes. Therefore, roadside applications are well suited for using horizontal filters. Existing roadside ditches are likely prime candidates for retrofit with horizontal filter trenches. Horizontal filter trenches can be incorporated into any swale or ditch system which has pretreatment for removal of particulates. The variability of sizing allows horizontal filter trenches to be incorporated into areas which may not otherwise be utilized (DEPBWM 2006).

For areas with flat topography the horizontal filter will act as an infiltration trench. Infiltration should not be allowed in the following circumstances:

- The seasonal high ground water level is within 3 ft (0.9 m) of the bottom of the system (MSSC 2005)
- Treating a pollutant hot spot (e.g., gas station) where groundwater contamination is possible
- Inadequately drained subgrades (hydraulic conductivity ≤ 0.50 $^{in}/_{hr}$ (1.3 $^{cm}/_{hr}$))
- Potential interference with foundations/infiltration into basements
- Infiltration interferes with the subgrade of roadways

If used where high solids loadings could occur, horizontal filter trenches should be located downstream of a pretreatment system which removes solids. When receiving sheet flow, vegetated filter strips are an ideal pretreatment. If remediating concentrated flow (e.g., end of pipe scenarios) a vegetated swale or rip-rap lined fore-bay can be employed. Pretreatment is important for these systems to prevent clogging with particulates and to avoid the large costs of rehabilitation.

Maintenance and Operation Considerations:

Maintenance associated with horizontal filter trenches focuses on limiting particulate loading to the trench. As with all BMPs, proper maintenance is required to extend the functional life of horizontal filter trenches and to prevent failure and costly rehabilitation. A summary of typical maintenance activities is provided in Table 20.

Table 20: Operations and maintenance considerations and suggested corrective procedures for horizontal filter trenches

Inspection Frequency	Problem	Suggested Corrective Procedure
Annually	Filter media clogs with sediment.	Remove and wash or replace clogged media.
	Filter fabric clogs.	Remove sediment from filter fabric. Cobbles may need to be replaced as well.
Semi-Annually	Trees growing near filter trench.	Remove woody vegetation without harmful chemicals and with minimal soil disturbance. Re-vegetate with grass as soon as possible (MSSC 2005).
	Erosion at the inlet or outlet of the trench.	Fill eroded area with cobbles.
	Solids deposit on cobble armoring.	Replace cobbles or wash in a location that does not drain to the trench.
	Check dam gets clogged with debris or sediment	Remove debris and replace rip-rap or wash in a location which does not drain into the trench.
Regularly/As Needed	Contributing area shows rilling or substantial erosion.	Reseed or otherwise stabilize contributing area.
	Weeds or unwanted vegetation begin to dominate the trench.	Weeds should be removed without using environmentally harmful chemicals (CalTrans 2004).
	Sediment build-up unacceptable in pretreatment (dependent on type of pretreatment).	Remove sediment from pretreatment.
	Litter and detritus build up.	Remove litter which is aesthetically unpleasant, negatively affects performance of the trench, or is itself harmful to the environment (FHA 1997b).

5.2 Design Guides

Once the fact sheets are reviewed and the ideal BMP for a site is selected, the BMP design guide is consulted to ensure proper use of the BMP. The design guides typically include:

- Design process: Provides the procedure for designing the BMP.
- Design criteria: Identifies BMP-specific design parameters.
- Design example: Provides an example site and performs the design process.

Design guides for the vegetated filter strip and vegetated swale will be based on the peak flow of the water to be treated. The bioretention cell and basin filter design will be based on the volume of water to be treated (WQV), and design of the horizontal filter trench will be based on the peak flow as well as the WQV.

5.2.1 Vegetated Filter Strip

Design Process:

Step 1: Evaluate applicability of vegetated filter strip considering site constraints.

Step 2: Calculate Peak Water Quality Flow (Q_{WQV}).

Step 3: Calculate Water Quality Flow Depth (D_{WQV}).

Step 4: Calculate Water Quality Flow Velocity (V_{WQV}).

Step 5: Check scour velocity for 10 year storm (V_S).

Step 6: Determine pretreatment method.

Step 7: Specify vegetation plan.

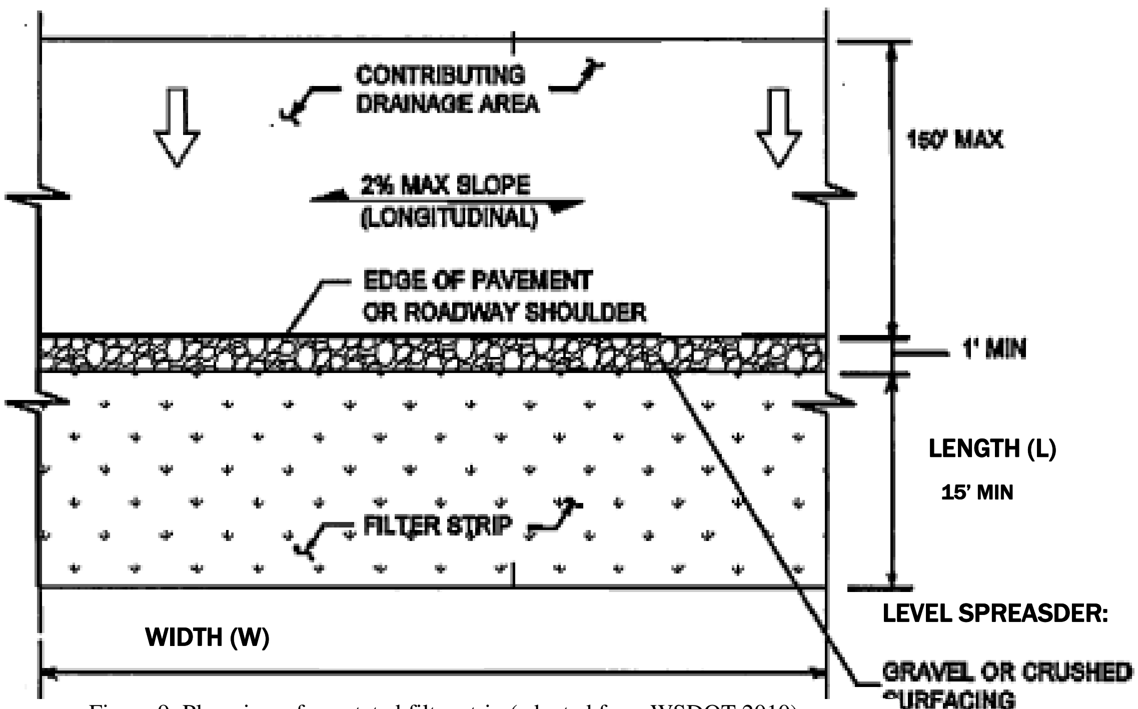


Figure 9: Plan view of vegetated filter strip (adapted from WSDOT 2010)

Design Criteria

Table 21 contains the criteria to be considered while working through the design process.

Table 21: Design criteria for vegetated filter strip

Design Parameter	Minimum	Maximum
Vegetated strip slope parallel to flow	2%[6,7,8]	15%[2,4,7,8]
Strip length (parallel to flow)	15 ft (4.6 m)[1,5,8]	Pollutant removal plateaus at 65 ft (20 m)[2,8]
Ground cover	80%[1]	-
Flow through strip	-	Must not cause erosion during events larger than the Water Quality Flow.
Side slope (perpendicular to flow)	-	2%[4,7]
Velocity through strip	-	$1\ \frac{ft}{s}\ (0.3\ \frac{m}{s})$[1,3]
Depth through strip	-	1 inch (0.39 cm)[1,4,7]
Runoff flow path before entering BMP	-	75 ft (23 m)[2,3] over impermeable surface or 150 ft (46 m)[2,3,4,7] over permeable surface.
1) Caltrans (2010a) 2) Clar et al. (2004) 3) FHA (2002a) 4) KCDNRP (2009)		5) Li et al. (2008) 6) MSSC (2005) 7) WSDOT (2010) 8) Zhang et al. (2010)

Step 1: Evaluate applicability of vegetated filter strip considering site constraints.

Vegetated filter strips can be applied adjacent to roadways, parking areas, or as an end-of-pipe (i.e., storm sewer outlet) BMP. They are best suited in locations where they can receive sheet flow from relatively horizontal surfaces such as parking lots or level roadways. When adjacent to roadways the cross slope (parallel to the roadway) is often the controlling factor in hilly areas. The cross-slope must be smaller than 2% in order for runoff to flow parallel to the design length of the strip. Locating a vegetated strip adjacent to a roadway in an urban setting may require too much area. If there is not enough space for the strip next to the road, it may be possible to install a vegetated filter strip as an end-of-pipe BMP, or another BMP more suited to an ultra-urban environment may be selected. For end-of-pipe applications vegetated filter strips generally must incorporate level spreaders and may require pretreatment such as sediment basins or velocity reduction systems. Design considerations for these facilities can be found in Step 6.

Outlet works for vegetated filter strips include unmanaged discharge directly into receiving waters or swale systems. Direct discharge may require slope stabilization, such as rip-rap if the slope to the waterway is susceptible to erosion. When direct discharge is not an option, a swale system may be constructed at the base of the strip to transport the runoff to receiving waters or another intermediate conveyance system such as a pipe. Adequately designed swales can also provide additional treatment. Design for swale systems can be found in the Vegetated Swale Design Guide section of this work.

Step 2: Calculate Peak Water Quality Flow (Q_{WQV}).

Peak flows have been calculated and displayed in Table 6 for impervious surfaces, such as pavement, up to 5 acres. For pervious areas or areas larger than 5 acres, peak flow rates are determined by using the 0.75 inch design storm with a type II NRCS 24-hour distribution and Equation 5-1(NRCS 1986). A detailed description of the use of this equation is given in section 4.3.

$$q_p = q_u A_m Q F_p \qquad (5\text{-}1)$$

Where:

q_p: Peak discharge (cfs)

q_u: Unit peak discharge $\left(\frac{cfs}{mi^2 * in}\right)$ (Figure 1 or Table 7)

A_m: Drainage area (mi^2)

Q: Runoff corresponding to 24-hr rainfall (in) (Table 8)

F_p: Pond or swamp adjustment factor (1.0 for Nebraska)

When considering a watershed with both impervious and pervious ground cover, the area can either be considered completely impervious, or a weighted flow may be calculated.

Assuming total imperviousness would result in larger than actual flows and, therefore, oversized BMPs. Therefore, the weighted flow method is recommended, as described in the Hydrology Section of this work.

Step 3: Calculate Water QualityFlow Depth (D_{WQV}).

The design flow depth can be calculated using the peak flow rate (Q_{WQV}) found in Step 2 and Equation 5-2, which is derived from the Manning equation (WSDOT 2010, Cal Trans 2010):

$$d = \left(\frac{Q_{wqf}n}{kWS^{1/2}}\right)^{3/5} \tag{5-2}$$

Where:

Q_{wqv}: Water Quality Flow (cfs or cms)

S: Slope parallel to flow $\left(\frac{ft}{ft} \text{ or } \frac{m}{m}\right)$

n: Manning's coefficient (0.24 for well-established dense grass (CalTrans 2010a))

k: constant (1 for Metric Units 1.486 for English Units)

W: Width of strip perpendicular to flow (ft or m)

d: Depth (ft or m)

Assuming that the width of the sheet flow is significantly larger than the depth, Equation 5-2 can be rearranged into Equation 5-3:

$$Q_{wqv} = \frac{k}{n} W d^{5/3} S^{1/2} \tag{5-3}$$

If the depth is greater than 1 inch (0.39 cm), measures need to be taken to reduce flow or to expand width; otherwise, vegetated filter strips should not be used (Caltrans 2010a, KCDNRP 2009, WSDOT 2010). Depths greater than 1 inch (0.39 cm) will not be effective in treatment and will pose a higher risk of scour.

For new construction or end-of-pipe considerations, solving for the minimum width may be beneficial. A maximum depth of 1 inch (0.39 cm) will be used to determine the minimum width of the filter strip. Solving for W, Equation 5-3 is reorganized into Equation 5-4:

$$W = \frac{Q_{wqv} n}{k S^{1/2} d^{5/3}} \tag{5-4}$$

For existing grass filter strips adjacent to roadways, the width generally coincides with the length of the roadway. This existing infrastructure should be checked against Equation 5-4 to determine if it will act as a properly designed vegetated filter strip.

Step 4: Calculate Water Quality Flow Velocity. (V_{WQV})

The flow rate and flow depth can be used to calculate the runoff velocity through the BMP with Equation 5-5.

$$V_{wqv} = \frac{Q_{wqv}}{Wd} \tag{5-5}$$

The velocity of the water quality volume (V_{wqv}) must be less than 1 $\frac{ft}{s}$ (0.3$\frac{m}{s}$) over the entire length of the filter strip (FHWA, Caltrans 2010a). Excess velocities will result in scour and short circuiting of the system. Short circuiting will adversely affect pollutant removal by not allowing the runoff to interact with an adequate amount of vegetation.

Step 5: Check scour velocity for 10-year storm (V_S)

Vegetated filter strips are often flow-through BMPs. This means that they will be required to facilitate flows greater than the water quality design flow. Vegetated filter strips must be able to accommodate these flows without being damaged.

Scour velocity will be calculated with the same process used for the water quality flow analysis (i.e., equations 5-1 to 5-5); however, scour velocity is calculated for the 10-year, 24 hour storm, which is 5 inches according to TP 40 (Hershfield 1961). The resulting velocity will then be compared to the values in Table 22, which show the scour velocities for common soil classes and their retardance classes. Retardance classes are defined in Table 23.

Table 22: Scour velocities in channels with various soil types and ground covers (USDA 1979)

Soil Texture	Bare Channel Scour Velocity (ft/s)	Vegetated Channel Scour Velocity (ft/s)			
		Retardance Class	Vegetation Condition		
			Poor	Fair	Good
Sand, silt, sandy loam, silty loam	1.5	B	1.5	3	4
		C	1.5	2.5	3.5
		D	1.5	2	3
Silty clay loam, sandy clay loam	2	B	2.5	4	5
		C	2.5	3.5	4.5
		D	2.5	3	4
Clay	2.5	B	3	5	6
		C	3	4.5	5.5
		D	3	4	2

Table 23: Ground cover retardance classes (Kilgore & Cotton 2005)

Retardance Class	Ground Cover	Condition
B	Kudzu	Very dense growth, uncut
	Bermuda Grass	Good stand, tall, average 300 mm (12 in)
	Native Grass Mixture (little bluestem, bluestem, blue gamma, and other long and short midwest grasses)	Good stand, unmowed
	Weeping lovegrass	Good stand, tall, average 610 mm (24 in)
	Lespedeza sericea	Good stand, not woody, tall, average 480 mm (19 in)
	Alfalfa	Good stand, uncut, average 280 mm (11 in)
	Weeping lovegrass	Good stand, unmowed, average 330 mm (13 in)
	Kudzu	Dense growth, uncut
	Blue Gamma	Good stand, uncut, average 280 mm (11 in)
C	Crabgrass	Fair stand, uncut 250 to 1200 mm (10 to 48 in)
	Bermuda grass	Good stand, mowed, average 150 mm (6 in)
	Common Lespedeza	Good stand, uncut, average 280 mm (11 in)
	Grass-Legume mixture--summer (orchard grass, redtop, Italian ryegrass, and common lespedeza)	Good stand, uncut, 150 to 200 mm (6 to 8 in)
	Centipede grass	Very dense cover, average 150 mm (6 in)
	Kentucky Bluegrass	Good stand, headed, 150 to 300 mm (6 to 12 in)
D	Bermuda Grass	Good stand, cut to 60 mm (2.5 in) height
	Common Lespedeza	Excellent stand, uncut, average 110 mm (4.5 in)
	Buffalo Grass	Good stand, uncut, 80 to 150 mm (3 to 6 in)
	Grass-Legume mixture-fall, spring (orchard grass, redtop, Italian ryegrass, and common lespedeza)	Good stand, uncut, 100 to 130 mm (4 to 5 in)
	Lespedeza sericea	After cutting to 50 mm (2 in) height. Very good stand before cutting.

Step 6: Determine pretreatment methods.

Vegetated filter strips may require pretreatment to slow runoff, remove coarse sediment, and evenly distribute flow over the width of the BMP. Level spreaders can be used to adequately address these three concerns. Runoff must be slowed and evenly distributed if it is entering the system as concentrated flow, or if it has traveled greater than 75 ft over impervious ground cover or greater than 150 ft over impervious ground cover (Clar et al. 2004).

When located adjacent to an impervious surface, a simple gravel trench, such as shown in Figure 10, is adequate as a level spreader. These trenches should be 1 foot (0.3 m) wide and 2–3 ft (0.61–0.91 m) deep. The fill gravel should consist of clean washed, uniformly graded coarse aggregate to the AASHTO # 3 specification (SEMCOG 2008). There should also be a 1–2 inch (2.5–5.1 cm) drop from the impervious surface to the trench (SEMCOG 2008).

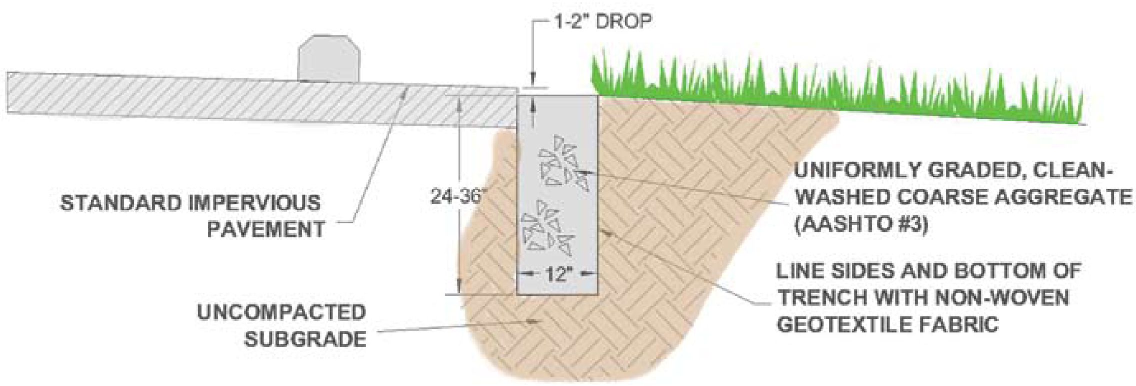


Figure 10: Level spreader adjacent to roadway or parking lot (SEMCOG 2008)

Level spreaders are made up of a trench with one edge which is lower and level allowing water to exit evenly along its length. This trench can be open or filled with gravel. If the trench is open, it is acceptable to line it with vegetation or concrete. Pipes discharging into the level spreader should be oriented parallel to the trench. Discharging into the trench lengthwise will minimize overloading and over-flow in a localized section of the level spreader. Figure 11 shows proper entrance to a flow spreader, and Figure 12 shows an improper entry angle.

Figure 12: Improper entry to level spreader (Winston et al. 2010)

Figure 11: Proper entry to level spreader (Winston et al. 2010)

The downstream (level) edge of the level spreader may be reinforced with treated wood, gravel, or concrete. Regardless of reinforcement the downstream edge must be level and straight to uniformly distribute the runoff. It must also be more than 1 inch (2.54 cm) lower than the uphill edge. If flow enters a level spreader as sheet flow the trench may be filled with evenly graded coarse aggregate. The gravel adds filtration as well as controlling mosquito breeding. Gravel may not be ideal for trenches which accept concentrated flow because the gravel would inhibit uniform filling of the trench, causing uneven discharge along the length of the level spreader.

The storage volume in the level spreader must be large enough to adequately handle and distribute the peak runoff flows. Level spreaders designed for handling concentrated flow should not have depths exceeding 1 foot (0.3m), and they should be as wide as the vegetated filter strip it discharges into. Level spreaders should be wide enough to discharge the WQV flow, which was found with Equation 5-1, without exceeding a flow depth of 1 inch (2.54 cm). Equation 5-4 can be used to find the minimum width of the level spreader. Gravel-filled level spreaders, which are ideal for handling sheet flow, may be 2–3 feet (0.6–0.9 m) deep (SEMCOG 2008).

Overflow bypass should be provided for large flows. The manner of bypass structure will be largely dependent on the BMP's surroundings. Bypass solutions may include a spillway at the end of the trench which discharges into a swale or under-drains discharging into a sewer system. Drainage measures must be implemented in open-channel level spreaders to allow draw-down within 24 hours to control mosquitoes. Vegetated trenches may need an under-drain if local soils do not allow for the infiltration of the design storm within the required 24 hours. The under-drains should discharge into the same structure as the overflow.

Step 7: Specify vegetation plan.

The vegetation in vegetated filter strips should be able to survive periods of saturation and periods of drought. Plants must also be able to withstand salts associated with deicing processes necessary in Nebraska's seasonal climate. Vegetation should be limited to grasses, or other vegetation which provides low ground cover. Nebraska's regional climate and soil compositions make it impractical to identify a single seed mix for the entire state. The Nebraska Department of Roads (NDOR) has established 6 landscape regions and has determined applicable grass mixtures for each. These suggested mixes are presented in Appendix A.

Design Example

A 2-lane highway is being constructed which adds 0.5 acres of impervious area. There is an existing 30 ft adjacent grass strip at an 8% slope away from the roadway. The longitudinal slope of the highway, and subsequent cross slope of the vegetated filter strip, is 1%. Figure 13 shows the plan view for this design example.

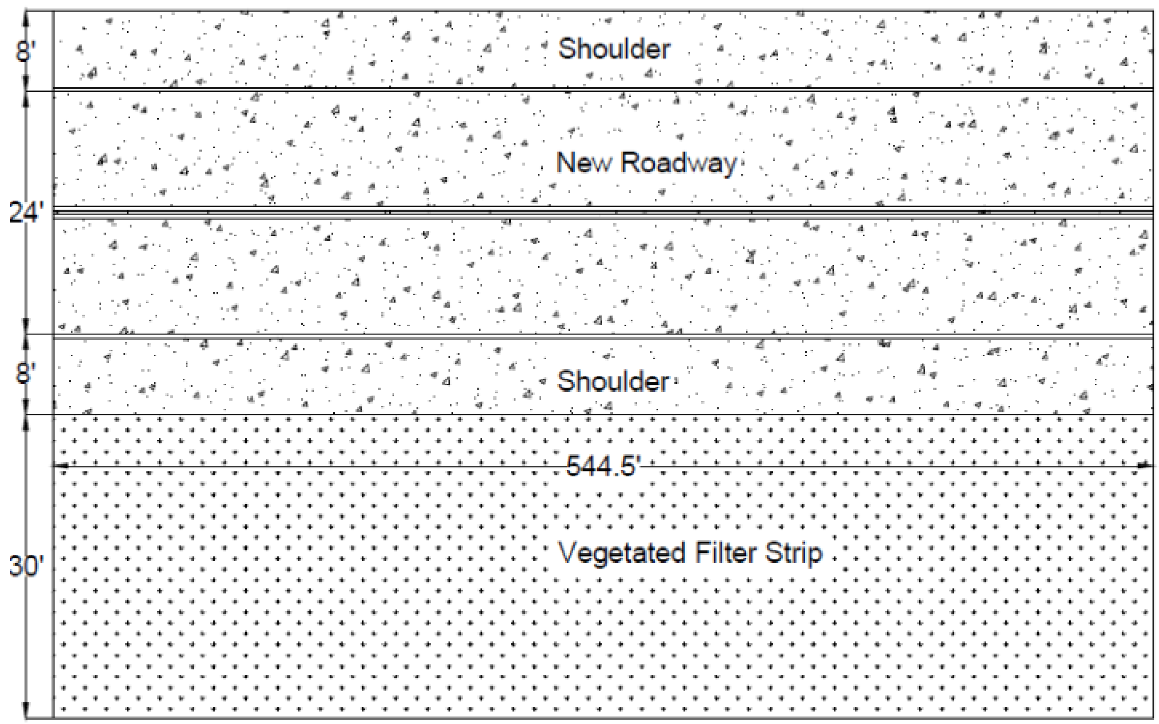


Figure 13: Plan view for vegetated filter strip design example 1

Step 1: Evaluate applicability of vegetated filter strip considering site constraints.

The lack of other structures in the right-of-way and acceptable slopes make this an ideal site to employ a vegetated filter strip adjacent to the roadway.

Step 2: Calculate Peak Water Quality Flow (Q_{WQV}).

Table 6 shows that the design peak water quality flow is 0.474 cfs for 0.5 impervious acres.

Step 3: Calculate Water Quality Flow Depth (D_{WQV}).

Equation 5-2 is used to determine the flow depth:

$$d = \left(\frac{Q_{wqv}n}{kWS^{1/2}}\right)^{3/5} = \left(\frac{0.474\frac{\text{ft}^3}{\text{s}} * 0.24}{1.486 * 544.5\ ft * \sqrt{0.08}}\right)^{3/5} = 0.125\ in$$

Using Q_{wqv}= 0.474 $\frac{\text{ft}^3}{\text{s}}$, n = 0.24, k = 1.486, S = 0.08, and W = 544.5 ft, d = 0.01 ft.

The width of the vegetated filter is equivalent to the length of the roadway. Two 12 ft wide lanes with an 8 ft shoulder were assumed for this example. The calculated depth of 0.125 inches is less than the maximum of 1 inch, so the width is satisfactory.

<u>Step 4</u>: Calculate Water Quality Flow Velocity (V_{WQV}).

Equation 5-5 is used to determine the flow velocity.

$$V_{wqv} = \frac{Q_{wqv}}{Wd} = \frac{0.474\ \frac{\text{ft}^3}{\text{s}}}{544.5\ \text{ft} * 0.125\ \text{in}} = 0.084\ \frac{\text{ft}}{\text{s}}$$

Using Q_{wqv}= 0.474 $\frac{\text{ft}^3}{\text{s}}$, W = 544.5ft, and d = 0.125 inches: $V_{wqv} = 0.084\ \frac{\text{ft}}{\text{s}}$, the calculated velocity is less than the 1 $\frac{\text{ft}}{\text{s}}$ maximum, so it is acceptable.

<u>Step 5</u>: Check scour velocity (V_S).

The 10-year 24-hour storm is used to check scour velocities. Peak flow will be found using Equation 5-1:

$$q_p = q_u A_m Q F_p$$

Table 4 shows a curve number of 98 for impervious areas. The curve number is then used with the ratio of initial abstraction (I_a) to precipitation (P) to find the unit peak discharge (q_u). Figure 1 or Table 7 can both be consulted for the q_u value. A curve number of 98 produces a q_u of 1100 $\frac{cfs}{mi^2 * in}$. Table 9 shows a runoff depth (Q) of 4.76 in for the 10-year storm. The swamp adjustment factor (F_p) for the state of Nebraska is 1. Using Equation 5-1 gives:

$$q_p = 1100\ \frac{cfs}{mi^2 * in} * 0.00078\ mi^2 * 4.76\ \text{in} * 1 = 4.08\ \text{cfs}$$

Equation 5-2 is then used to find flow depth:

$$d = \left(\frac{Q_{10-yr}n}{kWS^{1/2}}\right)^{3/5} = \left(\frac{4.08\frac{\text{ft}^3}{\text{s}} * 0.24}{1.486 * 544.5\ ft * \sqrt{0.08}}\right)^{3/5} = 0.455\ in$$

Equation 5-5 is then used to find the flow velocity:

$$V_{10-yr} = \frac{Q_{10-yr}}{Wd} = \frac{4.08\ \frac{\text{ft}^3}{\text{s}}}{544.5\ \text{ft} * 0.455\ \text{in}} = 0.2\ \frac{\text{ft}}{\text{s}}$$

The calculated value is less than any value on Table 22 and therefore passes for any ground condition. For example, a fair stand of Kentucky Bluegrass, which has a retardance class of C according to Table 23, in a silty loam soil would be adequate as it resists velocities of 2.5 $\frac{\text{ft}}{\text{s}}$.

Step 6: Determine pretreatment methods.

Because the runoff did not travel 75 feet or more over an impervious surface before entering the filter strip it will enter as sheet flow, which does not require pretreatment. Had the runoff traveled over 75 feet, a 1 foot wide, 2 feet deep, gravel filled level spreader would be a sufficient pretreatment.

Step 7: Specify vegetation plan.

A grass mixture should be selected which can survive the climatic and roadway conditions (e.g., salt) expected at the site. Suggested mixtures are described in Appendix A.

Design Example 2

A 2-lane highway is being constructed which adds 0.5 acres of impervious area. There is little adjacent land area available, and acquiring it would be prohibitively expensive. However, there is ample room at the outfall, so an end-of-pipe vegetated filter strip will be employed. A slope of 8% will be used for the vegetated filter strip. Figure 14 shows the plan view for design example 2.

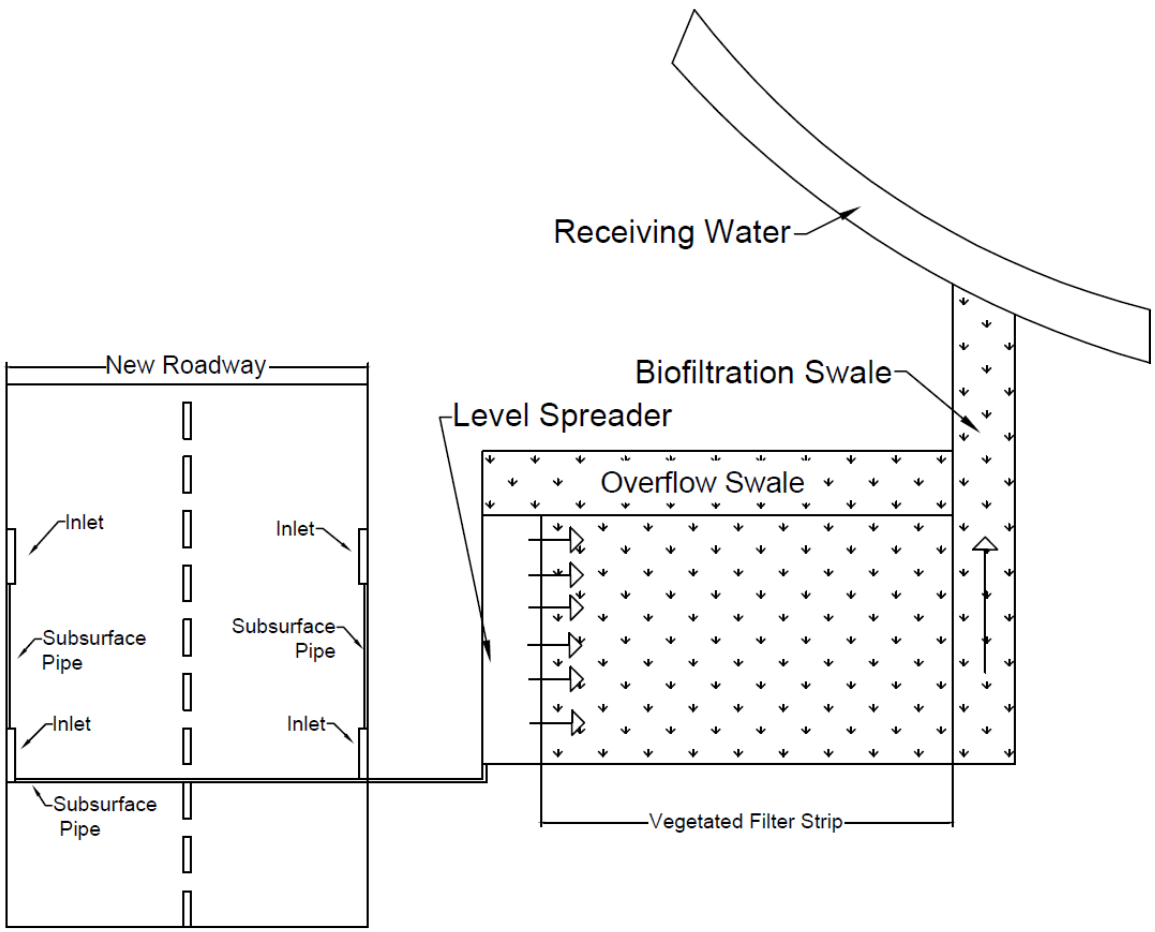


Figure 14: Plan view for vegetated filter strip design example 2

Step 1: Evaluate applicability of vegetated filter strip considering site constraints.

The lack of available space next to the road requires the vegetated filter strip to be used off-site as an end-of-pipe BMP.

<u>Step 2</u>: Calculate Peak Water Quality Flow (Q_{WQV}).

Table 6 gives a design peak flow of 0.474 cfs for 0.5 impervious acres.

<u>Step 3</u>: Calculate Water Quality Flow Depth (D_{WQV}).

For end-of-pipe applications, the filter width must be calculated. The minimum width is found using Equation 5-4 and assuming the flow depth to be the maximum 1 inch.

$$W = \frac{Q_{wqv}n}{KS^{1/2}d^{5/3}} = \frac{0.474\ \frac{\text{ft}^3}{\text{s}} * 0.24}{1.486 * 0.08^{1/2} * 1\ inch^{5/3}} = 17ft$$

Using $Q_{wqf} = 0.474\ \frac{\text{ft}^3}{\text{s}}$, n = 0.24, k = 1.486, S = 0.08, and d = 1 in, $W = 17\ ft$.

<u>Step 4</u>: Calculate Water Quality Flow Velocity (V_{WQV}).

Equation 5-5 is used to determine the flow velocity.

$$V_{wqv} = \frac{Q_{wqv}}{Wd} = \frac{0.474\ \frac{\text{ft}^3}{\text{s}}}{17\ ft * 1\ \ inch} = 0.334\ \frac{\text{ft}}{\text{s}}$$

Using $Q_{wqv} = 0.474\ \frac{\text{ft}^3}{\text{s}}$, W = 17 ft, and d = 1 in, $V_{wqv} = 0.334\ \frac{\text{ft}}{\text{s}}$, which is < 1 $\frac{\text{ft}}{\text{s}}$ so it is acceptable.

<u>Step 5</u>: Check scour velocity (V_S).

Using a vegetated filter strip in this configuration will not require a scour check, because it is not set up as a flow through BMP. An overflow weir is located 1 inch above the lip of the level spreader to allow the WQV to discharge at its maximum allowable depth while allowing excess flows to bypass. The level spreader configuration is shown in Figure 15.

<u>Step 6</u>: Determine pretreatment methods.

The runoff is being transported as concentrated flow, so a level spreader must be employed to slow and evenly distribute the design flow. The level spreader will be a trapezoidal trench with 3:1 side slopes, 1 ft of depth, and a bottom width of 2 ft. The downstream (level) edge of the trench will be reinforced by treated lumber and gravel armoring. Overflow bypass will be provided by a rectangular weir at the end of the trench, which is 1 inch higher than the edge of the filter strip. The overflow weir will discharge into a swale running parallel with the filter strip and discharge into the same receiving water. Figure 15 shows the level spreader set-up.

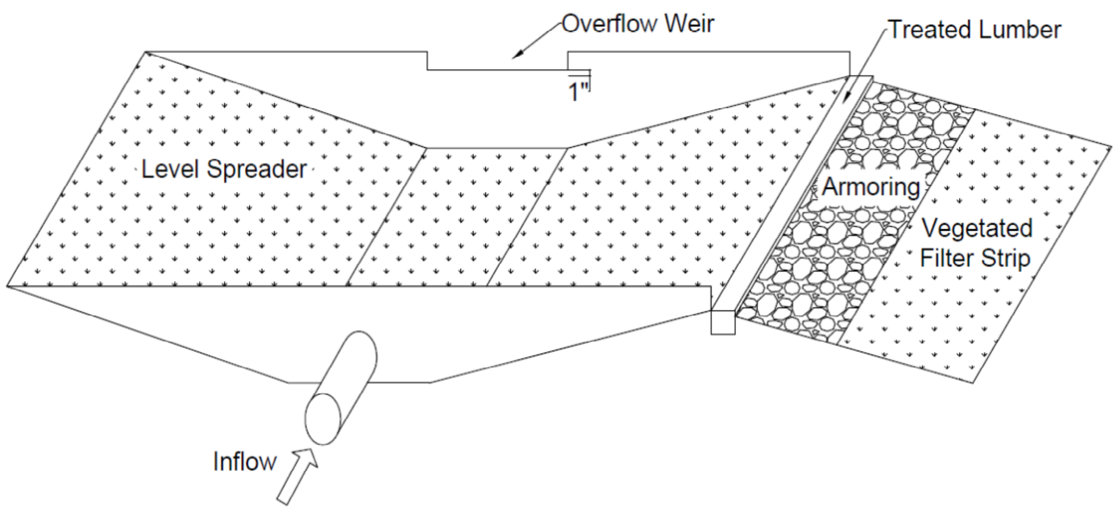


Figure 15: Level spreader for vegetated filter strip design example 2

<u>Step 7</u>: Specify vegetation plan.

A grass mixture should be selected which can survive the climatic and roadway conditions (e.g., salt) expected at the site. Suggested mixtures are described in Appendix A.

5.2.2 Vegetated Swale

Design Process:

Step 1: Evaluate applicability of vegetated swale considering site constraints.

Step 2: Calculate Peak Water Quality Flow (Q_{WQV}).

Step 3: Dimension the swale.

Step 4: Calculate the Water Quality Flow Depth (D_{WQV}).

Step 5: Calculate Water Quality Flow Velocity (V_{WQV}).

Step 6: Check scour velocity for 10 year storm (V_S).

Step 7: Design and position check dams (if necessary).

Step 8: Specify vegetation plan.

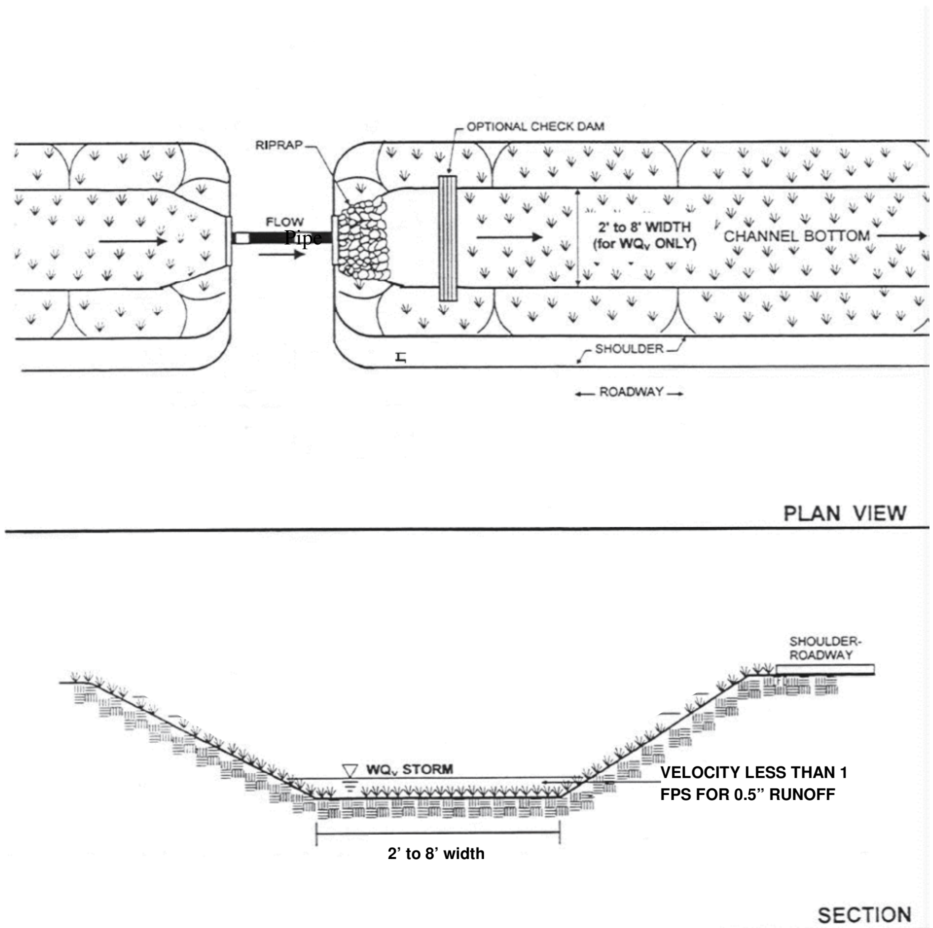


Figure 16: Plan and profile view of vegetated swale (adapted from Clar et al. 2004)

Design Criteria

Table 24 contains the criteria to be considered while working through the design process.

Table 24: Design criteria for vegetated swale

Design Parameter	Minimum	Maximum
Bottom Channel Slope of Swale	1%[4,5,6]	5%[1,3,5,7]
WQV Flow Depth Across Swale	-	4 in[4,5]
WQV Velocity Parallel to Swale	-	$1\ \frac{ft}{s}$[1,2,4,7]
Bottom Channel Width of Swale	2 ft[1,2,4,5,6,7]	8 ft[1,5,6]
Channel Side Slope	-	3:1[2,5,6,7]
1) CalTrans (2010b) 2) CalTrans (2004) 3) Clar et al. (2004) 4) KCDNRP (2009)		5) MSSC (2005) 6) SEMCOG (2008) 7) WSDOT (2010)

Step 1: Evaluate applicability of vegetated swale considering site constraints.

Vegetated swales may not provide enough treatment to be considered a stand-alone BMP (EPA 2006b). However, when site conditions are satisfactory, vegetated swales are a significant and viable BMP. They are particularly useful where soils are relatively permeable (NRCS hydrologic soil groups A through C); soils should have infiltration rates of 0.18 $\frac{in}{hr}$ (4.5 $\frac{mm}{hr}$) or higher (Landphair et al. 2000). Vegetated swales are often effectively located up or down stream of other BMPs. When upstream they provide pretreatment by filtering out debris and other solids. When employed downstream they provide additional treatment while transporting the treated runoff from the primary BMP to a discharge point. In addition to the treatment benefits, vegetated conveyance systems are more aesthetically pleasing than concrete-lined channels.

The linear nature of vegetated swales makes them excellent treatment and conveyance systems for runoff from roadways. Roadway drainage systems may already be functioning swale systems, or they may be easily retrofit for pollutant removal (CalTrans 2003).

Vegetated swales may not be suited for ultra-urban areas due to the necessity for relatively large areas. For densely developed areas, pipes are likely a more efficient and cost effective conveyance system, as they do not require as much area.

Step 2: Calculate Peak Water Quality Flow (Q_{WQV})

Peak flows have been calculated and displayed in Table 6 for impervious surfaces, such as pavement, up to 5 acres. For pervious areas or areas larger than 5 acres, peak flow rates are determined by using the 0.75 inch design storm with a type II NRCS 24-hour distribution and Equation 5-6 (NRCS 1986).

$$q_p = q_u A_m Q F_p \tag{5-6}$$

Where:

q_p: Peak discharge (cfs)

q_u: Unit peak discharge $\left(\frac{cfs}{mi^2 * in}\right)$ (Figure 1 or Table 7)

A_m: Drainage area (mi^2)

Q: Runoff corresponding to 24-hr rainfall (in) (Table 8 for WQV)

F_p: Pond or swamp adjustment factor (1.0 for Nebraska)

When considering a watershed with both impervious and pervious ground cover, the area can either be considered completely impervious, or a weighted flow may be calculated. Assuming total imperviousness would result in larger than actual flows and, therefore, oversized BMPs. Therefore, the weighted flow method is recommended, as described in the Hydrology Section of this work.

Step 3: Dimension the swale

Swale dimensions include the channel's bottom width, side slopes, and longitudinal slope. The design guidelines and limitations for these parameters are presented in Table 24. Swale dimensions will largely rely on site-specific considerations and existing drainage strategies.

Step 4: Calculate the Water Quality Flow Depth (D_{WQV})

Once the shape of the swale is decided upon, Equation 5-7 (Manning's Equation) can be applied to determine flow depth (NRCS 1986).

$$Q_{wqv} = \frac{k}{n} A R^{2/3} S^{1/2} \tag{5-7}$$

Where:

Q_{wqv}: Peak Water Quality Flow (cfs or cms)

S: Slope in direction of flow $\left(\frac{\text{ft}}{\text{ft}} \text{ or } \frac{\text{m}}{\text{m}}\right)$

R: Hydraulic Radius $\left(R = \frac{A}{P_w}\right)$

A: Cross sectional area of flow (ft^2 or m^2)

P_w: Wetted Perimeter (ft or m)

n: Manning's coefficient (0.24 for well-established dense grass (Caltrans 2010))

k: constant (1 for Metric Units; 1.486 for English Units)

The necessary equations for the elements of trapezoidal cross-sections can be found in Table 25.

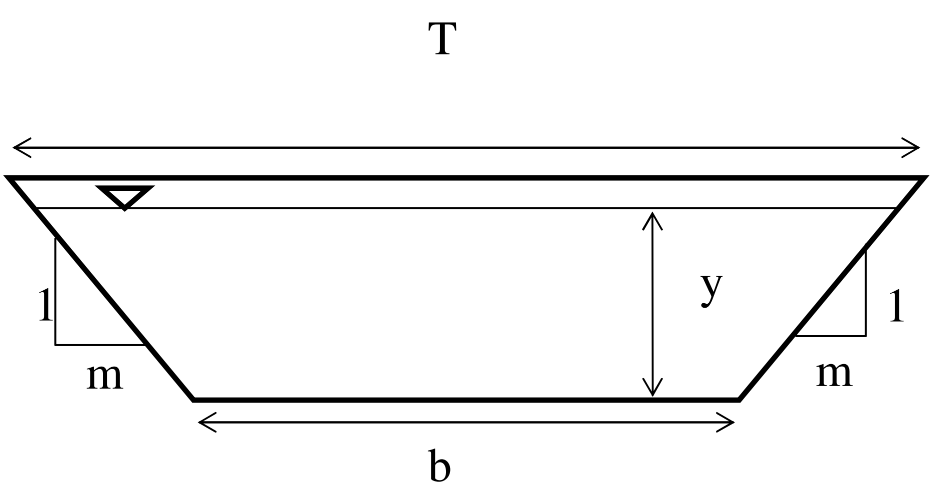


Figure 17: Reference shape for Table 25

Table 25: Geometric elements of trapezoidal cross section (Adapted from WSDOT 2010)

Area of flow (A) (ft^2 or m^2)	$(b+my)y$
Wetted perimeter (P_w) (ft or m)	$b+2y\sqrt{1+m^2}$
Hydraulic radius (R) (ft or m)	$\frac{(b+my)y}{b+2y\sqrt{1+m^2}}$

Inserting these geometric elements into Equation 5-7 results in Equation 5-8:

$$Q_{wqv} = \left(\frac{k}{n}\right) * (b+my)y * \left[\frac{(b+my)y}{b+2y\sqrt{1+m^2}}\right]^{2/3} * S^{1/2} \tag{5-8}$$

Equation 5-8 with the peak water quality flow found in Step 2 and the dimensions decided upon in Step 3 can be used to verify whether the depth of the flow will be less than 4 inches (7.6 cm) (Table 24). If the depth is > 4 inches the swale will need to be redimensioned, or check dams can be employed.

Step 5: Calculate Water Quality Flow Velocity (V_{WQV})

The velocity of the flow through the BMP can be determined with Equation 5-9 through the flow rate and the cross-sectional area of flow. The cross-sectional area can be found using Table 25.

$$v = \frac{Q_{wqv}}{A} \tag{5-9}$$

The velocity for the water quality flow parallel to swale should not exceed 1.0 $\frac{ft}{s}$ (Table 24). Higher flows will result in less treatment of the runoff.

<u>Step 6:</u> Check scour velocity for 10 year storm (V_S)

Vegetated swales are often flow-through BMPs. This means they will be required to handle flows greater than the water quality flow. Vegetated swales must be able to accommodate these flows without being damaged.

Scour velocity is found using the same methodology as the WQV velocity (steps 2 through 5). However, scour velocity analysis is performed based on the 10-year, 24 hour storm. For the state of Nebraska, the maximum rainfall depth for the 10-year, 24 hour storm is 5 inches according to TP 40 (Hershfield 1961). The resulting velocity (calculated using steps 2 through 5) is then compared to the values in Table 26, which shows the appropriate scour velocities for common soil classes and their retardance classes. Retardance classes are defined in Table 27.

Table 26**:** Scour Velocities in channels with various soil types and ground covers (USDA 1979)

Soil Texture	Bare Channel Scour Velocity (ft/s)	Vegetated Channel Scour Velocity (ft/s)			
		Retardance Class	Vegetation Condition		
			Poor	Fair	Good
Sand, silt, sandy loam, silty loam	1.5	B	1.5	3	4
		C	1.5	2.5	3.5
		D	1.5	2	3
Silty clay loam, sandy clay loam	2	B	2.5	4	5
		C	2.5	3.5	4.5
		D	2.5	3	4
Clay	2.5	B	3	5	6
		C	3	4.5	5.5
		D	3	4	2

Table 27: Ground cover retardance classes (Kilgore & Cotton 2005)

Retardance Class	Ground Cover	Condition
B	Kudzu	Very dense growth, uncut
	Bermuda Grass	Good stand, tall, average 300 mm (12 in)
	Native Grass Mixture (little bluestem, bluestem, blue gamma, and other long and short midwest grasses)	Good stand, unmowed
	Weeping lovegrass	Good stand, tall, average 610 mm (24 in)
	Lespedeza sericea	Good stand, not woody, tall, average 480 mm (19 in)
	Alfalfa	Good stand, uncut, average 280 mm (11 in)
	Weeping lovegrass	Good stand, unmowed, average 330 mm (13 in)
	Kudzu	Dense growth, uncut
	Blue Gamma	Good stand, uncut, average 280 mm (11 in)
C	Crabgrass	Fair stand, uncut 250 to 1200 mm (10 to 48 in)
	Bermuda grass	Good stand, mowed, average 150 mm (6 in)
	Common Lespedeza	Good stand, uncut, average 280 mm (11 in)
	Grass-Legume mixture-summer (orchard grass, redtop, Italian ryegrass, and common lespedeza)	Good stand, uncut, 150 to 200 mm (6 to 8 in)
	Centipede grass	Very dense cover, average 150 mm (6 in)
	Kentucky Bluegrass	Good stand, headed, 150 to 300 mm (6 to 12 in)
D	Bermuda Grass	Good stand, cut to 60 mm (2.5 in) height
	Common Lespedeza	Excellent stand, uncut, average 110 mm (4.5 in)
	Buffalo Grass	Good stand, uncut, 80 to 150 mm (3 to 6 in)
	Grass-Legume mixture-fall, spring (orchard grass, redtop, Italian ryegrass, and common lespedeza)	Good stand, uncut, 100 to 130 mm (4 to 5 in)
	Lespedeza sericea	After cutting to 50 mm (2 in) height. Very good stand before cutting.

Step 7: Design and position check dams (if necessary)

Check dams may be necessary to keep the WQV velocity below 1 $\frac{ft}{s}$. Check dams are installed perpendicular to the flow. Although certain check dams provide some treatment through sedimentation or filtration, those effects are secondary to velocity dissipation and are not the focus of check dam design.

Roadside check dams should be easily maintained while not interfering with maintenance of the swale itself. Swale mowing operations, in particular, should not be adversely affected by the check dams. This is done by maintaining small slopes (5:1 to 10:1 (Clar et al. 2004)) on the up and downstream sides of the check dams, respectively. The low slopes also prevent check dams from being a hazard to motorists who could potentially crash into or ramp off of them.

A roadside check dam can be constructed by installing rip-rap, railroad ties, wood chips, or a vegetated berm across the width of a swale. Regardless of the material, the check dam height should not exceed 2 feet (0.61 m) (Landphair et al. 2000 ; Clar et al. 2004). A 1 ft (0.3 m) wide gravel trench may be required to protect the downstream edge of the check dam from erosion (Landphair et al. 2000). This trench will serve as a flow spreader to evenly distribute flows and act as armor for the soil. Figure 18 shows an example of a check dam design. It is important for the top of the check dam to be level, so it can evenly distribute detained flows. If flows are allowed to concentrate, erosion will occur, and the check dam will have a negative effect on both the flow and water quality.

Some check dams may require an under-drain or weep holes to discharge runoff trapped after storm events. Areas with NRCS soil types A,B, or C can safely assume that any trapped water will infiltrate prior to providing mosquito breeding habitat.

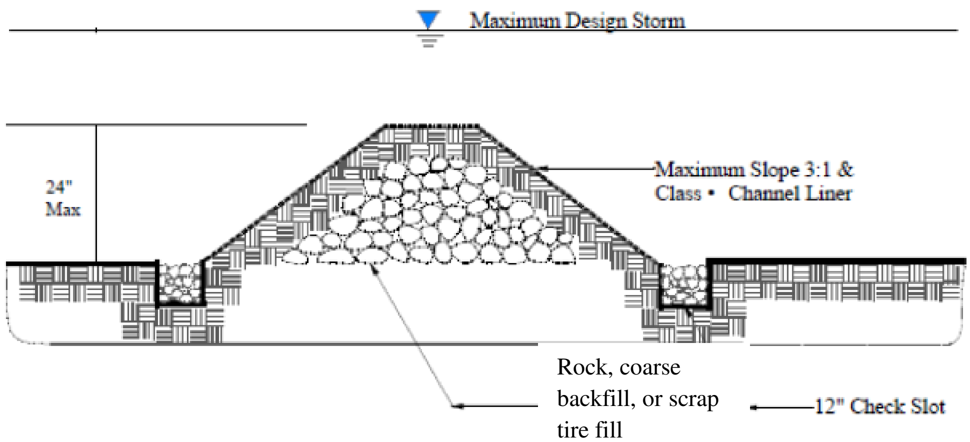


Figure 18: Check dam cross-section (Landphair et al. 2000)

The minimum spacing of check dams should be such that the lower edge of an upstream check dam is at the same elevation as the peak of a downstream check dam, as shown in Figure 19. Equation 5-10 is used to calculate the minimum check dam spacing (Landphair et al. 2000).

$$L = \frac{h}{g} \tag{5-10}$$

Where:

L: Minimum horizontal distance between check dams (ft or m)

h: Height of check dam (ft or m)

g: Longitudinal channel slope $\left(\frac{ft}{ft} \text{ or } \frac{m}{m}\right)$

It is suggested (Landphair et al. 2000) that the check dams be placed at six times the minimum required distance. Spacing should, therefore, be found with Equation 5-11:

$$L = 6 * \frac{h}{g} \tag{5-11}$$

Spacing of check dams should also help maintain sheet flow in the BMP. Sheet flow typically channelizes after 150 feet (45.7 m) of flow over pervious ground cover (Clar et al. 2004); therefore, a check dam should be located every 150 feet (45.7 ft) regardless of whether flow velocities are calculated to be large enough to create scour (Clar et al. 2004).

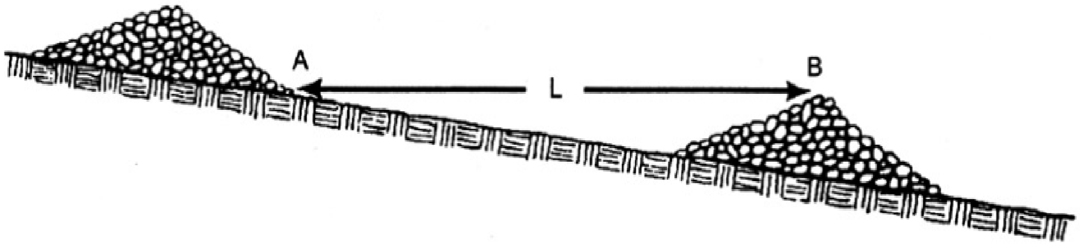


Figure 19: Minimum check dam spacing (BE 2001)

Step 8: Specify vegetation plan.

The vegetation in vegetated swales should be able to survive periods of saturation and also be drought resistant. Plants must also be able to withstand salts associated with deicing processes necessary in Nebraska's seasonal climate. Vegetation should be limited to grasses, or other vegetation which provides low ground cover. Nebraska's regional climate and soil compositions make it impractical to identify a single seed mix for the entire state. The Nebraska Department of Roads (NDOR) has established 6 landscape regions and determined applicable grass mixtures for each. These suggested mixes are presented in Appendix A.

Design Example

A 0.5 mile long, 2 lane highway (Area = 1.94 ac; CN = 98) is being constructed, as shown in Figure 20. The highway drainage system will also have to handle run-on from an 8 foot wide grass segment running parallel to the highway (Area = 0.97 ac; CN = 80). A vegetated swale which has a longitudinal slope of 3% is being considered as a conveyance BMP for runoff from the highway which has passed through an end-of-pipe vegetated filter strip. The swale must transport the runoff 200 feet before discharging into receiving waters. To simplify the example, calculations will be done assuming no infiltration occurs in the filter strip.

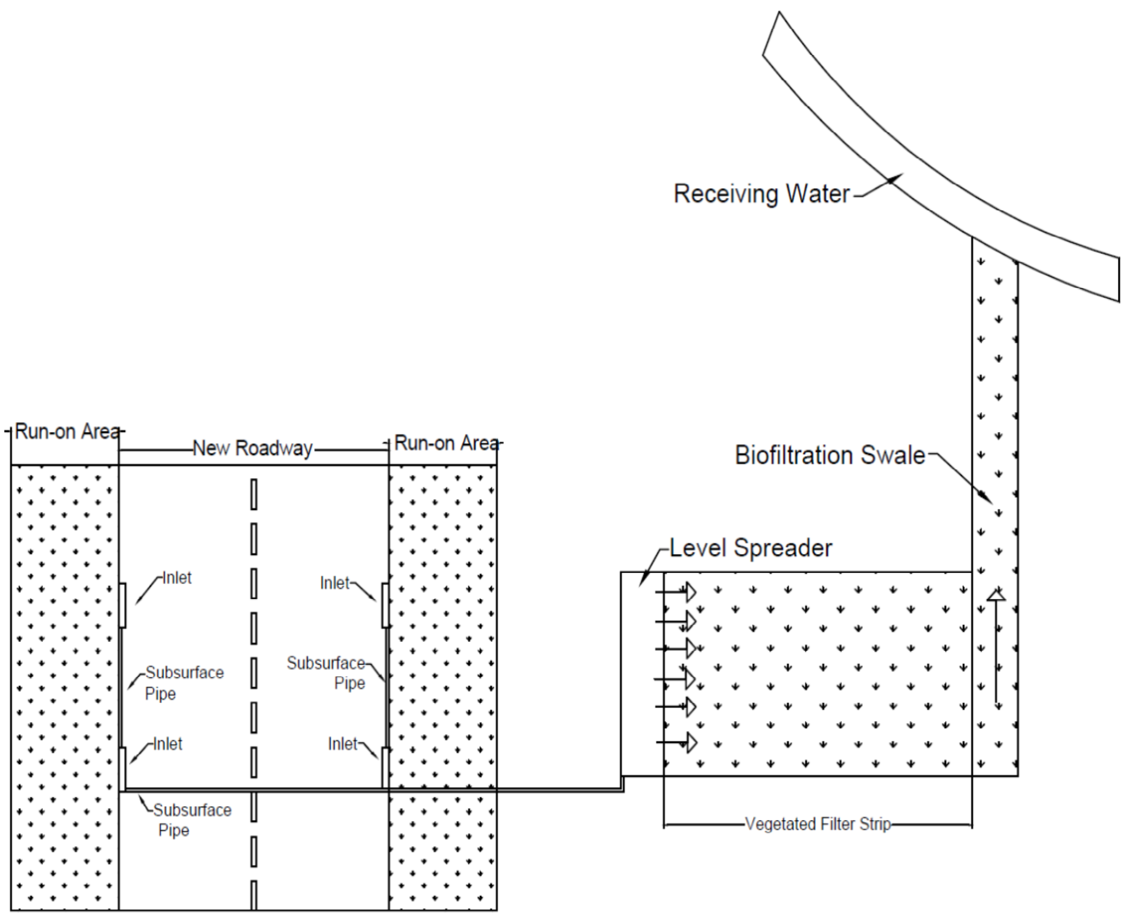


Figure 20: Plan view of vegetated swale design example

Step 1: Evaluate applicability of vegetated swale considering site constraints.

A drainage ditch was selected to convey the flows from the vegetated filter strip to the receiving water. The drainage ditch can be designed so that it acts as a vegetated swale, thereby treating the water as it is conveyed.

Step 2: Calculate Peak Water Quality Flow. (Q_{WQV})

Interpolation of Table 6 shows that the peak water quality flow is approximately 1.84 cfs from an impervious area of 1.94 acres. Table 8 shows that there will be 0.023 inches of runoff from the run-on areas from the WQV storm. Equation 5-6 is then used to determine the flow from run-on:

$$q_p = q_u A_m Q F_p = 550 \frac{\text{cfs}}{\text{mi}^2 * \text{in}} * 0.0015\ \text{mi}^2 * 0.023\ \text{in} * 1 = 0.019\ \text{cfs}$$

The Hydrology Section of this work contains the values for q_u in Table 7 or Figure 1 and Q in Table 8 for the WQV, F_p is 1 for Nebraska.

The flow from the new development is then added to the run-on flow to find the flow occurring at the WQV storm, which results in a total flow of 1.86 cfs.

$$1.84\ \text{cfs} + 0.019\text{cfs} = 1.86\text{cfs}$$

Step 3: Dimension the swale

Propose a side slope of 4:1 (Table 24, max m = 3:1) with an 8 foot bottom width, as shown in Figure 21, and a longitudinal slope of 3% which matches the existing topography. If the WQV depth (from Design Step 4) or velocity (from Design Step 5) is not satisfactory, increase the bottom width and/or side slopes to reduce the values until they are within the requirements in Table 24.

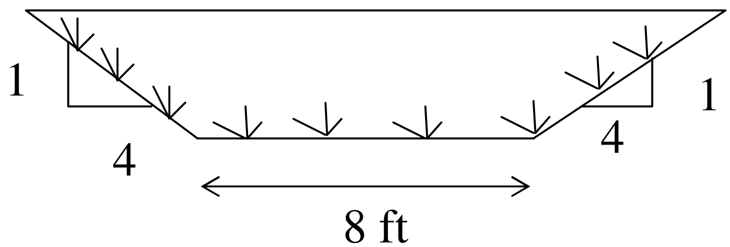


Figure 21: Design example swale cross-section

Step 4: Calculate Design Flow Depth (D_{WQV}).

Equation 5-7 and the geometric elements associated with this channel were combined to obtain Equation 5-8:

$$q_p = \left(\frac{k}{n}\right) * (b+my)y * \left[\frac{(b+my)y}{b+2y\sqrt{1+m^2}}\right]^{2/3} * S^{1/2}$$

$$1.86 = \left(\frac{1.49}{0.24}\right) * \left((8+4y)y\right) * \left[\frac{(8+4y)y}{8+2y\sqrt{1+4^2}}\right]^{2/3} * (0.03)^{1/2}$$

The depth (y) was calculated to be 4.6 inches which is more than 4 so it is unacceptable based on the parameters in Table 24. The channel bottom width is already at the maximum allowable shown in Table 24 so either the side slope should be increased or check dams should be used to improve the design. Here, check dams will be employed to slow velocities and prevent rilling along the swale.

Step 5: Calculate Design Flow Velocity. (V_{WQV})

The flow and area are utilized to determine the velocity with Equation 5-9:

$$v = \frac{Q}{A} = \frac{Q}{\left((8+4y)y\right)} = \frac{1.86}{\left((8+4*0.383)0.383\right)} = 0.5\ \frac{ft}{s}$$

Using the WQV flow of 1.86 cfs and the depth of 4.6 inches (0.383 ft) as found in the previous step, the velocity is 0.5 $\frac{ft}{s}$, which is less than 1 $\frac{ft}{s}$; therefore it is satisfactory.

Step 6: Check scour velocity for 10 year storm. (V_S)

The scour velocity is checked using the same process as the WQV design but with a 10-yr, 24-hr storm. The first step is to determine the peak flow. Peak flow is found with Equation 5-6:

$$q_p = q_u * A_m * Q * F_p$$

Values for q_u for various CNs are found in Table 7, values for Q are found in Table 9.

Flow contribution from new construction (CN = 98)

$$q_p = 1100 \frac{\text{cfs}}{\text{mi}^2 * \text{in}} * 0.003\text{mi}^2 * 4.76 \text{ in} * 1 = 15.7 \text{ cfs}$$

Flow contribution from run-on (CN = 80)

$$q_p = 1000 \frac{\text{cfs}}{\text{mi}^2 * \text{in}} * 0.0015 \text{ mi}^2 * 2.89 \text{ in} * 1 = 4.3 \text{ cfs}$$

The contributing flows are summed to find a total peak flow (q_p) of 20 cfs.

The flow depth in this BMP is found with Equation 5-8:

$$20 = \left(\frac{1.49}{0.24}\right) * \left((8+4y)y\right) * \left[\frac{(8+4y)y}{8+2y\sqrt{1+4^2}}\right]^{2/3} * (0.03)^{1/2}$$

The flow depth (y) in this BMP is found to be 1.4 feet. The depth is then used to find the area which is used with the calculated flow to obtain velocity by the following equation:

$$v = \frac{Q}{A} = \frac{Q}{\left((8+4y)y\right)} = \frac{20}{\left((8+4*1.4)1.4\right)} = 1.1 \ \frac{\text{ft}}{\text{s}}$$

The velocity is found to be 1.1 $\frac{\text{ft}}{\text{s}}$ which is less than the limiting velocities for all parameters shown in Table 26.

Step 7: Design check dams (if necessary)

Since the flow depth for the water quality storm was unacceptable, a check dam is required. A check dam is also required because the swale has a length greater than 150 feet. The check dam height will be 6 inches, to mitigate the unacceptable 4.6 inch flow depth for the WQV storm. Equation 5-11 is used to determine spacing of the check dams:

$$L = 6 * \frac{h}{g} = 6 * \frac{0.5 \text{ ft}}{0.03} = 100 \text{ ft}$$

The calculated spacing of 100 ft is acceptable because it does not allow flows to travel greater than 150 ft, which is the estimated length where rills begin to form for flows over pervious surfaces. The check dam will have a 5:1 front slope and 10:1 back-slope. This swale is being installed in NRCS type B soil so any water detained by the check dam after a rainfall event will infiltrate. An earthen check dam will be used. Establishment and maintenance of vegetation on the check dam will coincide with the vegetated swale.

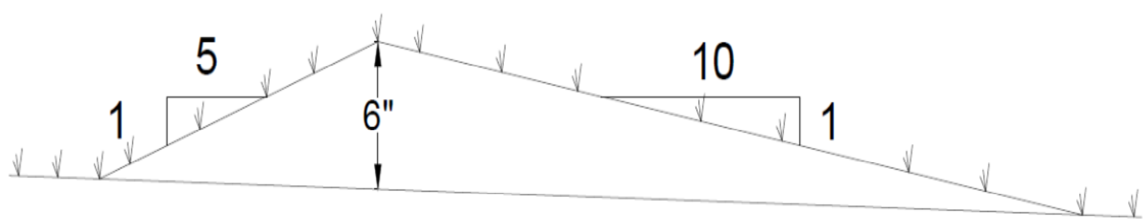


Figure 22: Swale design example check dam profile

<u>Step 8</u>: Specify vegetation plan.

A grass mixture should be selected which can survive the climatic and roadway conditions (e.g., salt) expected at the site. Suggested mixtures are described in Appendix A.

5.2.3 Bioretention Cell

<u>Design Process</u>:

Step 1: Evaluate applicable location considering site constraints.

Step 2: Calculate water quality volume to be treated (WQV).

Step 3: Specify filter media type.

Step 4: Determine necessary media depth.

Step 5: Calculate surface area.

Step 6: Select dimensions for bioretention area.

Step 7: Design inlet system and pretreatment.

Step 8: Design under-drain (If necessary).

Step 9: Select and size overflow method.

Step 10: Specify vegetation plan.

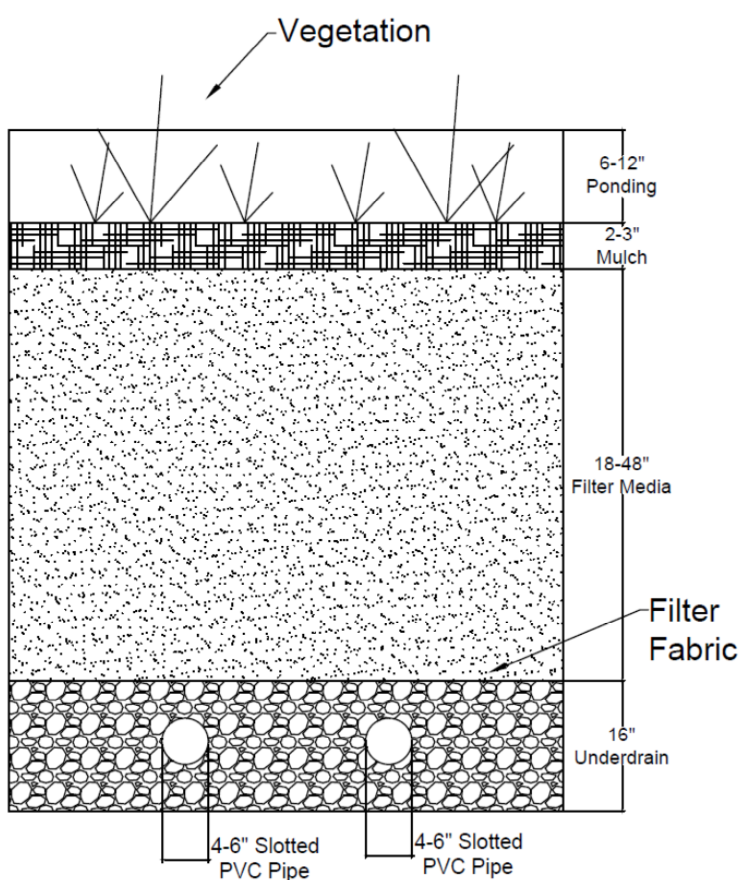


Figure 23: Bioretention cross-section

Design Criteria

Table 28 contains the criteria to be considered while working through the design process.

Table 28: Design Considerations

Design Parameter	Minimum	Maximum
Depth of ponding	6 inches[2,5]	12 inches[2,3,4,6]
Depth of amended filter media	18 inches[3,4,5]	48 inches[2,6]
Ponding drawdown time	24 hours[1,4,5]	48 hours[2,5]
1) Atchison et al. (2006) 2) Clar et al. (2004) 3) Hartsig and Rodie (2010)	4) Hinman (2005) 5) MDEP (2009) 6) NCDENR (2007)	

Step 1: Evaluate applicable location considering site constraints.

Bioretention is a flexible BMP which can be located in many locations, from a downtown setting to the interchange of a rural highway. Bioretention's pleasing aesthetics result in a socially acceptable means of treating runoff. Bioretention is also flexible in that it does not require a large or contiguous footprint. If a watershed is too large for a single cell, there are often multiple locations on-site to facilitate the use of multiple cells. Bioretention BMPs are also strong candidates for retrofit projects due to the adaptability of their layout.

Bioretention facilities can be designed as either infiltration or filtration BMPs. Infiltration is encouraged to facilitate ground water recharge. However, when the subsurface has a permeability less than 0.5 $\frac{\text{in}}{\text{hr}}$ (1.3 $\frac{\text{cm}}{\text{hr}}$), the bioretention cell will not drain properly and will function as a filter requiring an under drain (Davis et al. 2009). Under drains may also be included if infiltration will be detrimental to surrounding structures or roadways. Under drains should also be used when treating runoff from pollutant hot spots (e.g., gas stations).

Step 2: Calculate water quality volume to be treated (WQV)

The water quality volume (WQV) is the amount of runoff requiring treatment. The water quality volume is calculated by summing the volume which comes from newly constructed impervious areas and the volume of run-on from adjacent property which comingles with run off from the new development. The WQV can be found by summing Equations 5-12 and 5-13:

The volume from the new development is found with Equation 5-12:

$$WQV_{New\ Dev} = 0.5in * \frac{\text{Area Treated (ft}^2\text{)}}{12\frac{in}{ft}} \quad (5\text{-}12)$$

The volume of run-on is found with the following Equation 5-13:

$$WQV_{Run\text{-}On} = Q\ (in) * \frac{\text{Area Treated (ft}^2\text{)}}{12\frac{in}{ft}} \quad (5\text{-}13)$$

Q is the runoff depth found in Table 8.

Step 3: Specify filter media type

The filter media shall be a uniform mix, free of stones, stumps, roots or other similar objects larger than two inches (MDE 2000). Media in a bioretention cell needs to accommodate vegetation, drain adequately, and provide treatment. These goals can be accomplished with a variety of soil mixes, suggested by a variety of agencies. A common thread throughout is requiring a homogenous mix free of detritus or roots.

The Minnesota Pollution Control Agency provides two sets of soil media. The first is primarily based on water quality and is 55–65% construction sand, 10–20% top soil, and 25–35% organic leaf compost (MSSC 2008). The second mix is designed for enhanced filtration and includes 50–70% construction sand and 30–50% organic leaf compost (MSSC 2008). The water quality mix will have higher nutrient removal than the filtration mix, which is primarily designed to remove solids and metals. Construction sand for these two mixes should meet AASHTO M-6 or ASTM C-33 specifications (MSSC 2008), or have similar gradations as described in Appendix B. A bioretention garden design manual prepared for the Omaha region suggests a 50/50 mix of fine sand and compost or sphagnum peat mix (Hartsig and Rodie 2010). Loamy sand or sandy loam has been suggested by the North Carolina Department of Environment and Natural Resources (NCDENR 2007) and the Puget Sound Action Team (Hinman 2005), while the Maine Department of Environmental Protection suggests using a silty sand mix (MDEP 2009). The EPA has published specifications calling for loamy sand, sandy loam, or a loam, sand mix and notes that the minimum sand content should be 50%, and the maximum fines should be 10% (Clar et al. 2004). The EPA also states that amending the soil with 20–50% compost can be very beneficial for plant growth and pollutant removal (Clar et al 2004).

Selecting which mix is right for a certain location is at the discretion of the designing engineer. Site-specific problem pollutants should be considered as well as media cost. If nutrient removal is the primary concern, a higher percentage of compost and top soil should be used. However, if solids or metals are the main problem using a higher percentage of sand will result in adequate treatment.

Step 4: Determine necessary media depth

The depth of the filter media must be between 18 and 48 inches (45.7–121.9 cm) (Clar et al. 2004). The depth can vary depending on what types of pollutants require remediation. Metal concentrations have been shown to decrease exponentially while moving down through the soil column (Weiss et al. 2010). This was supported in another study which found that most metals accumulate within 4–8 in (10–20 cm) of the surface (Li and Davis 2008). Similar results were found for total suspended solids (TSS) removal. TSS was shown to be removed within 2–8 inches (5–20 cm) of the surface (Li and Davis 2008). Lab and field tests have both shown that petroleum hydrocarbons are removed and biodegraded primarily in the layer of mulch (Davis et al. 2010). Sorbed phosphorous removal coincides with TSS removal, and dissolved phosphorous removal begins at approximately 12 inches (30.5 cm) below grade (NCDENR 2007). Nitrogen removal has been shown to begin at around 30 inches (76 cm) (NCDENR 2007). Researchers in North Carolina suggest that the addition of a permanent saturated zone, at least 12 inches deep (30.5 cm), within the media can increase nitrogen removal by facilitating de-nitrification (NCDENR 2007). An anaerobic zone can be created by having the under drain discharge through an upturned pipe or a weir in the discharge area. Figure 24 shows general profiles of the riser pipe and weir method.

Media depth must also be thick enough to sustain the vegetation in the cell. Sufficient depth is needed for the root zone for the health of the plants and to keep roots away from the under drain system. Different types of vegetation have varying root penetration. Plant selection should be factored into selecting an adequate depth of filter media. Plants selection is discussed in Step 10 of this section.

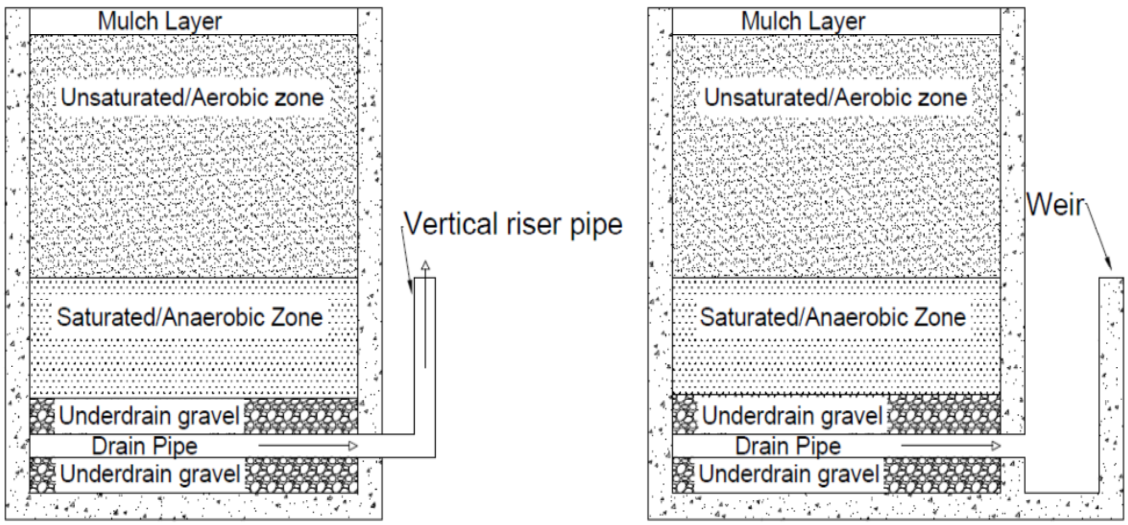


Figure 24: General saturated zone discharge designs

Step 5: Calculate surface area

The surface area of the bioretention facility must be large enough to accommodate the WQV while not exceeding the maximum ponding depth (6-12 inches). Equation 5-14 is used to determine the required surface area (NCDENR 2007):

$$A = \frac{WQV}{D_{Max\ Pond}} \tag{5-14}$$

Where:

A: Area of bioretention facility (ft^2 or m^2)

WQV: Water quality volume (ft^3 or m^3)

$D_{Max\ Pond}$: Maximum ponding depth (0.5 - 1 ft or 0.15 – 0.3 m)

Equation 5-14 conservatively calculates the required surface area due to the assumption that the entire WQV will require ponding before it enters the filter media. Equation 5-18 accounts for flow through the media and can be used if an appropriate hydraulic conductivity (K) can be found for the selected media (Clar et al. 2004).

Step 6: Select dimensions for bioretention area

The bioretention system needs to be sized in conjunction with the area found in Step 5. The required surface area does not need to be one centralized bioretention cell. The potential for division of the surface area over the watershed makes bioretention a flexible BMP. Although multiple cells can be employed, each bioretention cell must account for the first half inch of runoff from the sub-watershed draining into it. The Maine Department of Environmental Protection suggests that no single cell be greater than 2,000 ft^2 (186 m^2) (MDEP 2009).

Step 7: Design inlet system and pretreatment

Inflow to bioretention can be concentrated from a pipe, culvert, or curb, or it can enter the system as sheet flow. Bioretention cells receiving concentrated flow should incorporate a forebay which will slow runoff, reduce erosion, and function as pretreatment by allowing solids to settle out.

Figure 25: Properly utilized forebay (NCDENR 2007)

Figure 25 shows a properly constructed and utilized forebay. The volume of the forebay should be 0.05 inches (0.13 cm) multiplied by the impervious drainage area (Clar et al 2004). Rip-rap is suggested as lining for the forebay due to its drainage potential and its resistance to erosion during times of high flow.

Pretreatment will focus on removal of solids which could clog the media. Pretreatment methods for sheet flow include: grass filter strips, gravel diaphragms, or a mulch layer (MSSC 2005). Grass filter strips are excellent pretreatment systems, and their design can be found in the Vegetated Filter Strip Design Guide section of this work. Gravel diaphragm systems consist of a small gravel filled trench. These trenches should be at least 1 foot (0.3 m) wide and 2–3 ft (0.61–0.91 m) deep. The gravel fill should consist of clean washed, uniformly graded coarse aggregate to the AASHTO # 3 specification (SEMCOG 2008), as described in Appendix B. There should also be a 1–2 inch (2.5–5.1 cm) drop at the inlet to the gravel diaphragm (SEMCOG 2008). A layer of mulch can be used as pretreatment if grass is not selected as vegetation. The mulch should be 2–3 inches deep (5.1–7.6 cm) (MDEP 2009; Clar et al. 2004). Aged, shredded hard wood bark mulch is recommended (Clar et al. 2004).

When capturing runoff from gutters, a curb cut may be used, as shown in Figure 26. It is suggested to armor the entrance to the BMP from the curb cut to prevent erosion. Erosion needs to be avoided as it adds solids to the system which may result in clogging. Control measures for erosion include implementing a gravel diaphragm (as described above) or using rip rap. The rip rap in this case does not need to be as large as it does in forebays receiving concentrated flow. It can be decorative, as well as functional, and it can be used to complement the aesthetic appeal of the bioretention cell. Figure 26 demonstrates the use of aesthetically pleasing rip rap to prevent erosion using a curb cut.

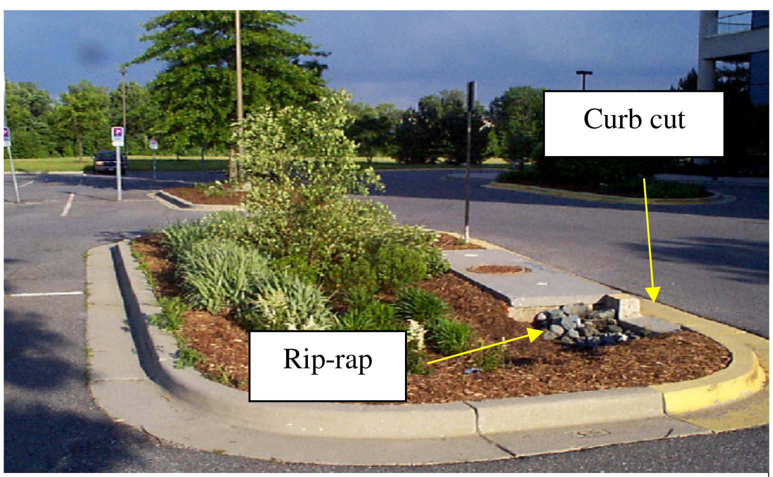


Figure 26: Curb cut inlet system (NCDENR 2007)

Curb cuts can be used in series to achieve a more uniform application to the bioretention cell. Using a series of curb cuts allows less flow, and velocity, entering at each location while also maintaining a curb for the majority of the roadway for traffic safety.

Step 8: Design under-drain (If necessary)

Bioretention facilities in areas where infiltration is an acceptable and possible alternative generally do not require under-drains. In fact, under-drains are not recommended in these situations to promote groundwater recharge and to decrease the impact of impervious areas on peak stream flows. However, the following situations will require the use of an under-drain:

- Inadequately drained subgrades (hydraulic conductivity ≤ 0.50 $^{in}/_{hr}$ (1.3 $^{cm}/_{hr}$)),
- Infiltration is harmful to surrounding structures (e.g., possible damage to foundations),
- The seasonal high groundwater table is within 3 ft (0.9 m) of the bottom of the bioretention cell (MSSC 2005),
- Treating a pollutant hot spot (e.g., gas station) where groundwater contamination is probable.

For situations where infiltration would be particularly harmful, a concrete vault is suggested to house the bioretention system. Not all systems which require an underdrain will call for a concrete vault encasement. Infiltration should not be avoided unless it is detrimental to the bioretention system or neighboring structures.

If required, the under-drain system will consist of 4–6 inch (10.2–15.3 cm) diameter slotted PVC pipes wrapped in geotextile and set in a 16 inch (40.6 cm) thick gravel bed at a 1% down slope to the outlet (NVPDC & ESI 1996). The gravel will over-top the pipes by at least 2 inches (5.1 cm) and conform to the AASHO #3 standard as described in Appendix B (VCSQMP 2001, NVPDC & ESI 1996). The pipes will be no more than 8 feet (2.4 m) apart (MDEP 2009).

There must also be a nonwoven geotextile layer between the BMP filter media and the under drain media. The geotextile must meet the specification presented in Table 29.

Table 29: Geotextile specifications (VCSQMP 2001)

Geotextile property	Specification	Test
Grab strength	90 lbs	ASTM D4632
Elongation at peak load	50%	ASTM D4632
Puncture strength	24 lbs	ASTM D3787
Permitivity	0.7 sec^{-1}	ASTM D4491
Burst strength	180 psi	ASTM D3786
Toughness	5500 lbs	% Elongation * Grab strength
Ultraviolet resistance	70%	ASTM D4355

Step 9: Select and size overflow method

Bioretention facilities can be designed as either on-line or off-line facilities. For on-line facilities any volume beyond the WQV must be allowed to bypass. For off-line facilities the WQV can be separated before it enters the system, while excess flows are allowed to bypass.

Flow splitters are the primary means for separating out the WQV before it enters the BMP. Figure 27 is a potential layout for a bioretention cell using a flow splitter. Flow splitters can use a weir overflow device that is generally located in either a manhole or vault, as shown in Figure 28. The elevation of the overflow weir is often set at the WQV elevation of the cell. Keeping these elevations constant will allow for bypass of flows beyond the allowable depth while ensuring the WQV enters the bioretention facility.

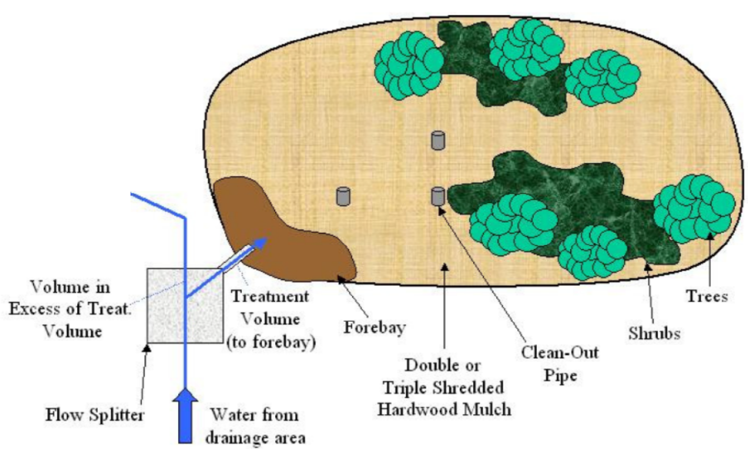


Figure 27: Off-line bioretention cell layout (adapted from NCDENR 2007)

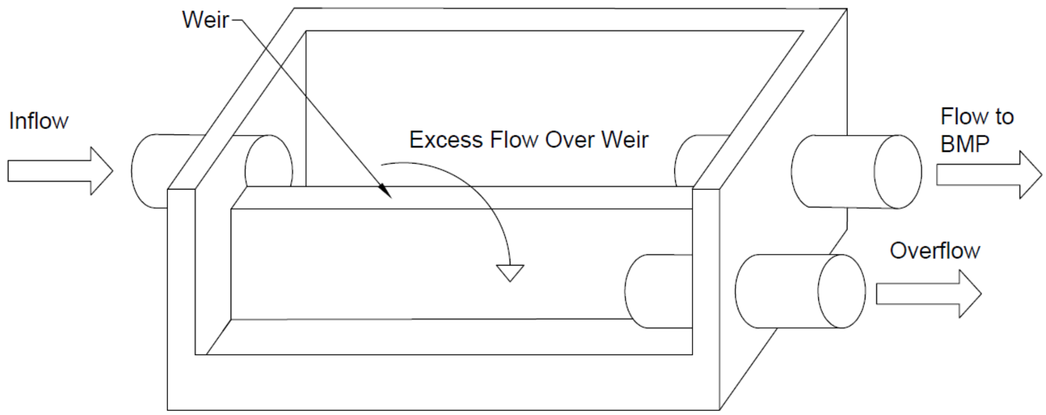


Figure 28: Flow splitter

The hydraulics of the flow splitter are very important design elements. A long weir is ideal to maximize flow rate while minimizing head. However, a longer weir will require a larger vault, which may not be as cost effective. The outlet pipe to the bioretention cell must be sized to pass the WQV regardless of storm intensity. If the pipe is inadequately sized, flows could back up and discharge over the weir prematurely.

When used as an on-line system, bioretention facilities should include an overflow structure, such as a weir or grate, to discharge excess runoff. Overflow structures should be sized to discharge volumes greater than the WQV. The outlet should be located at the design depth of the cell, which will ensure the WQV is trapped in the cell.

Step 10: Specify vegetation plan

Vegetation can be widely varied in bioretention cells. Although plants and shrubs are generally considered to be an integral part of the system, grass-only cells have been proven equally as effective in pollutant remediation (Davis et al. 2009), albeit without the aesthetic value which accompanies blooming plants.

The majority of vegetation used should be native to Nebraska or the Great Plains, although they can be integrated with non-native plants which are not intrusive and have proven they thrive regionally. *Nebraska Bioretention and rain Garden Plants Guide* is a publication which includes descriptions for a wide variety of applicable plants as well as their applications within bioretention facilities (Rodie & Todd 2010).

Design Example

Bioretention is selected as the BMP for a new roadway going through a developed downtown area. Bioretention was selected due to its flexibility in sizing and aesthetic benefits. A maintenance plan was developed with business owners to take care of day-to-day maintenance and monitoring of the cells. Figure 29 is the plan view showing the area.

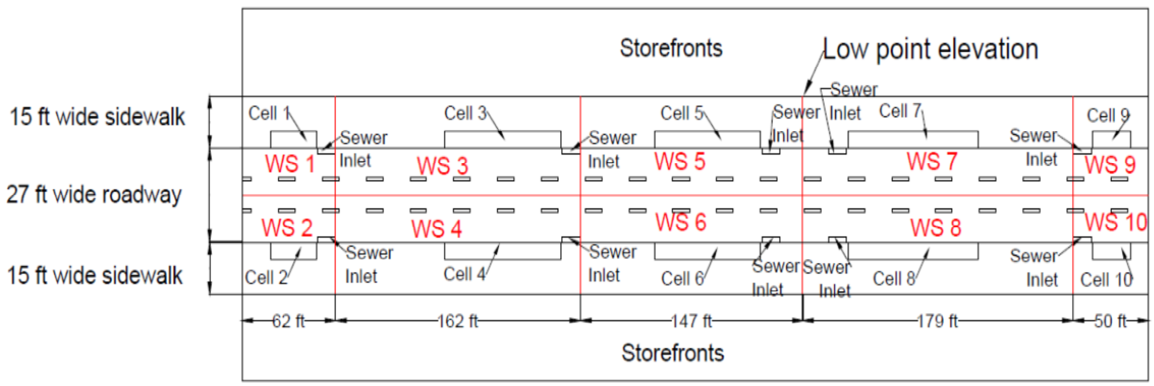


Figure 29: Site plan view for bioretention example

Step 1: Evaluate applicable location considering site constraints.

The watershed (i.e., the new roadway surface and area contributing run-on) has been broken down into 10 sub-watersheds labeled WS1-WS10 in Figure 29. The sub-watersheds discharge into the bioretention cells with the corresponding numbers. Each bioretention cell is responsible for treating the runoff from half of the new 27 ft wide roadway as well as the 15 ft wide sidewalk, no other run-on comingles with the roadway runoff. Each cell intrudes 5 ft into the sidewalk, which leaves 10 ft of walking room for pedestrian traffic at the bioretention areas.

Although the bioretention cells are not treating a pollutant hotspot, the seasonal high groundwater table is well below the bottom of the cells, and the subsurface has permeability greater than 0.5 $\frac{\text{in}}{\text{hr}}$, infiltration, for this example, cannot be used due to the harm it would cause the adjacent roadway and building foundations. Therefore, an under-drain system must be employed. In this situation a concrete vault should be employed to enclose each bioretention cell.

Step 2: Calculate water quality volume to be treated (WQV).

The water quality volume must be calculated for each sub-watershed. The sidewalks and roadway are impervious and have a curve number of 98. The area of each sub-watershed can be found by multiplying its length by half the width of the roadway (13.5 ft) for contributing drainage area from new development or the width of the sidewalk (15 ft) for the contributing run-on area. It should be noted that the watersheds are symmetrical from the center of the road and are calculated as such.

Contributing drainage area for new development in WS1:

$$A = L * W = 62 \text{ ft}*13.5 \text{ ft} = 837 \text{ ft}^2$$

Contributing drainage area for run-on for WS 1:

$$A = L * W = 62 \text{ ft}*15 \text{ ft} = 930 \text{ ft}^2$$

The volume from new impervious area of WS 1 is found by using Equation 5-12:

$$WQV_{\text{New Dev}} = 0.5\text{in}*\frac{837 \text{ ft}^2}{12\frac{\text{in}}{\text{ft}}} = 35 \text{ ft}^3$$

The run-on volume from the sidewalks in WS 1 is found by using Equation 5-13:

$$WQV_{\text{Run-on}} = 0.5 \text{ in}*\frac{930 \text{ ft}^2}{12\frac{\text{in}}{\text{ft}}} = 39\text{ft}^3$$

The total WQV of WS 1 is found by taking the sum of Equation 5-12 and Equation 5-13:

$$35 \text{ ft}^3 + 39\text{ft}^3 = 74\text{ft}^3$$

Table 30 shows the WQVs for the other sub-watersheds.

Table 30: WQV for each sub-watershed

WS 1 & 2	WS 3 & 4	WS 5 & 6	WS 7 & 8	WS 9 & 10
74 ft^3	192 ft^3	175 ft^3	213 ft^3	59 ft^3

Step 3: Specify filter media type.

The filter media for each bioretention cell will consist of 60% clean washed AASHTO M-6 sand, 5% fines, and 35% compost, per the EPA's guidance (Clar et al. 2004).

Step 4: Determine necessary media depth.

The depth will be designed for treatment of solids, metals, petroleum hydrocarbons (with the mulch), and sorbed phosphorous. Nitrogen and dissolved phosphorous will also be treated, but are not critical to design as they are not the priority pollutants in this case. For this reason there will not be a permanent saturated zone in these bioretention cells. Depth of roots must also be considered. Therefore, each cell will have 36 inches of filter media. Figure 30 show the media profile for Cell 2.

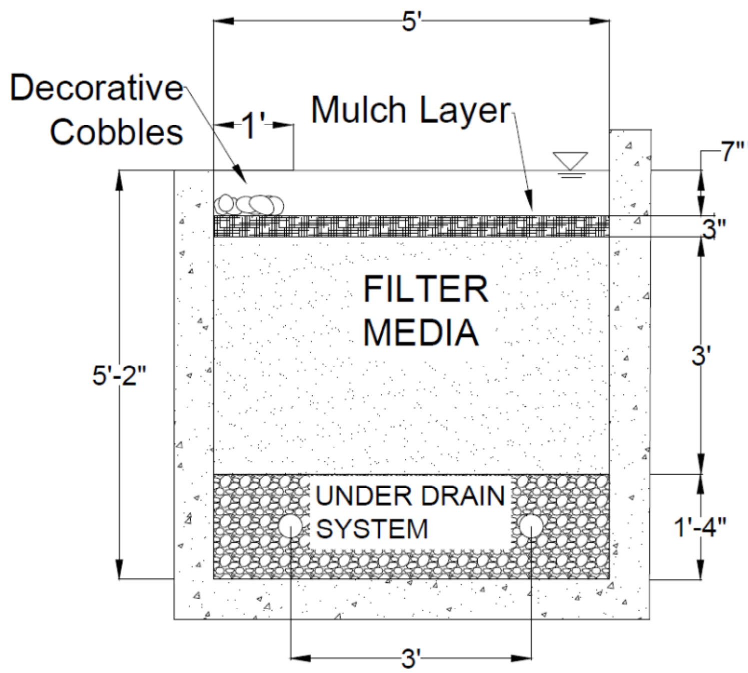


Figure 30: Cross-section A for bioretention example

Step 5: Calculate surface area.

Equation 5-14 will be used to calculate the area of Cell 1, and Table 31 shows the results for all watersheds:

$$A_{\text{Cell 1}} = \frac{WQV}{D_{\text{Max Pond}}} = \frac{74\text{ft}^3}{0.5\text{ ft}} = 148\text{ ft}^2$$

Table 31: Required bioretention area per sub-watershed

WS 1 & 2	WS 3 & 4	WS 5 & 6	WS 7 & 8	WS 9 & 10
$148ft^2$	$385ft^2$	$350ft^2$	$426ft^2$	$119ft^2$

Step 6: Select dimensions for bioretention area.

The bioretention area will be limited to 5 feet wide to accommodate pedestrian traffic on the sidewalk. Each cell will run parallel to and directly adjacent to the roadway. The required length for each cell is shown in Table 32. Figure 31 shows the plan view for Cell 2.

Table 32: Required lengths for bioretention cells

Cell 1 & 2			Cell 7 & 8		
Length (ft)	Width (ft)	Area (ft^2)	Length (ft)	Width (ft)	Area (ft^2)
30.00	5	150	86.00	5	430
Cell 3 & 4			Cell 9 & 10		
Length (ft)	Width (ft)	Area (ft^2)	Length (ft)	Width (ft)	Area (ft^2)
77.00	5	385	25.00	5	125
Cell 5 & 6					
Length (ft)	Width (ft)	Area (ft^2)			
70.00	5	350			

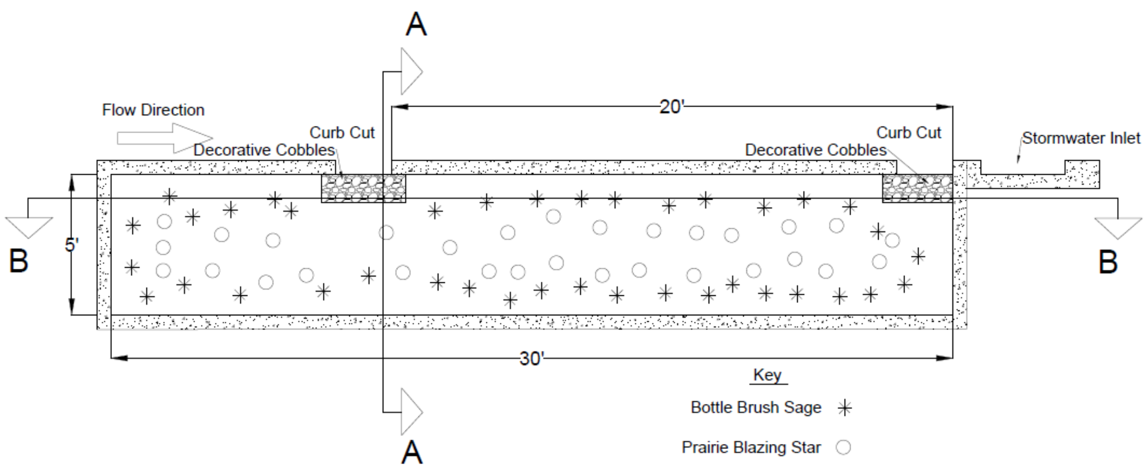


Figure 31: Plan view of Cell 2 for bioretention example

Step 7: Design inlet system and pretreatment.

Curb cuts will be used to divert runoff to the bioretention cells. These cuts will be placed 1 foot upstream of the stormwater inlets and then every 20 feet upstream from there, as shown in Figure 31 for Cell 2. This will allow for the majority of the runoff to be captured and distributed over the length of the cell. Upon entering the cells, the runoff will be passed over decorative cobbles which will act to slow the runoff and prevent erosion. Additional removal of solids will be achieved with a 3 inch thick layer of shredded hard wood mulch spread evenly over the cells.

Step 8: Design under-drain (If necessary).

Each cell will require the installation of an underdrain. It will be 2, 4-inch diameter slotted PVC pipes spaced 3 feet apart running longitudinally down the length of the cells. Two pipes are used to ensure functionality if one clogs. The pipes will be laid in a 16 inch deep bed of gravel, with 6 inches of gravel above the pipe and 6 inches below. The pipes will have a 1% slope towards the outlet. A geotextile to the specifications presented in Table 29 will overlay the gravel to prevent transport of the filter media into the gravel layer and outlet pipe. The pipe will discharge into the existing sewer system, as shown in Figure 32.

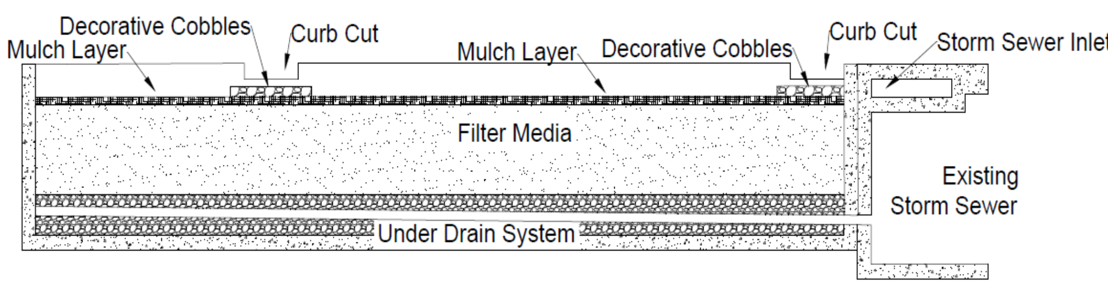


Figure 32: Cross-section B of Cell 2 for bioretention example

Step 9: Select and size overflow method.

This system will not incorporate an overflow system. The bottom of the curb cuts will be positioned at the same elevation as the design depth of the WQV. This orientation will allow for volumes greater than the WQV to either discharge from the curb cuts or flow by without entering the cell.

Step 10: Specify vegetation plan.

The vegetation for each cell will be a mix of local herbaceous grasses and flowers. Bottlebrush sedge will be coordinated with prairie blazing star in each cell. These species are both well-suited for saturated conditions. Drought conditions should also be factored into plant selection. Although these plants are not drought resistant, the local business owners who are doing the day-to-day maintenance of these cells will water them between rainfall events. Areas with less intensive maintenance opportunities should put a greater emphasis on drought resistance.

5.2.4 Basin Sand Filter

Design Process:

Step 1: Evaluate applicable location considering site constraints.

Step 2: Calculate water quality volume to be treated (WQV).

Step 3: Size sediment basin.

Step 4: Determine filter media characteristics.

Step 5: Select filter bed depth.

Step 6: Calculate filter surface area.

Step 7: Design sediment basin outlet riser.

Step 8: Specify filter inlet characteristics.

Step 9: Design under-drain.

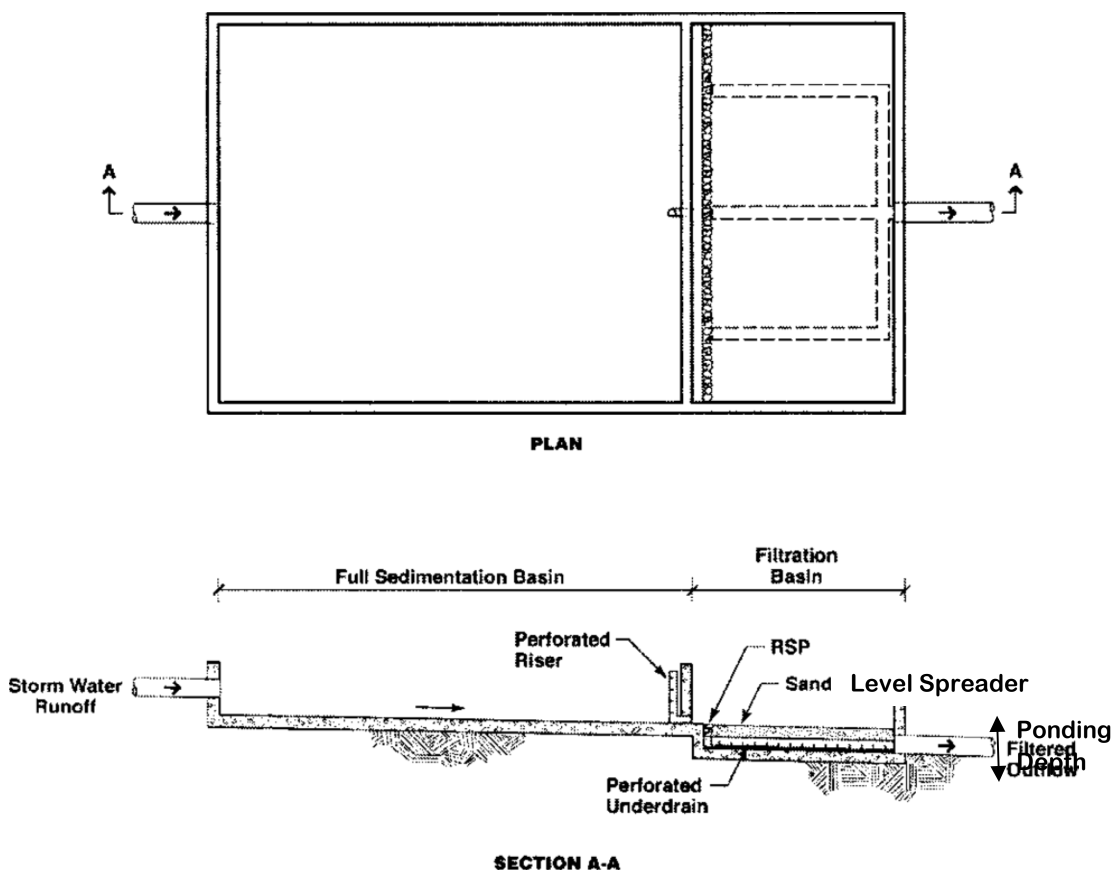


Figure 33: Sand filter design (Barrett 2003)

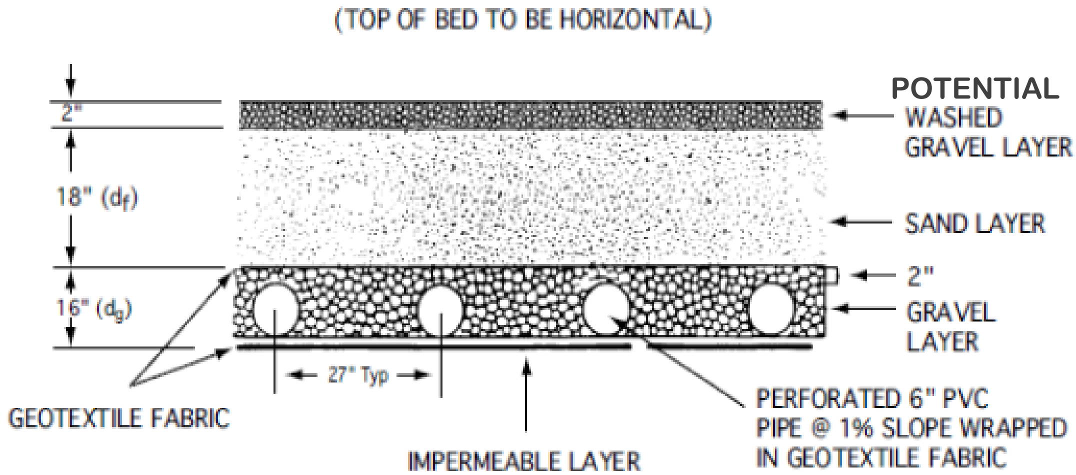


Figure 34: Filter bed cross section (NVPDC 1996)

Design Criteria

Table 33 contains the criteria to be considered while working through the design process.

Table 33: Design considerations for basin filters

Design Parameter	Minimum	Maximum
Sediment basin layout (L:W)	2: 1[3,5,7,8]	4: 1[8]
Depth of filter media	18 inches[1,2,3,4,5,6,7]	-
Infiltration rate of filter media	$3.5\ \frac{\text{in}}{\text{day}}$[2,4,5,6,7,8]	
Diameter of under drain pipes	6 inches[1,2,4,7]	
Slope of under drain pipes	1%[1,5,7,8]	
Slope of sedimentation basin	2%[1]	
Time for filter surface drawdown	40 hrs[2,6,7,8]	
Drawdown time for sediment basin	24 hrs[7]	
1) CalTrans (2010c) 2) Landphair et al. (2000) 3) KCDNRP (2009) 4) SEMCOG (2008)	5) MSSC (2005) 6) NCDENR (2007) 7) NVPDC (1996) 8) VCSQMP (2001)	

Step 1: Evaluate applicable location considering site constraints

Filtration systems are a viable option for ultra-urban situations due to their small footprint and layout flexibility. They can be located at stormwater inlets and discharge into the existing sewer system. Sand filters can also serve as end-of-pipe BMPs with the forebay acting as an energy dissipater.

Sand filters perform best when treating highly impervious watersheds (MSSC 2005). Impervious watersheds contribute less total suspended solids, thus limiting the amount of fines entering the system (CalTrans 2004). Treating impervious areas will extend the life of the filter as well as reduce maintenance costs.

Step 2: Calculate water quality volume to be treated (WQV)

The water quality volume (WQV) is the amount of runoff requiring treatment. The water quality volume is calculated by summing the volume which comes from newly constructed impervious areas and the volume of run-on from adjacent property which comingles with run off from the new development. The volume from impervious areas can be found with Equation 5-15:

$$\mathbf{WQV_{New\ Dev} = 0.5in * \frac{Area\ Treated\ (ft^2)}{12\frac{in}{ft}}} \quad (5\text{-}15)$$

The volume running off pervious areas is found with Equation 5-16:

$$\mathbf{WQV_{Run-On} = Q\ (in) * \frac{Area\ Treated\ (ft^2)}{12\frac{in}{ft}}} \quad (5\text{-}16)$$

The runoff depth (Q) can be found in Table 8.

Step 3: Size sediment basin

The sediment basin should be sized to retain the entire WQV. A riser pipe will discharge into the infiltration basin. The basin geometry should have at least a 2:1 length-to-width ratio (NVPDC & ESI 1996). This ratio will facilitate the settlement of particles within the basin. The inlet and riser pipe outlet should be on opposite ends of the basin to promote residence time and to decrease the amount of dead zones within the system. Runoff should enter the basin at 3 $\frac{ft}{s}$ (0.9 $\frac{m}{s}$) or less. An energy dissipation device, such as a rip-rap apron or basin, should be used for larger velocities (NVPDC & ESI 1996).

The minimum surface area of the sediment basin is calculated using Equation 5-17 (Camp Hazen Equation) (NCDENR 2007):

$$\mathbf{A_B = -\left(\frac{Q_o}{w}\right) * (\ln(1 - E))} \tag{5-17}$$

Where:

A_B: Surface area of sedimentation basin $(ft^2\ or\ m^2)$

Q_o: Outflow (cfs or cms)

E: Trap efficiency of the chamber (unitless) (E = 0.9) (KCDNRP 2009)

w: Critical settling velocity of particle $\left(\frac{ft}{s}\ or\ \frac{m}{s}\right)$

Settling velocity is a function of particle size, and therefore, percent imperviousness of the watershed. For watersheds with ≥ 75% impervious, w = 0.0033 $\frac{ft}{s}$ and for watersheds < 75%, w = 0.0004 $\frac{ft}{s}$ (KCDNRP 2009).

Sedimentation chambers should be at least 1.5 feet (0.46 m) wide (parallel to flow) (NCDENR 2007), with an L:W ratio between 4:1 and 2:1 (Table 33) (VCSCQMP 2001). Ponding depth in the sedimentation basin should be 2–6 ft (0.61–1.8 m) (CEI & NHDES 2008).

Step 4: Determine filter media characteristics

Filter media can be sand or a mixture of sand, mulch, clay, or wood fiber. Different mixes have varying hydraulic characteristics, pollutant removal capabilities, and costs. Costs are largely based upon the availability of the media in question. Regardless of the media mixture, an infiltration rate of 3.5 $\frac{in}{day}$ (8.9$\frac{cm}{day}$) must be maintained throughout the life of the system. If sand is the only media being used, it should be similar to ASTM C-33 Concrete Sand, as described in

Appendix B (NVPDC &ESI 1996). The King County Surface Water Design Manual (KCDNRP 2009) suggests use of sand meeting the specifications presented in Table 34, which is based on the weight of sand which will pass standard sieves. Each of these sand specifications is ideal due to the small portions of fines they contain. Fines should be avoided in the filter media to avoid premature media clogging.

Table 34: Sand Media Specifications

U.S. Sieve Size	Percent passing
U.S. No. 4	95 to 100 percent
U.S. No. 8	70 to 100 percent
U.S. No. 16	40 to 90 percent
U.S. No. 30	25 to 75 percent
U.S. No. 50	2 to 25 percent
U.S. No. 100	Less than 4 percent
U.S. No. 200	Less than 2 percent

(KCDNRP 2009)

Step 5: Select filter bed depth

As shown in Table 33, the filter depth must be at least 18 inches (45.7 cm) deep (KCDNRP 2009). The minimum is acceptable but may require more labor intensive maintenance. A deeper filter bed will allow for the top 2 inches (5 cm) where the majority of clogging occurs, (CalTrans 2004 & Hatt et al. 2010), to be removed without the immediate addition of more media.

Hydraulic requirements may limit the depth of media. The elevation change between the inlet and outlet must exceed the total depth of the water over the filter, the filter media, and the underdrain system (CalTrans 2004). Deeper media may not allow gravity flow through the system and into existing sewer systems. Pumping can be employed but increases expenses and potential problems.

Step 6: Calculate filter surface area

The surface area of the filter is determined by Equation 5-18, the Austin Sand Filter Equation (NCDENR 2007):

$$\mathbf{A_f} = \frac{\mathbf{WQV*d_f}}{\mathbf{K(h+d_f)t}} \tag{5-18}$$

Where:

A_f = Surface area of sand bed (ft^2 or m^2)

WQV: Water Quality Volume (ft^3 or m^3)

d_f= sand bed depth (ft or m)

K = Hydraulic conductivity for sand filter (0.29 $\frac{ft}{hr}$ or 0.088 $\frac{m}{hr}$)

(NCDENR 2007)

h = average depth of water above surface of sand media (ft or m); half of maximum ponding depth

t = time required for runoff volume to pass through filter media (hours)

The average filter head (h) is half of the maximum filter head. Ponding above the filter should be limited to 6 inches (15.2 cm) (SEMCOG 2008) to ensure drainage in 40 hrs.

<u>Step 7</u>: Design sediment basin outlet riser

The riser between the sediment basin and the filter bed should be designed to drawdown the WQV within 24 hours (NVPDC & ESI 1996). There should be a grate around the riser which will act as a trash rack preventing debris from clogging the orifices. Figure 35 shows a profile view of a riser pipe.

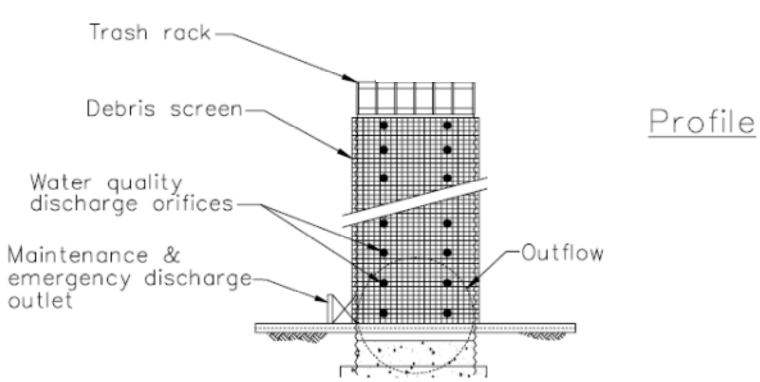


Figure 35: Profile of riser pipe (CASQA 2003)

Riser pipe design is done using Equation 5-19 (CASQA 2003):

$$\mathbf{a_t} = \frac{\mathbf{2A(h_{max})}}{\mathbf{3600CT(2g[h_{max} - h_{centroid\ of\ orifices}])^{0.5}}} \qquad (5\text{-}19)$$

Where:

a_t: Total area of orifices (ft^2)

A: Surface area of sedimentation basin (ft^2)

h_{max}: Maximum height from lowest orifice to highest water level (ft)

$h_{centroid\ of\ orifices}$: Height from lowest orifice to centroid of orifices (ft)

C: Orifice coefficient (0.66 for pipe material equal to or less than the diameter of the orifice or 0.8 for pipe material thicker than the diameter of the orifice) (CASQA 2003)

T: Drawdown time of full basin (hrs)

g: Gravity (32.2 $\frac{ft}{s^2}$)

In order to maintain drainage if an area of the riser is clogged, orifices should be placed on the riser in 2 even rows. These rows should be 120 degrees apart horizontally. Vertical spacing between holes should be three times the diameter of the hole (CASQA 2003). This spacing will protect against clogging of multiple holes simultaneously.

Step 8: Specify filter bed inlet characteristics

Discharge from the riser pipe must be evenly and safely distributed over the area of the filter. Concentrated flows could create scour or short circuiting of the filtration process. For this purpose energy dissipaters or flow spreaders are required at the filter bed inlet.

The King County Surface Water Design Manual suggests criteria for an effective flow spreader (KCDNRP 2009):

"a) If the sand filter is curved or an irregular shape, a flow spreader shall be provided for a minimum of 20 percent of the filter perimeter.

b) If the length-to-width ratio of the filter is 2:1 or greater, a flow spreader must be located on the longer side and for a minimum length of 20 percent of the facility perimeter.

c) In other situations, use good engineering judgment in positioning the spreader."

Figure 36 demonstrates placement of flow spreaders for irregular shapes as discussed above.

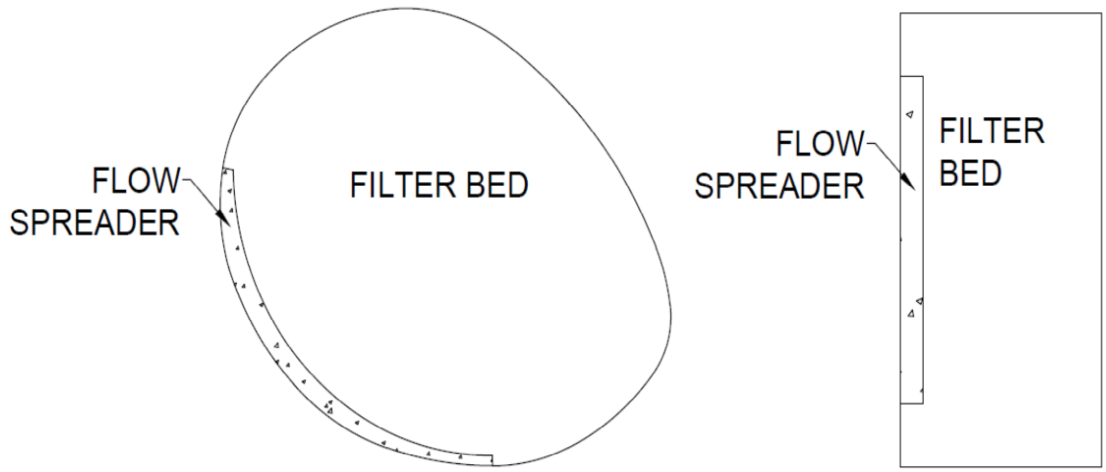


Figure 36: Flow spreader placement for irregular shaped filters

The King County Surface Water Design Manual (KCDNRP 2009) also requires 1 foot (0.3 m) of erosion protection between the flow spreader and the filter bed. Use of weighted-down geotextile or coarse aggregates are acceptable erosion protection practices. Figure 37 shows a profile of the transition between the sediment basin and the filter bed. Level spreaders constructed from concrete must utilize weep holes so the entire WQV can drain into the filter bed.

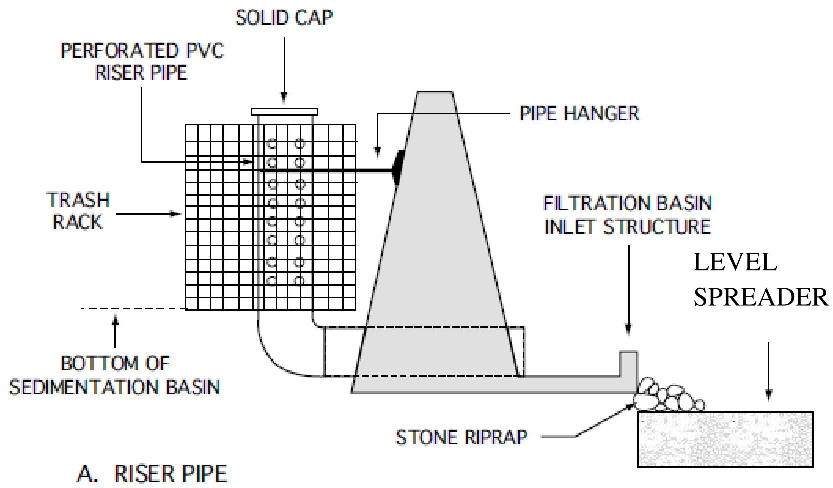


Figure 37: Transition from sedimentation basin to filter bed (NVPDC 1996)

Step 9: Design under-drain

Once the runoff has passed through the filter media it will be collected and discharged by an under-drain system. This system will be composed of 6 inch diameter slotted PVC pipes wrapped in geotextile and set in a 16 inch thick gravel bed at a 1% down-slope to the outlet (NVPDC & ESI 1996). The gravel will over top the pipes by at least 2 inches and conform to the AASHO #3 standard as described in Appendix B (VCSQMP 2001, NVPDC & ESI 1996). The pipes will be no more than 10 feet apart (NVPDC & ESI 1996). There must also be a nonwoven geotextile layer between the filter and under-drain media. The geotextile must meet the specification presented in Table 35.

Table 35: Geotextile specifications (VCSQMP 2001)

Geotextile property	Specification	Test
Grab strength	90 lbs	ASTM D4632
Elongation at peak load	50%	ASTM D4632
Puncture strength	24 lbs	ASTM D3787
Permitivity	0.7 sec^{-1}	ASTM D4491
Burst strength	180 psi	ASTM D3786
Toughness	5500 lbs	% Elongation * Grab strength
Ultraviolet resistance (% strength after 500 Weatherometer hours)	70%	ASTM D4355

<u>Design Example</u>:

A newly constructed section of urban highway requires treatment of runoff from 0.8 acres of impervious surface (CN 98) and 0.2 acres of adjacent grass (CN 83). The area requires the use of a BMP with a relatively small footprint, so a sand filter is selected. Figure 38 shows the plan view of the site.

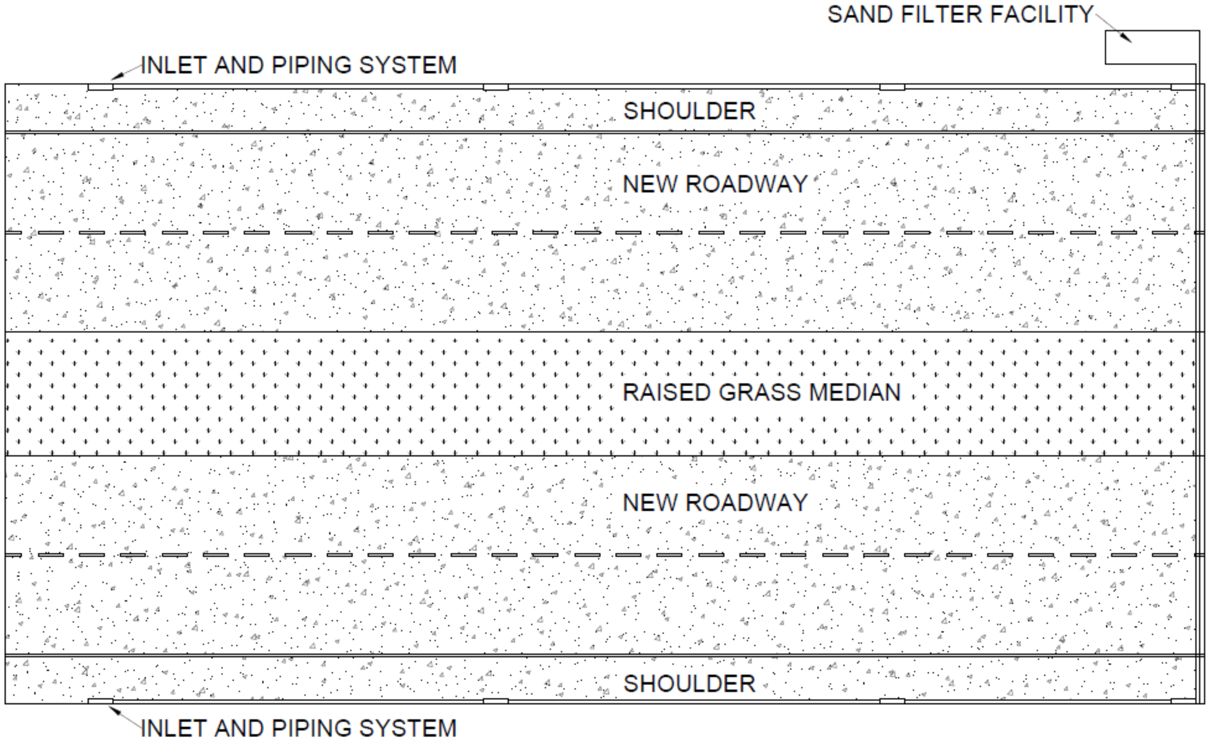


Figure 38: Example site plan view for basin sand filter

<u>Step 1</u>: Evaluate applicable location considering site constraints

A sand filter was selected for this location because the watershed is highly impervious, and the available land in the right-of-way is very limited..

<u>Step 2</u>: Calculate water quality volume to be treated (V_{WQ})

The water quality volume can be found by summing the volumes of runoff from the pervious and impervious surfaces. This can be done through summing Equations 5-15 and 5-16:

$$WQV_{New\,Dev} = 0.5in * \frac{Area\,Treated\,(ft^2)}{12\,\frac{in}{ft}}$$

$$WQV_{Run-On} = Q * \frac{Area\,Treated\,(ft^2)}{12\,\frac{in}{ft}}$$

Table 8 shows a Q of 0.049 inches for CN = 83.

$$\mathrm{WQV_T} = 0.5\ \mathrm{in}* \frac{0.8\ \mathrm{Ac}*43560\,\frac{\mathrm{ft}^2}{\mathrm{Ac}}}{12\,\frac{\mathrm{in}}{\mathrm{ft}}} + 0.049\mathrm{in}\ * \frac{0.2\mathrm{Ac}*43560\,\frac{\mathrm{ft}^2}{\mathrm{Ac}}}{12\,\frac{\mathrm{in}}{\mathrm{ft}}} = 1488\mathrm{ft}^3$$

<u>Step 3</u>: Size sediment basin

The sediment basin needs to be sized to store the WQV and to drain within 24 hours. Equation 5-17 is used to find the minimum required area of the sediment basin:

$$A_B = -\left(\frac{Q_o}{w}\right) * (\ln\,(1 - E))$$

$$A_B = -\left(\frac{\frac{1488ft^3}{24\,hrs} * \frac{1\,hr}{3600\,s}}{0.0033\,\frac{ft}{s}}\right) * (\ln(1 - 0.9)) = 12\,ft^2$$

Settling velocity (w) is $0.0033\ \frac{ft}{s}$ because the contributing watershed had greater than 75% impervious area. A $12\ ft^2$ sedimentation basin would require a depth of 124 ft, which is unacceptable. With limiting depths of 2-6 ft, a 33x11 ft sedimentation chamber with a depth of 4 ft will be used. This configuration allows for a 3:1 ration which provides an adequate flow path while also storing the WQV at a depth of 4 ft, a 6 inch free board will be included. Figure 39 shows the orientation of the sedimentation basin.

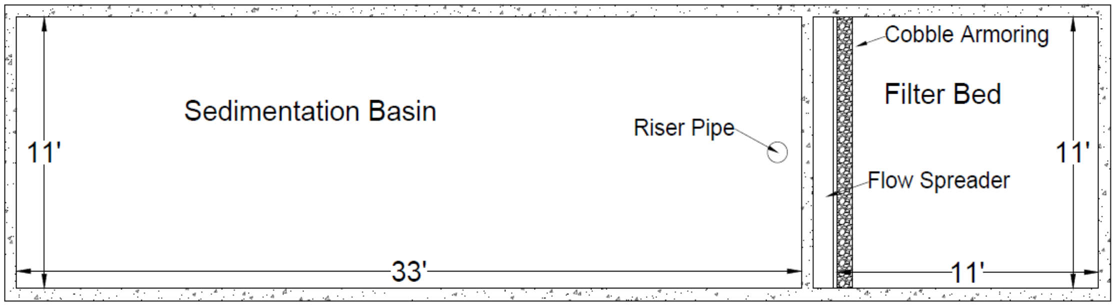


Figure 39**:** Plan view of example sand filter

Step 4: Determine filter media characteristics

Because there are no special requirements for treatment, the filter will be composed entirely of sand which adheres to the ASTM C-33 Concrete Sand standard. Fines should be avoided as they can clog the media.

Step 5: Select filter bed depth

This filter bed will be 24 inches deep initially. This will allow for maintenance crews to remove the top 3 inches 2 times before requiring additional sand be brought in to replenish the system. Figure 40 shows the cross-section of the system.

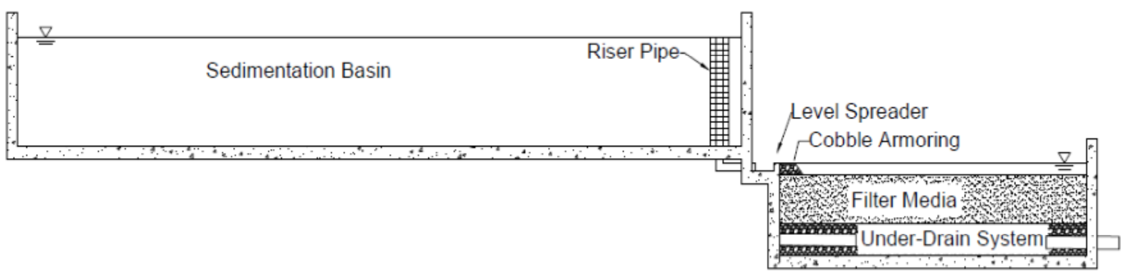


Figure 40: Profile view of example sand filter

Step 6: Calculate filter surface area

Equation 5-18 is used to determine the required surface area of the filter bed.

$$A_f = \frac{WQV * d_f}{k\left(h + d_f\right)t}$$

$$A_f = \frac{(1488\,ft^3) * 2ft}{0.29\,\frac{ft}{hr}(0.25\,ft + 2\,ft) * 40\,hrs} = 114\,ft^2$$

The dimensions of the filter bed will be 11 x 11 ft, which provides 121 ft^2 of surface area. Figure 39 shows the plan view of the system.

Step 7: Design sediment basin outlet riser

The required area of holes in the riser pipe can be found with Equation 5-19:

$$a_t = \frac{2A(h_{max})}{3600CT(2g[h_{max}\text{-}h_{centroid\ orifices}])^{0.5}} = \frac{2(363)(4)}{3600*0.66*24(2(32.2)[4\text{-}2])^{0.5}} =$$

$$0.0045\ ft^2 = 0.65\ in^2$$

Using the geometry of the sedimentation basin to determine a maximum depth of 4 feet and using a riser height of 4 feet, an area requirement of 0.65 in^2 is found.

An orifice diameter of 0.25 inches was selected for this riser. This diameter requires 13 orifices to account for the total required orifice area. These orifices will be positioned in two parallel columns 120 degrees apart from each. They will be vertically spaced 7 inches apart beginning 6 inches above the bottom of the sedimentation basin. Figure 41 shows the orifice spacing.

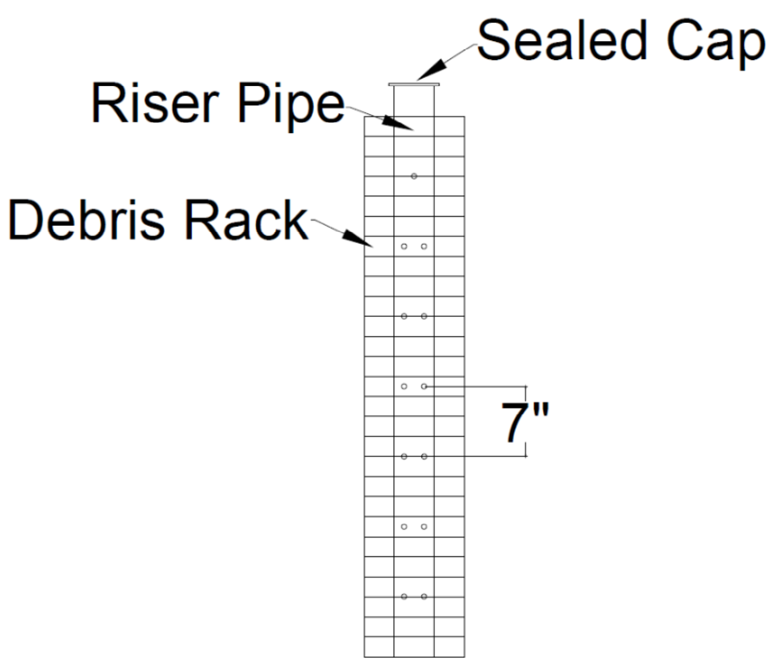


Figure 41: Detail of example riser pipe

Step 8: Specify filter inlet characteristics

The level spreader in the filter bed will be a 4 inch deep, 1 foot wide reinforced concrete trench. The trench will run against the wall the filter vault shares with the sedimentation basin. Water will discharge over a one foot wide strip of coarse gravel as it enters the sand filter.

Step 9: Design under-drain

The under-drain will consist of a 16 inch deep coarse aggregate layer which has 2, 6 inch diameter perforated PVC pipes which run the width of the chamber and slope down to the outlet at 1%. The pipes will be 3 ft from the outside walls. The uphill end of the PVC will be 5 inches beneath the top of the gravel layer. That depth will increase as the pipes slope downward. The two pipes will feed into a 6 inch collector pipe at the downhill edge of the filter chamber which will be discharged through a single outlet. There will be a geotextile between the sand layer and the gravel layer, as well as around the pipes, which conforms to the requirements set out in Table 35. Figure 42 show the layout of the under-drain system.

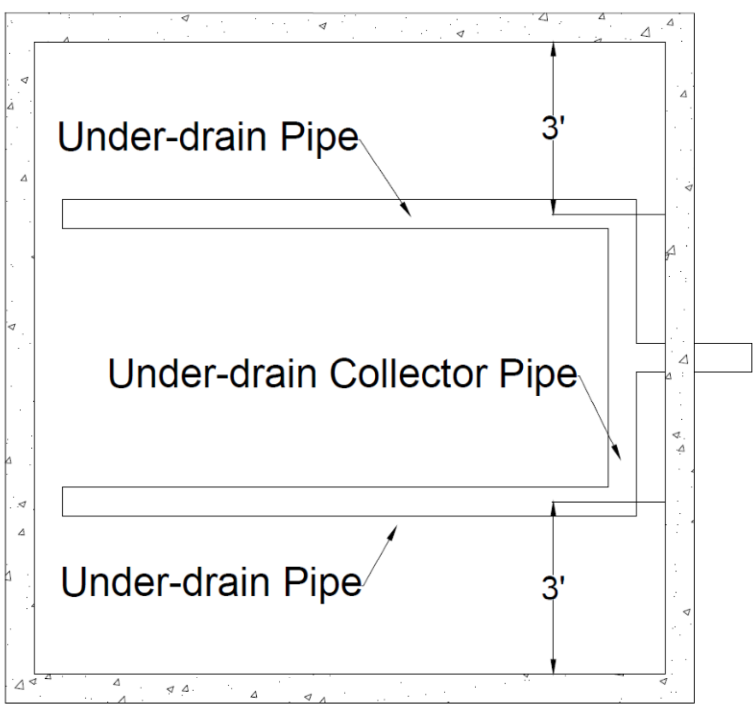


Figure 42: Underdrain layout for example sand filter

5.2.5 Horizontal Filter Trench

<u>Design Process</u>:

Step 1: Evaluate applicable location considering site constraints.

Step 2: Calculate water quality volume to be treated (WQV).

Step 3: Select filter media specifications.

Step 4: Select armoring specifications.

Step 5: Calculate trench dimensions.

Step 6: Verify armoring size by checking scour potential.

Step 7: Select pretreatment.

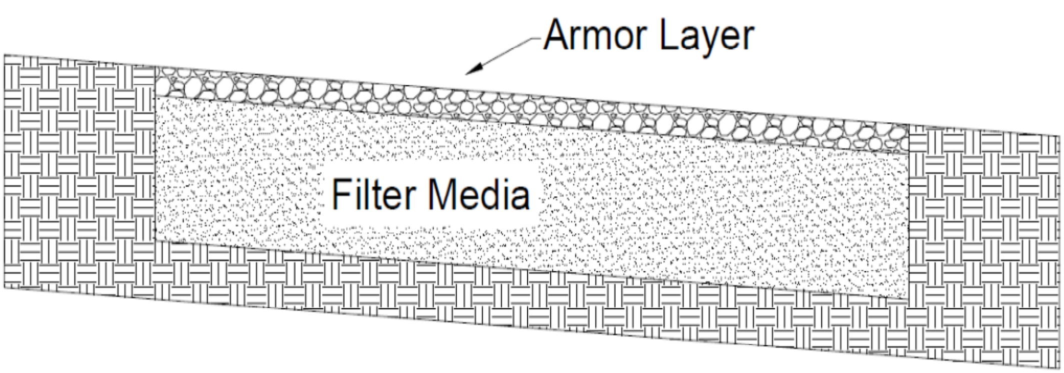


Figure 43: Profile of filter trench length

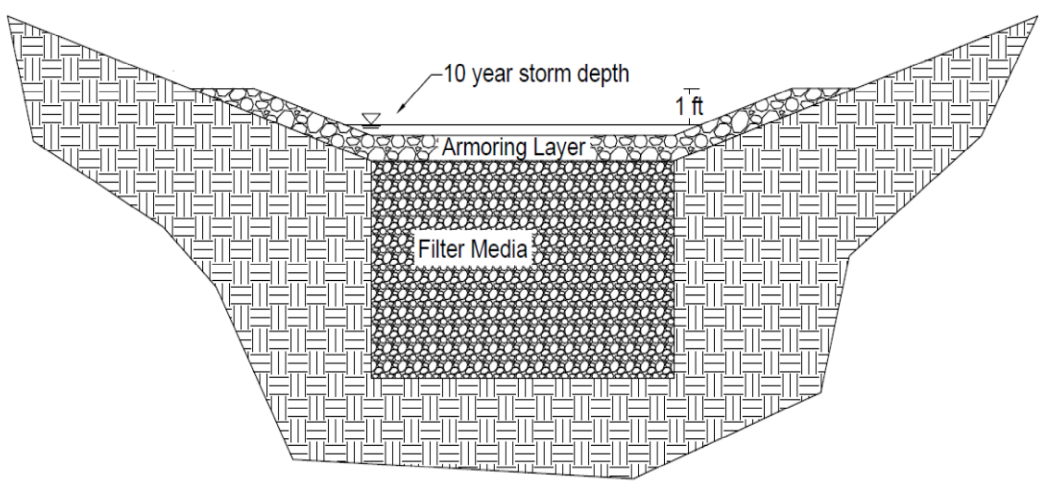


Figure 44: Profile of filter trench width

Step 1: Evaluate applicable location considering site constraints.

Horizontal filter trenches are best suited for linear applications. Prime siting areas are in ditches or swales along roadways and as end of pipe treatment systems. There is high retrofit potential for horizontal filter trenches in existing roadside drainage ditches. Although some existing infrastructure adds water quality benefits already, a horizontal filter trench can be placed in the bottom of drainage ditches if existing vegetation is insufficient to treat the runoff, or if expected flows will damage vegetated systems. Filter trenches may not be cost effective if the existing ditch is wide with gentle side sloped due to armoring requirements. However, these types of channels may already act as vegetated filter strip/vegetated swale systems, or they could with minor modifications. The trench can be placed the entire length of the treated roadway or downstream from the treated area, depending on site constraints such as availability of land in the right-of-way and slope adjacent to the roadway. Roadside vegetation on the slope leading to the bottom of the ditch may also act as a pretreatment for solids removal.

Horizontal filters are designed for use on sloped surfaces. If there is no slope, the filter will not be able to discharge and will act as an infiltration trench. Horizontal filter trenches should not be used as infiltration trenches in the following situations:

However, the following situations will require the use of an under-drain:

- Inadequately drained subgrades (hydraulic conductivity $\leq 0.50\ ^{in}/_{hr}$ ($1.3\ ^{cm}/_{hr}$)),
- Infiltration is harmful to surrounding structures (e.g., possible damage to foundations),
- The seasonal high groundwater table is within 3 ft (0.9 m) of the bottom of the bioretention cell (MSSC 2005),
- Treating a pollutant hot spot (e.g., gas station) where groundwater contamination is probable.

Step 2: Calculate water quality volume to be treated (WQV)

The water quality volume (WQV) is the amount of runoff requiring treatment. The water quality volume is calculated by summing the volume which comes from newly constructed impervious areas and the volume of run-on from adjacent property which comingles with run-off from the new development. The WQV can be found by summing Equation 5-20, which calculates the volume coming off newly developed areas, and Equation 5-21, which calculates the volume of run-on:

$$\mathbf{WQV_{New\ Dev} = 0.5in * \frac{Area\ Treated\ (ft^2)}{12\frac{in}{ft}}} \quad (5\text{-}20)$$

$$\mathbf{WQV_{Run-On} = Q\ (in) * \frac{Contributing\ Area\ (ft^2)}{12\frac{in}{ft}}} \quad (5\text{-}21)$$

Q is the runoff depth found in Table 8.

Step 3: Select filter media specifications

The filter media should be $^3/_8$–$^3/_4$ inch (0.95–1.9 cm) clean washed media. There should be very few fines to avoid clogging and to prolong the life of the BMP. Potential media constituents include pea gravel, shredded tires, or a mixture of the two. A porosity of 0.3 will be used in calculations for the filter media.

Shredded tires, if being considered, should conform to the same sizing criteria as pea gravel. If shredded tires are the only media, filter depth should not be greater than 3.3 ft (1 m) (Humphrey 1999) or self-heating may be a problem. Guidelines to avoid self-heating were established by an Ad Hoc Civil Engineering Committee of government and industry entities (AHCEC 1997) and published as ASTM D6270-98 (ASTM 1998). These guidelines for avoiding self-heating of scrap tires for depths of 3.3–10 ft (1–3 m) follow:

• Tire shreds shall be free of contaminants such as oil, grease, gasoline, diesel fuel, etc., that could create a fire hazard

• In no case shall the tire shreds contain the remains of tires that have been subjected to a fire

• Tire shreds shall have a maximum of 25% (by weight) passing 1½-in. sieve

• Tire shreds shall have a maximum of 1% (by weight) passing no. 4 (4.75-mm) sieve

• Tire shreds shall be free from fragments of wood, wood chips, and other fibrous organic matter

• Tire shreds shall have less than 1% (by weight) of metal fragments that are not at least partially encased in rubber

• Metal fragments that are partially encased in rubber shall protrude no more than 1 in. from the cut edge of the tire shred on 75% of the pieces and no more than 2 in. on 100% of the pieces

• Infiltration of water into the tire shred fill shall be minimized (see below)

• Infiltration of air into the tire shred fill shall be minimized

• No direct contact between tire shreds and soil containing organic matter, such as topsoil

• Tire shreds should be separated from the surround soil using a geotextile

• Use of drainage features located at the bottom of the fill that could provide free access to air should be avoided

For the purposes of the horizontal filter trench, water and air will need to infiltrate into the tire media. Self-heating can be avoided by mixing the tire with granular media (Edil et al. 2004) or by keeping the depth below 3.3 ft (1 m). Shredded tires have been shown to avoid self-heating when used in depths less than 3.3 ft (1 m) in several landfill drainage applications (Edil et al. 2004; Humphrey 1999).

Step 4: Select armoring specifications

Armoring for the trench should adhere to the Federal Highway Administrations definition of cobbles by having a diameter of 2.5–5 in (6.4 –13 cm) (Kilgore and Cotton 2005). They are generally alluvial, uniformly graded, and rounded. Armoring depth should be at least 1.75 times the diameter of stone for which 50%, by weight, of gradation is finer (d_{50}) (OES & WWE 2000).

Armoring must be placed over the filter trench and up the side slopes. The armoring should reach 1 ft (0.3 m) above the water surface for the 10 year, scour check, storm (OES & WWE 2000). A geotextile is required between the armoring and both the trench and the adjacent soil. The geotextile facilitates maintenance and prevents mobilization of the underlying media into the cobbles.

Step 5: Calculate trench dimensions

The trench must be sized so the WQV can be stored in its pore space. The armoring will store and slow runoff but will not be considered to add directly to the treatment, so pore space in the armoring will not be counted towards the WQV storage.

The required trench size to accommodate the WQV is found with Equation 5-22:

$$\frac{WQV}{p} = L * W * D \tag{5-22}$$

Where:

L: Trench length (ft or m)

W: Trench width (ft or m)

D: Media depth (ft or m)

p: Media porosity

The available width and length of the trench will be site-specific based on the geometry of the existing drainage ditches, available right-of-way, and existing grade. The depth of media should not be less than 1 foot (0.3 m). Trenches should not be deeper than 5 ft (1.5 m) due to the added costs of a protective system required at that depth (NIOSH 2011). The bottom of the trench should not be within 2 ft (0.61 m) of the seasonal high groundwater table.

Step 6: Verify armoring size by checking scour potential

Armoring must be able to withstand scouring effects of the peak flows. Peak scour flow rates are determined by using the 10 year design storm with a type II NRCS 24-hour distribution and Equation 5-23 (NRCS 1986).

$$\mathbf{q_p = q_u A_m Q F_p} \tag{5-23}$$

Where:

q_p: Peak discharge (cfs)

q_u: Unit peak discharge $\left(\frac{cfs}{mi^2 * in}\right)$ (Figure 1or Table 7)

A_m: Drainage area (mi^2)

Q: Runoff corresponding to 24-hr rainfall (in) (Table 9)

F_p: Pond or swamp adjustment factor (1.0 for Nebraska)

When considering a watershed with both impervious and pervious ground cover, the area can either be considered completely impervious, or a weighted flow may be calculated as described in the Hydrology Section of this work. Assuming total imperviousness would result in larger than actual flows and, therefore, oversized BMPs. For this reason the weighted flow method is recommended.

Once the peak flow is found, the roadside ditch geometry (channel's bottom width, side slopes, and longitudinal slope) must be determined. Side slopes should be no greater than 3:1 (horizontal:vertical) to avoid slope damage from channelization and to facilitate mowing.

Once the shape of the swale is determined, Equation 5-24 (Manning's Equation) can be applied to determine flow depth (NRCS 1986).

$$\mathbf{Q_{10-yr} = \frac{k}{n} A R^{2/3} S^{1/2}} \tag{5-24}$$

Where:

Q_{10-yr}: Flow from10-year storm (cfs or cms)

S: Slope in direction of flow $\left(\frac{\text{ft}}{\text{ft}} \text{ or } \frac{\text{m}}{\text{m}}\right)$

R: Hydraulic Radius $\left(R = \frac{A}{P_w}\right)$

A: Cross sectional area of flow (ft^2 or m^2)

P_w: Wetted Perimeter (ft or m)

n: Manning's coefficient

k: constant (1 for Metric Units; 1.486 for English Units)

The equations for the elements of trapezoidal cross-sections can be found in

Table **36**

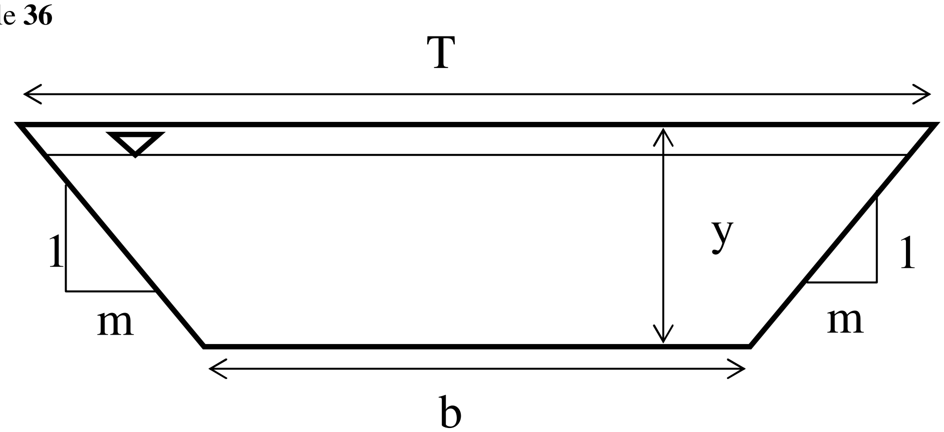


Figure 45: Reference shape for

Table 36

Table 36: Geo 0)

Area of flow (A) (ft^2 or m^2)	$(b+my)y$
Wetted perimeter (P_w) (ft or m)	$b+2y\sqrt{1+m^2}$
Hydraulic radius (R) (ft or m)	$\frac{(b+my)y}{b+2y\sqrt{1+m^2}}$

Inserting these geometric elements into the Manning's equation results in Equation 5-25, which is then used to solve for the depth of flow (y) by trial and error.

$$Q_p = \left(\frac{k}{n}\right) * (b+my)y * \left[\frac{(b+my)y}{b+2y\sqrt{1+m^2}}\right]^{2/3} * S^{1/2} \quad (5\text{-}25)$$

Manning's coefficient (n) can be calculated for rock-lined channels using Equation 5-26 (OES & WWE 2000).

$$\mathbf{n = 0.0395(d_{50})^{1/6}} \tag{5-26}$$

Where:

n: Manning's coefficient

d_{50}: Diameter of stone for which 50%, by weight, of gradation is finer (ft)

The velocity of the flow through the BMP can be determined with Equation 5-27 using the peak flow rate and area of flow. The cross-sectional area of flow can be found using Table 36.

$$\mathbf{V = \frac{Q_p}{A}} \tag{5-27}$$

The scour velocity found with Equation 5-27 must be less than or equal to 7 $\frac{ft}{s}$ (2.1 $\frac{m}{s}$) (Caltrans 2003). If the velocity found with Equation 5-27 is greater than 7 $\frac{ft}{s}$ (2.1 $\frac{m}{s}$) corrective action must be taken. Corrective action can consist of resizing the channel, selecting larger cobbles, or incorporating check dams.

Check dams for horizontal filter strips should not be earthen. Earthen check dams could leach fines which would contribute to clogging of the filter media. Rip-rap check dams are best suited for use with horizontal filters. The large void spaces associated with rip-rap check dams are not a problem in this situation as temporary ponding is not essential to the functionality of the trench. The check dams simply act to slow the flows, thereby preventing scour. The Vegetated Swale Design Guide section of this work describes sizing and spacing requirements for check dams.

Step 7: Select pretreatment

Pretreatment for horizontal filter trenches should be designed to remove solids and, if receiving concentrated flows, act as an energy dissipater. Pretreatment can extend the life of the trench dramatically by preventing clogging and scour.

When retrofitting an existing ditch, vegetation on the side slopes of the ditch can serve as vegetated filter strips. This pretreatment will remove solids but may not adequately attenuate velocities. However, runoff directly from the roadway will likely be in the form of sheet flow and will not require pretreatment for velocity. Vegetated filter strip design considerations can be found in the Vegetated Filter Strip Design Guide section of this work. If the filter trench does not run the entire length of the roadway it is treating, the ditch up stream of the filter trench may also act as pretreatment. The drainage ditch should be designed to the specifications in the Vegetated Swale Design Guide section of this work. If existing vegetation is not dense enough it may require refurbishing.

Shallow forebays at the initial point of the channel can be employed as treatment for solids and as energy dissipaters. Rip-rap forebays are well suited as pretreatment for horizontal filters. The volume of the forebay should be 0.05 inches (0.13 cm) multiplied by the impervious acres of the drainage area (Clar et al. 2004). Rip-rap is suggested as lining for the forebay because it will drain readily and will resist being washed away during times of high flow. Figure 46 shows a properly designed forebay.

Figure 46: Rip-rap forebay (NCDENR 2007)

Design Example

A 500 ft section of a 6-lane divided highway is being redeveloped. Figure 47 shows the layout of the 6 lane divided highway. Each direction features 3, 12 ft wide lanes with 6 ft shoulder on each side. The watershed will need to be broken into 3 subwatersheds to accommodate drainage from each side of the highway. There is also a 25 ft wide median and two 25 ft wide drainage ditches which run the length of the roadway. Horizontal filter trenches will be placed in the bottom of each ditch to treat the runoff, with a third filter trench in the median of roughly double the size. The median and ditches have a 3:1 side slopes with a 7 ft bottom width. The longitudinal slope is 5%. The vegetated areas have a CN of 80. There is no run-on from neighboring properties.

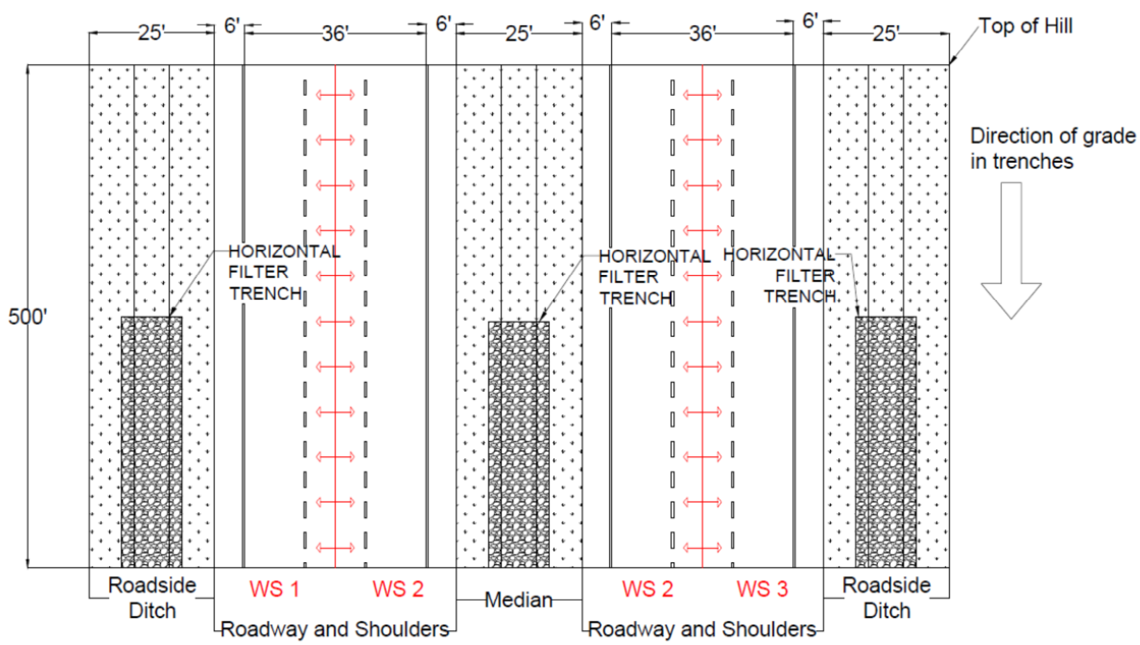


Figure 47: Site plan for horizontal filter example

Step 1: Evaluate applicable location considering site constraints

This is an ideal site for horizontal filters due to the existing median and drainage ditches. They are well suited to accommodate the filter trenches.

Step 2: Calculate water quality volume to be treated (WQV)

The WQV for the contributing area for each subwatershed must be calculated. Example calculations for WS 1 will be performed. Equation 5-20 will be used to find the runoff from the newly developed roadway for WS 1:

$$WQV_{New\ Dev\ WS1} = 0.5in * \frac{(18\ ft + 6\ ft) * 500\ ft}{12\frac{in}{ft}} = 500ft^3$$

The volume of run-on for WS 1 (i.e., the runoff from the grassy areas in WS 1) is found with Equation 5-21, Q for a CN of 80 is found to be 0.023 in from Table 8:

$$WQV_{Run\text{-}On} = 0.023\ in * \frac{25ft * 500\ ft}{12\frac{in}{ft}} = 24ft^3$$

The entire ditch and median area are considered for run-on because it is unknown at this stage what the dimensions of the horizontal filter trench will be.

The total WQV is then the sum of the runoff from the new development and the run-on volume:

$$500\ ft^3 + 24\ ft^3 = 524\ ft^3$$

Table 37 shows the calculated WQVs for each sub-basin

Table 37: Sub-basin WQVs

Sub Basin	WQV (ft^3)
WS 1	524
WS 2	1024
WS 3	524

Step 3: Select filter media specifications

The filter media will be clean washed pea gravel ranging in size from $^3/_8$–$^3/_4$ inches.

Step 4: Select armoring specifications

Cobbles with an average diameter by weight (d_{50}) of 3 inches will be initially selected for design, because it is readily available from a local quarry. If this selection proves to be insufficient at preventing erosion, the design process will revert to this step and select a larger cobble size which prevents scour.

Step 5: Calculate trench dimensions

The width of each trench will coincide with the 7 ft bottom width of the drainage ditches. The length and depth of the trench required for the filter media are dependent variables when considering the WQV. The required length and depth for WS 1 was calculated by using Equation 5-22. For example, the WQV for WS 1 is 524ft^{3}which requires a trench volume of 1747 ft^3 assuming a porosity of 0.3.

$$\frac{WQV_{WS1}}{p} = L * W * D = \frac{524 ft^3}{0.3} = 1747\ ft^3$$

Several length and depth relationships were checked, the dimensions decided upon are shown in Table 38.

Table 38: Dimensions of filter media in trench

Sub Basin	Width (ft)	Depth (ft)	Length (ft)	Volume
WS 1	7	1	250	1750
WS 2	7	2	245	3430
WS 3	7	1	250	1750

A constant length was selected which required the center sub basin (WS 2) to have a 2 ft depth. Had a constant 1 ft depth been used the WS 2 trench would have been 490 ft. A shorter trench was selected to be more cost effective for WS 2. Figure 48 shows the longitudinal profile of the horizontal filter trench for WS 2, and Figure 49 shows the width cross-section for WS 2.

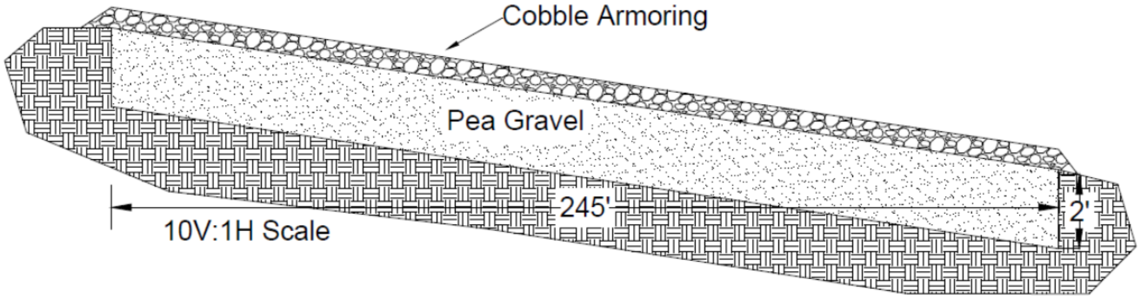


Figure 48: Longitudinal cross-section for example horizontal filter trench in WS 2

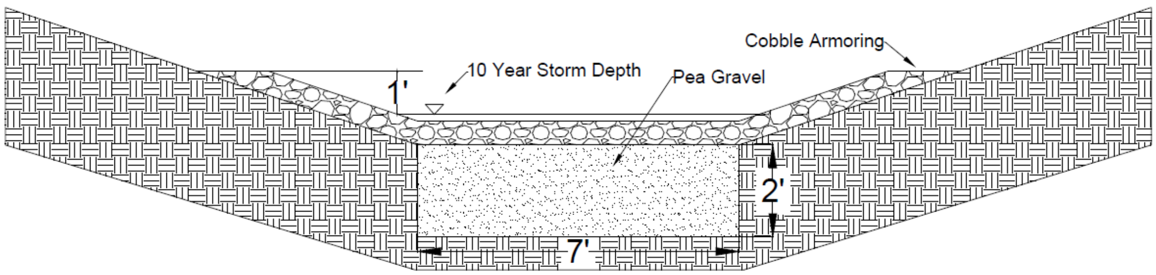


Figure 49: Width cross-section for example horizontal filter trench in WS 2

Step 6: Verify armoring size by checking scour potential

Sample calculations will be performed for WS 1. First the peak flows must be calculated for the new development and run-on using Equation 5-23. Then the flows from the two contributing areas are summed to find the total:

For new development:

$$q_p = q_u A_m Q F_p = 1100 \frac{cfs}{mi^2 in} * 0.00043mi^2 * 4.76in * 1 = 2.25cfs$$

For run-on:

$$q_p = q_u A_m Q F_p = 1000 \frac{cfs}{mi^2 in} * 0.00045mi^2 * 2.89in * 1 = 1.3cfs$$

Total peak flow:

$$2.25cfs + 1.3cfs = 3.55\ cfs$$

The peak flow (q_p) is then used to determine the flow depth and area, which leads to the scour velocity. Unit peak discharge (q_u) was found in Figure 1, and the runoff depth (Q) was found in Table 9. The swamp adjustment factor (F_s) for Nebraska is 1.

Equation 5-24 (Manning's Equation) is then used to find the depth of flow, which will be used in flow velocity calculations. The geometric elements of a trapezoid, from Table 36, are inserted into Equation 5-24 transforming it into Equation 5-25:

$$Q_p = \left(\frac{k}{n}\right) * (B+my)y * \left[\frac{(B+my)y}{B+2y\sqrt{1+m^2}}\right]^{2/3} * S^{1/2}$$

Equation 5-26 is used to find Manning's coefficient (n) for rock lined channels:

$$n = 0.0395(d_{50})^{1/6} = 0.0395(0.25)^{1/6} = 0.031$$

$$3.55 \text{ cfs} = \left(\frac{1.468}{0.031}\right) * (7\text{ft}+3y)y * \left[\frac{(7\text{ft}+3y)y}{7\text{ ft}+2y\sqrt{1+3^2}}\right]^{2/3} * 0.05^{1/2}$$

Flow depth (y) was found by trial and error to be 1.92 inches (0.16 ft). This depth is then used to calculate the area of flow with the equation in

Table **36**:

$$A = (B+my)y = (7\text{ft}+3*0.16\text{ft})0.16\text{ft} = 1.2\text{ft}^2$$

Flow velocity is then found with Equation 5-27:

$$v = \frac{Q_p}{A} = \frac{3.55\frac{\text{ft}^3}{\text{s}}}{1.2\text{ft}^2} = 3\frac{\text{ft}}{\text{s}}$$

The resulting velocity is less than $7\frac{\text{ft}}{\text{s}}$; therefore, it is satisfactory.

Step 7: Select pretreatment

Pretreatment for these trenches will be provided by the vegetated slopes. They must be maintained to specifications presented in the Vegetated Filter Strip Fact Sheet portion of this work. If they are not initially to those standards, the slope must be refurbished before installation of the trenches.

Section 6 Conclusions

Several conclusions can be drawn for each BMP which can be used to remediate run-off from highways and protect receiving waters.

- Vegetated filter strips are a viable option for pollutant removal. Existing roadside vegetation may already be acting as a BMP or may be easily retrofit to do so. The length, vegetation density, and slope are the primary design elements affecting performance of vegetated filter strips.
- Vegetated swales have shown to be a viable treatment option as stand-alone BMPs in some cases, as well as within a treatment train. They show high retrofit potential in existing drainage ditches which, when coupled with existing vegetated filter strips, may already be satisfying pollution removal requirements. Check dams may be required to protect vegetated swales from flow velocities which would damage, or limit their functionality.
- Bioretention is a flexible BMP which can add great aesthetic appeal. Bioretention is a very flexible BMP in regards to siting, targeting specific pollutants, vegetation, and infiltration capacity. Maintenance of bioretention facilities is generally higher than other BMPs, particularly early in the life of the BMP when plants are getting established.
- Sand filters have a track record as an effective BMP. Pollutant removal with sand filters has been shown to be very high. Although the initial construction cost of sand filters is substantial, maintenance is not overly burdensome or costly. The major component to the longevity of sand filters is the prevention of fine sediment reaching the filter, which can be done by stabilizing the watershed and incorporating a sedimentation basin.
- Horizontal filter trenches require more research, but their simplicity, applicability for roadside scenarios, and low maintenance burden suggest they are a strong candidate for

remediating roadway runoff. The primary concern for horizontal filter trenches is preventing fine sediment from clogging the system.

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Appendix A

Seed mixtures for Nebraska highways

Different regions in Nebraska are better suited for different grass mixtures. The Nebraska Department of Roads (NDOR) has separated the state into 6 landscape regions, as presented in Figure 50. There is a suggested seed mixture for each region in the following tables. Table 51 shows suggested mix for urban areas, which gives a manicured appearance and can tolerate frequent mowing (NDOR 2010).

Each region has grass mix suggestions for the shoulder region and the foreslope, ditch, backslope areas. The shoulder areas is the area within 16 ft (4.9 m) of the paved surface, and the foreslope, ditch, backslope areas is the area from the shoulder area to the end of the limits of the project (NDOR 2010).

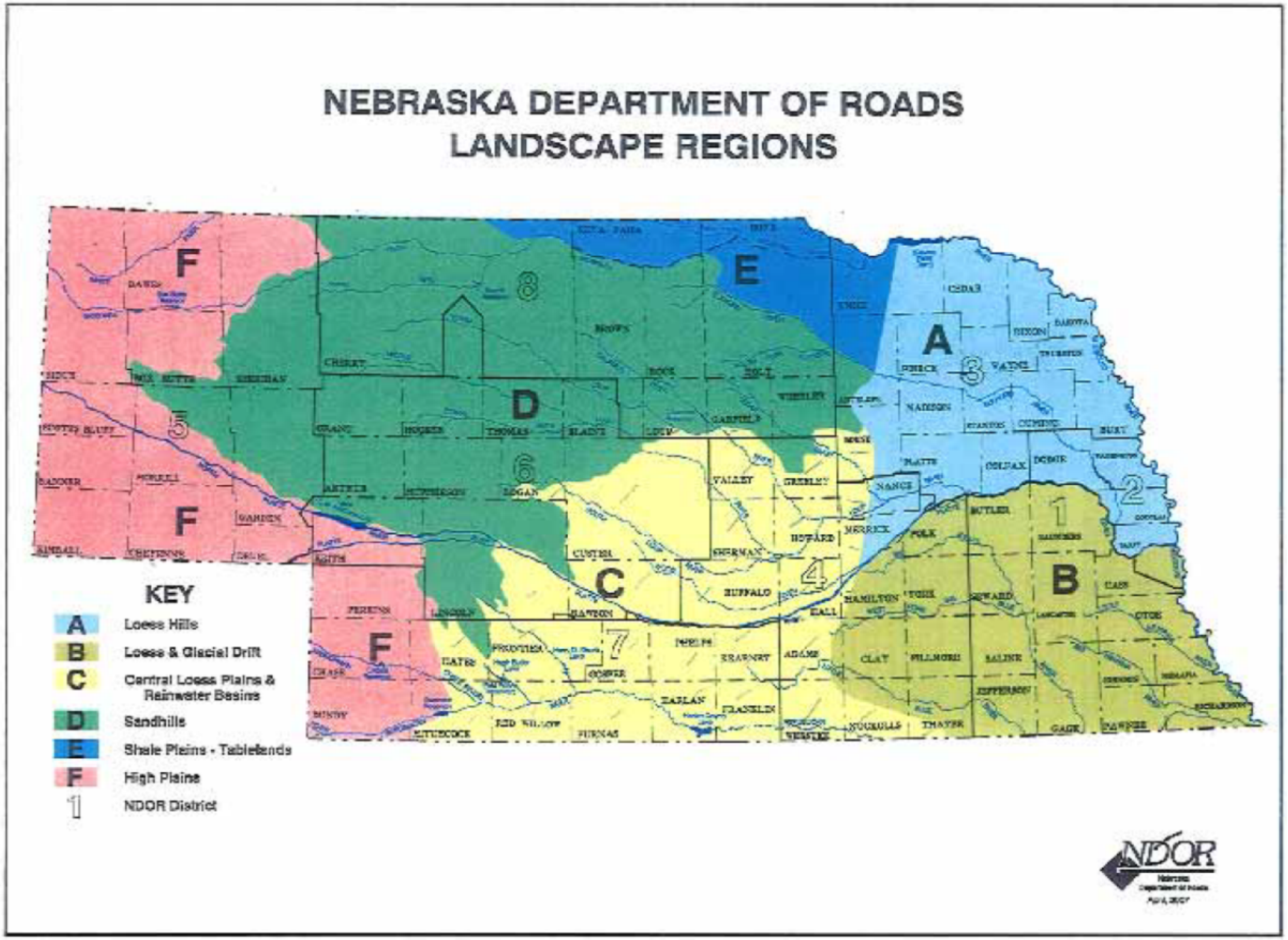

Figure 50: Nebraska Department of Roads landscape regions (NDOR 2010)

Seed Mixture for Region A: Loess Hills

Table 39: Rural highway shoulder mix Region A (NDOR 2010)

Rural Highway Shoulder Mixture

Species	Minimum Purity (percent)	Lbs. of PLS/acre
Perennial ryegrass – Linn	85	7
Slender wheatgrass	85	4
Western wheatgrass – Flintlock, Barton	85	6
Kentucky fescue	85	1.5
Blue grama – NE, KS, CO	30	2
Buffalograss – Cody, Bison, Sharp's Improved, Texoka	80	4
Sideoats grama – Trailway, Butte	75	3
Sand dropseed (Sporobolus cryptandrus)	90	0.2
Oats/Wheat (wheat in the fall)	90	14

Table 40: Grass mixture for foreslopes, ditches, and backslopes for Region A (NDOR 2010)

Foreslope, Ditch & Backslope Mixture

Species	Minimum Purity (percent)	Lbs. of PLS/acre
Canada wildrye – Mandan, Nebraska native	85	4
Slender wheatgrass	85	3
Western wheatgrass – Flintlock, Barton	85	4
Indiangrass – Oto, NE-54, Holt	75	3
Switchgrass – Pathfinder, Blackwell, Shawnee, Trailblazer	90	1.5
Big bluestem – Pawnee, Roundtree, Bonanza	60	3
Little bluestem – Blaze, Camper, Aldous, Nebr. native	60	2.5
Sand lovegrass – NE-27, Nebraska native	90	0.5
Purple prairie clover – Kaneb, inoculated OR Partridge pea - inoculated	90	0.25 or 0.25
Black-eyed Susan (Rudbeckia hirta)	85	0.4
Blue flax (Linum lewisii)	85	1
Plains coreopsis (Coreopsis tinctoria)	85	0.3
Mexican red hat (Ratibida columnifera, red)	85	0.5
Oats/Wheat (wheat in the fall)	90	10

PLS (pure live seed) describes the amount of seed that will germinate.

Seed Mixture for Region B: Loess and Glacial Drift

Table 41: Rural highway shoulder mix Region B (NDOR 2010)

Rural Highway Shoulder Mixture

Species	Minimum Purity (percent)	Lbs. of PLS/acre
Perennial ryegrass – Linn	85	7
Slender wheatgrass	85	4
Western wheatgrass – Flintlock, Barton	85	6
Kentucky fescue	85	1.5
Blue grama – NE, KS, CO	30	2
Buffalograss – Cody, Bison, Sharp's Improved, Texoka	80	5
Sideoats grama – Trailway, Butte, El Reno	75	4
Sand dropseed (Sporobolus cryptandrus)	90	0.2
Oats/Wheat (wheat in the fall)	90	14

Table 42: Grass mixture for foreslopes, ditches, and backslopes for Region B (NDOR 2010)

Foreslope, Ditch & Backslope Mixture

Species	Minimum Purity (percent)	Lbs. of PLS/acre
Canada wildrye – Mandan, Nebraska native	85	4
Slender wheatgrass	85	3
Western wheatgrass – Flintlock, Barton	85	4
Indiangrass – Oto, NE-54, Holt	75	3
Switchgrass – Pathfinder, Blackwell, Trailblazer	90	1.5
Big bluestem – Pawnee, Roundtree, Bonanza	60	3
Little bluestem – Aldous, Blaze, Camper, Nebraska native	60	2.5
Sideoats grama – Butte, El Reno, Trailway	75	4
Illinois bundleflower – inoculated OR Partridge pea – inoculated	90	0.25 or 0.25
Black-eyed Susan (Rudbeckia hirta)	85	0.4
Blue flax (Linum lewisii)	85	1
Rocky Mountain bee plant (Cleome serrulata)	85	0.3
Grayhead prairie coneflower (Ratibida pinnata)	85	0.25
Oats/Wheat (wheat in the fall)	90	10

PLS (pure live seed) describes the amount of seed that will germinate.

Seed Mixture for Region C: Central Loess Plains and Rainwater Basin

Table 43: Rural highway shoulder mix Region C (NDOR 2010)

Rural Highway Shoulder Mixture

Species	Minimum Purity (percent)	Lbs. of PLS/acre
Perennial ryegrass – Linn	85	7
Slender wheatgrass	85	4
Western wheatgrass – Barton, Flintlock	85	6
Kentucky fescue	85	1.5
Blue grama – NE, KS, CO	30	2.5
Buffalograss – Cody, Bison, Sharp's Improved, Texoka	80	5
Sideoats grama – Butte, Trailway	75	4
Sand dropseed (Sporobolus cryptandrus)	85	0.2
Oats/Wheat (wheat in the fall)	90	14

Table 44: Grass mixture for foreslopes, ditches, and backslopes for Region C (NDOR 2010)

Foreslope, Ditch and Backslope Mixture

Species	Minimum Purity (percent)	Lbs. of PLS/acre
Canada wildrye * – Mandan, Nebraska native	85	4
Virginia wildrye – Omaha, Cuivre River, Nebraska native	85	3
Slender wheatgrass	85	4
Western wheatgrass – Barton, Flintlock	85	4
Indiangrass – Holt, NE-54, Oto	75	3
Switchgrass – Blackwell, NE-28, Trailblazer	90	1.5
Big bluestem – Pawnee, Kaw, Bonanza, Champ	60	3
Little bluestem – Aldous, Cimarron, Camper, Nebraska native	60	2.5
Sideoats grama – Butte, Trailway	75	3
Sand lovegrass – NE-27, Nebraska native	90	0.5
Purple prairie clover – inoculated OR Partridge pea – inoculated	90	0.5 or 0.25
Maximilian sunflower (Helianthus maximiliani)	85	0.75
Rocky Mountain bee plant (Cleome serrulata)	85	0.3
Upright prairie coneflower (Ratibida columnifera)	85	0.5
Yarrow (Achillea millefolium)	85	0.2
Oats/Wheat (wheat in the fall)	90	10

* Don't include Canada wildrye in mixtures for Frontier, Hitchcock, or Red Willow Counties

PLS (pure live seed) describes the amount of seed that will germinate.

Seed Mixture for Region D: Sandhills

Table 45: Rural highway shoulder mix Region D (NDOR 2010)

Rural Highway Shoulder Mixture

Species	Minimum Purity (percent)	Lbs. of PLS/acre
Perennial ryegrass – Linn	85	7
Slender wheatgrass	85	4
Western wheatgrass – Rodan, Rosana, Barton, Flintlock	85	6
Kentucky fescue	85	1.5
Blue grama – NE, KS, CO	30	3
Sideoats grama – Pierre, Butte	75	4
Sand dropseed (Sporobolus cryptandrus)	90	0.2
Sand lovegrass – NE-27, Nebraska native	90	1
Purple prairie clover – inoculated	90	0.2
Rye	90	16

Table 46: Grass mixture for foreslopes, ditches, and backslopes for Region D

Foreslope, Ditch and Backslope Mixture

Species	Minimum Purity (percent)	Lbs. of PLS/Acre
Canada wildrye * – Mandan, Nebraska native	85	4
Western wheatgrass – Rodan, Rosana, Barton, Flintlock	85	5
Slender wheatgrass	85	4
Thickspike wheatgrass (western sandhills) – Critana	85	3
Indiangrass - Holt	75	3
Switchgrass – NE-28, Pathfinder, Trailblazer, Blackwell	90	1.5
Sand bluestem – Gold Strike, Garden County, Champ	60	3
Little bluestem – Cimarron, Pastura, Nebraska native	60	2
Prairie sandreed – Goshen, Pronghorn	40	0.75
Sand lovegrass – NE-27, Nebraska native	90	0.5
Purple prairie clover – inoculated	90	0.5
Blue flax (Linum lewisii)	85	1
Upright prairie coneflower (Ratibida columnifera)	85	1
Plains coreopsis (Coreopsis tinctoria)	85	0.3
Rocky Mountain bee plant (Cleome serrulata)	85	0.3
Cereal Rye	90	14

* Don't include Canada wildrye for mixtures in Frontier, Hayes, Keith, or Lincoln Counties

PLS (pure live seed) describes the amount of seed that will germinate.

Seed Mixture for Region E: Shale Plains-Tablelands

Table 47: Rural highway shoulder mix Region E (NDOR 2010)

Rural Highway Shoulder Mixture

Species	**Minimum Purity (percent)**	**Lbs. of PLS/acre**
Perennial ryegrass – Linn	85	7
Slender wheatgrass	85	4
Western wheatgrass – Rosana, Rodan, Barton, Flintlock	85	6
Kentucky fescue	85	1.5
Blue grama – NE, KS, CO	30	2.5
Buffalograss – Bison, Cody, Sharp's Improved, Texoka	80	5
Sideoats grama – Butte, Pierre, Trailway	75	4
Sand dropseed (Sporobolus cryptandrus)	90	0.1
Oats/Wheat (wheat in the fall)	90	14

Table 48: Grass mixture for foreslopes, ditches, and backslopes for Region E (NDOR 2010)

Foreslope, Ditch & Backslope Mixture

Species	**Minimum Purity (percent)**	**Lbs. of PLS/acre**
Canada wildrye – Mandan, Nebraska native	85	4
Green needlegrass (Nassella viridula) – Lodorm	75	2
Western wheatgrass – Rosana, Rodan, Barton, Flintlock	85	5
Switchgrass – Blackwell, NE-28, Pathfinder, Trailblazer	90	1.5
Big bluestem – Champ, Bonanza, Pawnee, Roundtree	60	3
Sideoats grama – Butte, Pierre, Trailway	75	4
Little bluestem – Camper, Blaze, Pastura, Nebraska native	60	2
Blue grama – NE, KS, CO	30	0.5
Purple prairie clover – inoculated	90	0.5
Black-eyed Susan (Rudbeckia hirta)	85	0.5
Blue flax (Linum lewisii)	85	1
Yarrow (Achillea millefolium)	85	0.2
Mexican red hat (Ratibida columnifera, red)	85	0.5
Oats/Wheat (wheat in the fall)	90	10

PLS (pure live seed) describes the amount of seed that will germinate.

Seed Mixture for Region F: High Plains

Table 49: Rural highway shoulder mix Region F (NDOR 2010)

Rural Highway Shoulder Mixture

Species	Minimum Purity (percent)	Lbs. of PLS/acre
Perennial ryegrass – Linn	85	8
Slender wheatgrass	85	4
Western wheatgrass – Arriba, Barton, Flintlock, Rodan, Rosana	85	6
Kentucky fescue	85	2
Blue grama – NE, KS, CO	30	2.5
Buffalograss – Bison, Cody, Sharp's Improved, Texoka	80	4
Sideoats grama – Butte, El Reno, Pierre	75	4
Sand dropseed (Sporobolus cryptandrus)	90	0.2
Sand lovegrass – NE-27, Nebraska native	90	0.5
Oats or wheat	90	14

* Use of Canada wildrye is limited to Banner, Box Butte, Dawes, Kimball, Morrill, Sheridan, Scotts Bluff, and Sioux Counties in this region

Table 50: Grass mixture for foreslopes, ditches, and backslopes for Region F (NDOR 2010)

Foreslope, Ditch & Backslope Mixture

Species	Minimum Purity (percent)	Lbs. of PLS/acre
Canada wildrye * – Mandan, Nebraska native	85	4
Slender wheatgrass	85	4
Thickspike wheatgrass – Critana	85	3
Western wheatgrass – Arriba, Barton, Flintlock, Rodan, Rosana	85	6
Switchgrass – NE-28, Trailblazer	90	1.5
Little bluestem – Camper, Cimarron, Pastura, Nebraska native	60	2.5
Blue grama – NE, KS, CO	30	0.5
Buffalograss – Bison, Cody, Sharp's Improved, Texoka	80	2
Sideoats grama – Butte, Pierre, El Reno	75	4
Sand dropseed (Sporobolus cryptandrus)	90	0.2
Purple prairie clover – inoculated	90	1
Blue flax (Linum lewisii)	85	2
Rocky Mountain bee plant (Cleome serrulata)	85	0.5
Upright prairie coneflower (Ratibida columnifera)	85	1
Mexican red hat (Ratibida columnifera, red)	85	1
Oats or wheat	90	10

PLS (pure live seed) describes the amount of seed that will germinate.

Table 51: Grass mixture for urban roadsides and lawns (NDOR 2010)

URBAN ROADSIDES AND LAWNS

Species	Minimum Purity	Broadcast or Hydraulic Seeder Application Rate in lb. of PLS/Acre	Approved Mechanical Drill Application Rate in lb. of PLS/Acre
Turf type perennial ryegrass	90	30	15
Turf type tall fescue	90	528	264
Kentucky bluegrass	90	42	21

PLS (pure live seed) describes the amount of seed that will germinate.

Appendix B

Gradation for common BMP media

Table 52: Gradation for AASHTO M-6 and ASTM C33 Sands

	Cumulative Passing by Weight	
U.S. Standard Seive Size	AASHTO M-6 (Belgard 2012)	ASTM C33 (Division 30 2008)
3/8"	100	100
#4	95 to 100	95 to 100
#8	80 to 100	85 to 100
#16	50 to 85	50 to 85
#30	25 to 60	25 to 60
#50	10 to 30	0 to 30
#100	2 to 10	2 to 10

Table 53: Gradation for AASHTO #3 gravel

	Cumulative Passing by Weight
U.S. Standard Seive Size	AASHTO # 3 (PROP 2003)
2.5"	
2"	100
1.5"	90 to 100
1"	35 to 70
3/4"	0 to 15
#4	0 to 5
#200	<5