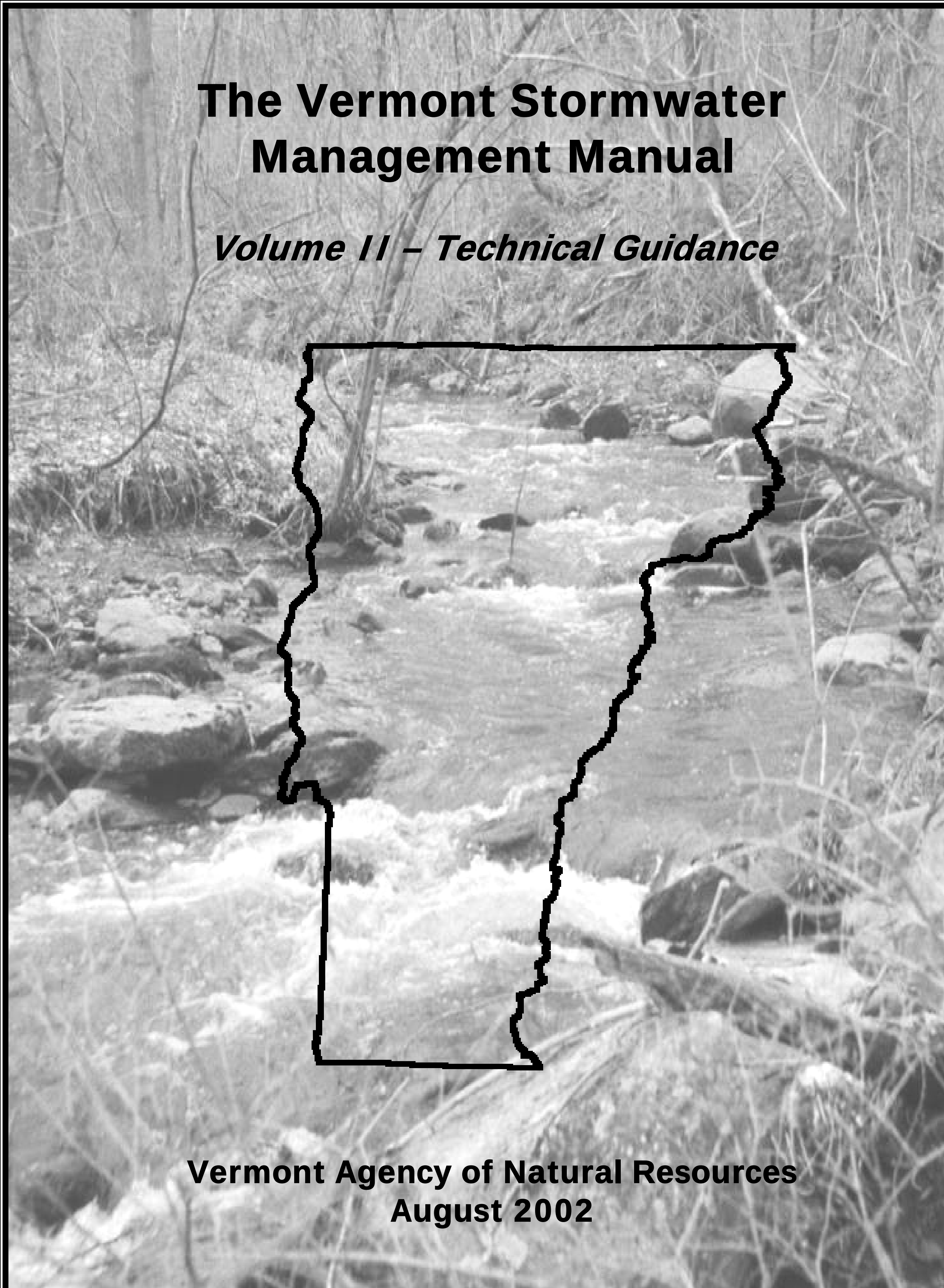
The Vermont Stormwater Management Manual

Volume II – Technical Guidance



Vermont Agency of Natural Resources
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Introduction

1. Purpose of Manual

The purpose of this manual is threefold:

A. To protect the waters of the State of Vermont from the adverse impacts of stormwater runoff.

B. To provide design guidance on the most effective stormwater treatment practices (STPs) for new development sites, and to improve the quality of STPs that are constructed in the State, specifically in regard to their performance, longevity, safety, ease of maintenance, community acceptance and environmental benefit.

C. To foster a comprehensive stormwater management approach that integrates site design and nonstructural practices with the implementation of structural STPs.

2. Regulatory Authority, Applicability, and Review

This manual was produced to provide technical analysis and design guidance for the Vermont Agency of Natural Resources Stormwater Management Program. This program is authorized under 10 V.S. A. §1264, with specific regulatory guidance provided by the Stormwater Management Rules, and the applicable Stormwater Discharge - General Permits.

This manual provides design guidance on the most effective stormwater treatment for a variety of site types. These treatments represent the best practices for a given site, and for the purposes of the Vermont stormwater regulatory program. The site design guidance in this manual is not intended to discourage growth center development nor encourage scattered development.

3. How to Use the Manual

The Vermont Stormwater Management Manual provides designers a general overview on how to size, design, select and locate STPs at a development site to comply with State stormwater performance goals. The Manual also contains appendices with more detailed information on landscaping, STP construction specifications, step-by-step STP design examples and other assorted design tools.

The Manual is organized as follows:

Appendix A. Landscaping Guidance/Plant Lists. Good landscaping can often be an important factor in the performance and community acceptance of many stormwater STPs. The Landscaping Guide provides general background on how to determine the appropriate landscaping region and hydrologic zone in Vermont. Appendix A also includes tips on how to establish more functional landscapes within stormwater STPs, and contains an extensive list of trees, shrubs, ground covers, and wetland plants that can be used to develop an effective and diverse planting plan.

Appendix B. STP Construction Specifications. Good designs only work if careful attention is paid to proper construction techniques and materials. Appendix B contains detailed specifications for constructing ponds, infiltration practices, filters, bioretention areas and open channels.

Appendix C. Step-by-Step Design Examples. Five design examples are provided to help designers and plan reviewers better understand the new criteria in this Manual. The examples demonstrate how the new stormwater sizing criteria are applied, and some of the design procedures and performance criteria that should be considered when siting and designing stormwater management practices. Examples are provided for a wet pond, sand filter, infiltration trench, open channel system, and bioretention facility.

Appendices D. Assorted Design Tools. Appendix D provides an assortment of design tools that can be used by engineers and designers to develop effective stormwater management plans for a site. Guidance is provided on site testing requirements for specific practices, design details for compliance with practice performance criteria, estimating water quality peak flow, critical erosive velocities, the distributed runoff control methodology, and inspection and maintenance of practices.

4. Symbols and Acronyms

As an aid to the reader, Table 1 outlines common symbols and acronyms that are used throughout the text. In addition, a glossary is provided at the end of this volume that defines the terminology used in the text.

Table 1. Key Symbols and Acronyms Cited in Manual

Symbol	Description	Symbol	Description
a	channel cross-sectional area	P	precipitation depth
A	drainage area	Q_a	unit runoff for water quality storm
A_f	filter bed area	q_i	peak inflow discharge
A_s	surface area, sedimentation basin	q_o	peak outflow discharge
STP	stormwater treatment practice	Q_{p10}	overbank flood control peak discharge
cfs	cubic feet per second	Q_{p100}	extreme flood peak discharge
Cp_v	channel protection storage volume	q_u	unit peak discharge
CMP	corrugated metal pipe	Q_{wq}	water quality peak discharge
cms	cfs per square mile	Re_v	recharge volume
CN	curve number	Re_a	recharge area requiring treatment
d_f	depth of filter bed	R_v	volumetric runoff coefficient
du	dwelling units	ROW	right of way
AOT	Agency of Transportation	SD	separation distance
DPW	Department of Public Works	t_c	time of concentration
ED	extended detention	t_t	time to drain filter bed
F	soil specific recharge factor	TP	Total Phosphorus
f_c	soil infiltration rate	TR-20	Technical Release No. 20 Project Formulation-Hydrology, computer program
fps	feet per second	TR-55	Technical Release No. 55 Urban Hydrology for Small Watersheds
g	Acceleration due to gravity	TSS	total suspended solids
h_f	head above filter bed	v	velocity
HECRAS	water surface profile analysis computer program	Vr	volume of runoff
HSG	hydrologic soil group	Vs	volume of storage
Ia	initial abstraction	Vt	total volume
I	percent impervious cover	Vv	volume of voids
K	coefficient of permeability	WQ_v	water quality storage volume
NPDES	National Pollution Discharge Elimination System	WSE	water surface elevation
NRCS	Natural Resources Conservation Service		

5. Why Stormwater Matters: Impact of Runoff on Vermont Watersheds

Land development can have a significant influence on the quality of Vermont's waters. To start, development seriously alters the local hydrologic cycle (see Figure 1). The hydrology of a site changes during the initial clearing and grading that occur during construction. Trees that had intercepted rainfall are removed, and natural depressions that had temporarily ponded water are graded to a uniform slope. The spongy humus layer of the forest floor that had absorbed rainfall is scraped off, eroded or severely compacted. Having lost its natural storage capacity, a cleared and graded site can no longer prevent rainfall from being rapidly converted into stormwater runoff. With this increase in runoff volume comes an increase in sediment load that can significantly affect receiving water body health.

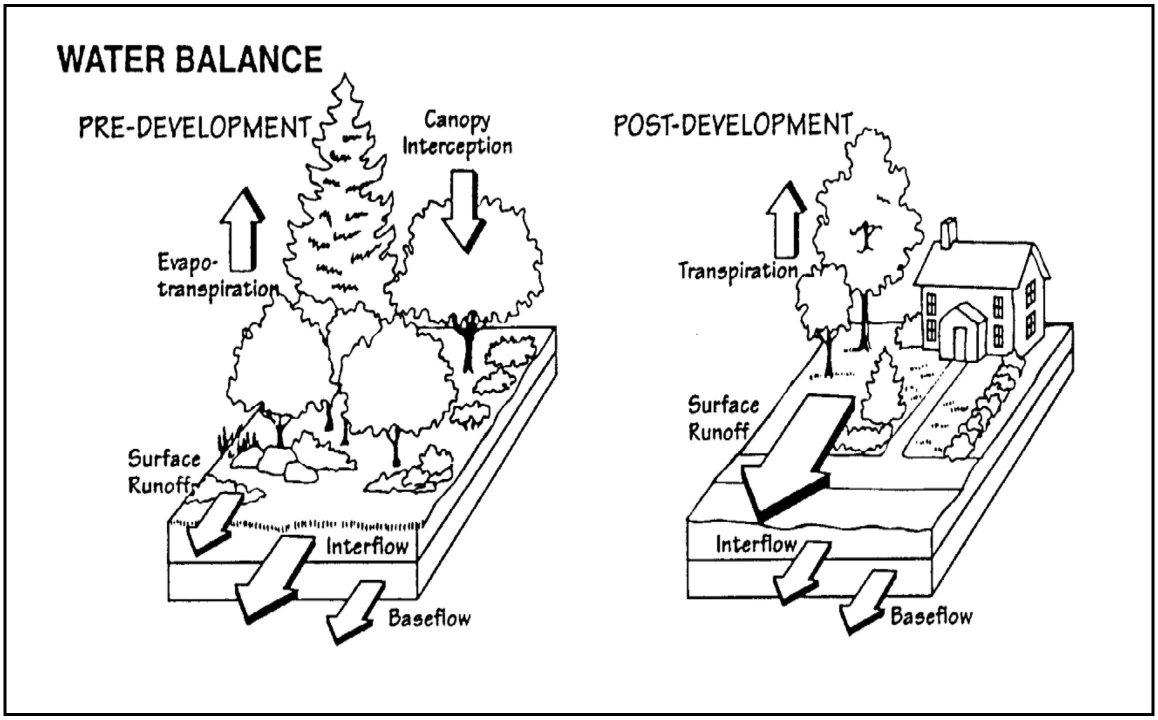

Figure 1. Water Balance at a Developed and Undeveloped Site (Schueler, 1987)

The situation degrades after construction. Rooftops, roads, parking lots, driveways and other impervious surfaces no longer allow rainfall to soak into the ground. Consequently, most rainfall is directly converted into stormwater runoff. This phenomenon is illustrated in Figure 2, which shows the increase in the volumetric runoff coefficient (R_v) as a function of site imperviousness. The runoff coefficient expresses the fraction of rainfall volume that is converted into stormwater runoff. As can be seen, the volume of stormwater runoff increases sharply with impervious cover. For example, a one-acre parking lot can produce 16 times more stormwater runoff than a one-acre meadow each year (Schueler, 1994). Similarly, conversion of agriculture lands to impervious surfaces has a corresponding increase in surface runoff.

The increase in stormwater runoff can be too much for the existing drainage system to handle. As a result, the drainage system is often “improved” to rapidly collect runoff and quickly convey it away (using curb and gutter, enclosed storm sewers, and lined channels). The stormwater runoff is subsequently discharged to downstream streams, reservoirs, or lakes. This phenomenon is of particular concern for mountainous areas such as much of Vermont, where flow energy and erosive forces increase with increased slope.

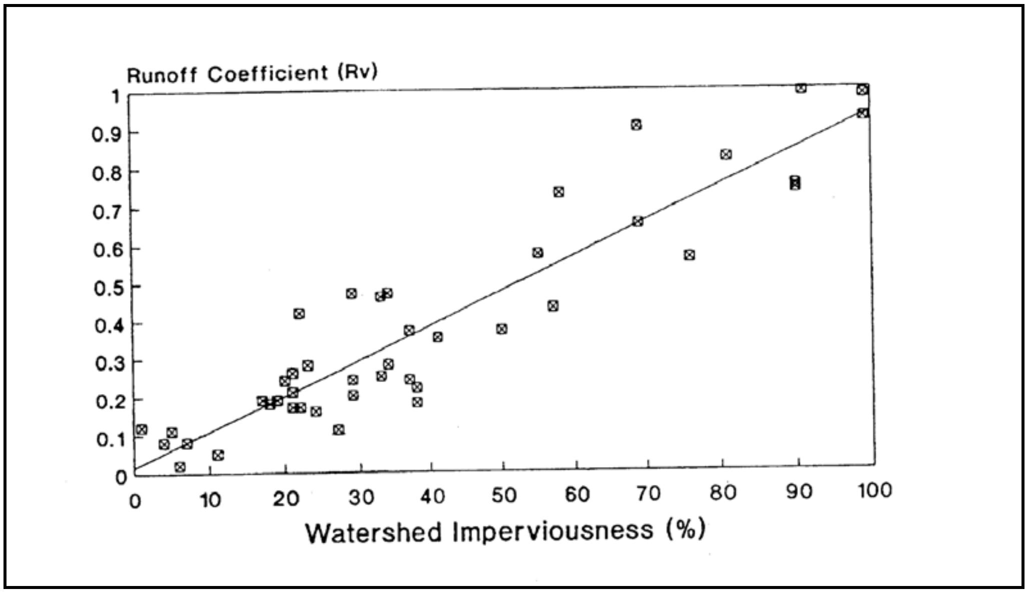

Figure 2. Relationship Between Impervious Cover & Runoff Coefficient (Schueler,1987)

5.1. Declining Water Quality

Impervious surfaces accumulate pollutants deposited from the atmosphere, leaked from vehicles, or windblown in from adjacent areas. During storm events, these pollutants quickly wash off, and are rapidly delivered to downstream waters. Some common pollutants found in stormwater runoff are described below and profiled in Table 2.

<u>Sediment (Suspended Solids)</u>

Sources of sediment include washoff of particles that are deposited on impervious surfaces and the erosion of streambanks and construction sites. Streambank erosion is a particularly important source of sediment, and some studies suggest that streambank erosion accounts for up to 70% of the sediment load in urbanizing watersheds (Trimble, 1997).

Both suspended and deposited sediments can have adverse effects on aquatic life in streams and lakes. Turbidity resulting from sediment can reduce light penetration for submerged aquatic vegetation critical to lake littoral zones. In addition, the energy from light reflecting off of suspended sediment can increase water temperatures (Kundell and Rasmussen, 1995). Sediment can physically alter habitat by destroying the riffle-pool structure in stream systems and smothering benthic organisms. Finally, sediment transports many other pollutants to the receiving waters.

Table 2. National Median Concentrations for Common Chemical Constituents found in Stormwater

Constituent	Units	Concentration
Total Suspended Solids[1]	mg/l	54.5
Total Phosphorus[1]	mg/l	0.26
Soluble Phosphorus[1]	mg/l	0.10
Total Nitrogen[1]	mg/l	2.00
Total Kjeldhal Nitrogen[1]	mg/l	1.47
Nitrite and Nitrate[1]	mg/l	0.53
Copper[1]	ug/l	11.1
Lead[1]	ug/l	50.7
Zinc[1]	ug/l	129
BOD[1]	mg/l	11.5
COD[1]	mg/l	44.7
Organic Carbon[2]	mg/l	11.9
PAH[3]	mg/l	3.5*
Oil and Grease[4]	mg/l	3.0*
Fecal Coliform[5]	col/100 ml	15,000*
Fecal Strep[5]	col/100 ml	35,400*
Chloride (snowmelt)[6]	mg/l	116

* Represents a Mean Value
Source:
1: Pooled NURP/USGS (Smullen and Cave, 1998)
2: Derived from the National Pollutant Removal Database (Winer, 2000)
3: Rabanal and Grizzard 1996
4: Crunkilton et al. (1996)
5: Schueler (1999)
6: Oberts 1994

Nutrients

Runoff from developed land has elevated concentrations of both phosphorus and nitrogen, which can enrich streams, lakes, and reservoirs (known as eutrophication). Significant sources of nitrogen and phosphorus include fertilizer, atmospheric deposition, pet waste, organic matter, and stream bank erosion. Another significant source of nitrogen is fossil fuel combustion from automobiles, power plants and industry. Data from the upper Midwest suggest that lawns are a significant contributor, with concentrations as much as four times higher than other land uses, such as streets, rooftops, or driveways (Steuer et al., 1997; Waschbusch et al., 2000; Bannerman et al., 1993).

Nutrients, particularly phosphorus, are of particular concern to fresh water lakes, and are a source of documented degradation in some of Vermont's waters, including Lake Champlain.

Organic Carbon

Organic matter, washed from impervious surfaces during storms, can present a problem in slower moving downstream waters. Some sources include organic material blown onto the street surface, and attached to sediment from stream banks, or from bare soil. In addition, organic carbon is formed indirectly from phosphorus loading to lake systems.

As organic matter decomposes, it can deplete dissolved oxygen in lakes. Low levels of oxygen in the water can have an adverse impact on aquatic life. Another concern is that tetrahalomethane (THM), a carcinogenic disinfection by-product, is formed when chlorine is mixed with water high in organic carbon in drinking water supply reservoirs.

Bacteria

Bacteria levels in stormwater runoff routinely exceed public health standards for water contact recreation. Some stormwater sources include pet waste and wildlife. Other sources in developed land include sanitary and combined sewer overflows, wastewater, and illicit connections to the storm drain system.

Bacteria is a leading contaminant in many of Vermont's waters, and has lead to beach closures in the Burlington Area.

Hydrocarbons

Vehicles leak oil and grease that contain a wide array of hydrocarbon compounds, some of which can be toxic at low concentrations to aquatic life. Most sources are automotive, and higher runoff concentrations have been documented from land uses such as: gas stations, vehicle service and maintenance yards, and public works storage areas (Schueler, 1994).

Trace Metals

Cadmium, copper, lead and zinc are routinely found in stormwater runoff. Many of the sources are associated with automotive uses. Other sources include paints, road salts, and galvanized pipes.

These metals can be toxic to aquatic life at certain concentrations, and can also accumulate in the bottom sediments of lakes. Specific concerns in aquatic systems include bioaccumulations in fish and macro-invertebrates, and the impact of toxic bottom sediments on bottom-dwelling species.

Pesticides

A modest number of currently used and recently banned insecticides and herbicides have been detected in urban and suburban streamflow at concentrations that approach or exceed toxicity thresholds for aquatic life. Key sources of pesticides include lawns and road rights-of-way.

Chlorides

Salts that are applied to roads and parking lots in the winter months appear in stormwater runoff and meltwater at much higher concentrations than many freshwater organisms can tolerate. One study of four Adirondack streams found severe impacts to macroinvertebrate species attributed to chlorides (Demers and Sage, 1990). In addition to the direct toxic effects, chlorides can impact lake systems by altering their mixing cycle. A primary source of chlorides in Vermont, particularly in the northern regions, is salt applied to road surfaces as a deicer.

Thermal Impacts

Impervious surfaces may increase temperature in receiving waters, adversely impacting aquatic life that requires cold and cool water conditions. Data suggest that increasing development can increase stream temperatures by between five and twelve degrees Fahrenheit, and that the increase is related to the level of impervious cover in the drainage area (Galli, 1990a). Thermal impacts are a serious concern in trout waters, where cold temperatures are critical to species survival.

Trash and Debris

Considerable quantities of trash and debris are washed through the storm drain networks. The trash and debris accumulate in streams and lakes and detract from their natural beauty.

Snowmelt Concentrations

The snow pack can store hydrocarbons, oil and grease, chlorides, sediment, and nutrients. In cold regions, the pollutant load during snowmelt can be significant, and chemical traits of snowmelt change over the course of the melt event. Oberts (1994) studied this phenomenon, and describes four types of snowmelt runoff (Table 3). Oberts and others have reported that 90% of the hydrocarbon load from snowmelt occurs during the last 10% of the event. From a practical standpoint, the high hydrocarbon loads experienced toward the end of the season suggest that stormwater management practices should be designed to capture a percentage of the seasonal snowmelt event.

Table 3. Runoff and Pollutant Characteristics of Snowmelt Stages (Oberts, 1994)

Snowmelt Stage	Duration/ Frequency	Runoff Volume	Pollutant Characteristics
Pavement Melt	Short, but many times in winter	Low	Acidic, high concentrations of soluble pollutants, Cl, nitrate, lead. Total load is minimal.
Roadside Melt	Moderate	Moderate	Moderate concentrations of both soluble and particulate pollutants.
Pervious Area Melt	Gradual, often most at end of season	High	Dilute concentrations of soluble pollutants, moderate to high concentrations of particulate pollutants, depending on flow.
Rain-on-Snow Melt	Short	Extreme	High concentrations of particulate pollutants, moderate to high concentrations of soluble pollutants. High total load.

5.2. Diminishing Groundwater Recharge and Quality

The slow infiltration of rainfall through the soil layer is essential for replenishing groundwater. Groundwater is a critical water resource across the State. Not only do many residents depend on groundwater for their drinking water, but the health of many aquatic systems is also dependent on its steady discharge. For example, during periods of dry weather, groundwater sustains flows in streams and helps to maintain the hydrology of wetlands.

Because development creates impervious surfaces that prevent natural recharge, a net decrease in groundwater recharge rates has been documented in urban watersheds (Spinello and Simmons, 1992). Thus, during prolonged periods of dry weather, streamflow sharply diminishes. In smaller headwater streams, the decline in stream flow can cause a perennial stream to become seasonally dry.

Urban land uses and activities can also degrade *groundwater quality,* if stormwater runoff is directed into the soil without adequate treatment. Certain land uses and activities are known to produce higher loads of metals and toxic chemicals and are designated as *stormwater hotspots*. Soluble pollutants, such as chloride, nitrate, copper, dissolved solids and some polycyclic aromatic hydrocarbons (PAH's) can migrate into groundwater and potentially contaminate wells. Stormwater runoff should never be infiltrated into the soil if a site is a designated hotspot, unless it receives full pretreatment with another practice.

5.3. Impacts to the Stream Channel

As pervious meadows and forests are converted into less pervious urban soils, or pavement, both the frequency and magnitude of storm flows increase significantly. As a result, the bankfull event occurs two to seven times more frequently after development occurs (Leopold, 1994). In addition, the discharge associated with the original bankfull storm event can increase by up to five times (Hollis, 1975). As Figure 3 demonstrates, the total flow beyond the "critical erosive velocity" increases substantially after development occurs. The increased energy resulting from these more frequent bankfull flow events results in erosion and enlargement of the stream channel, and consequent habitat degradation.

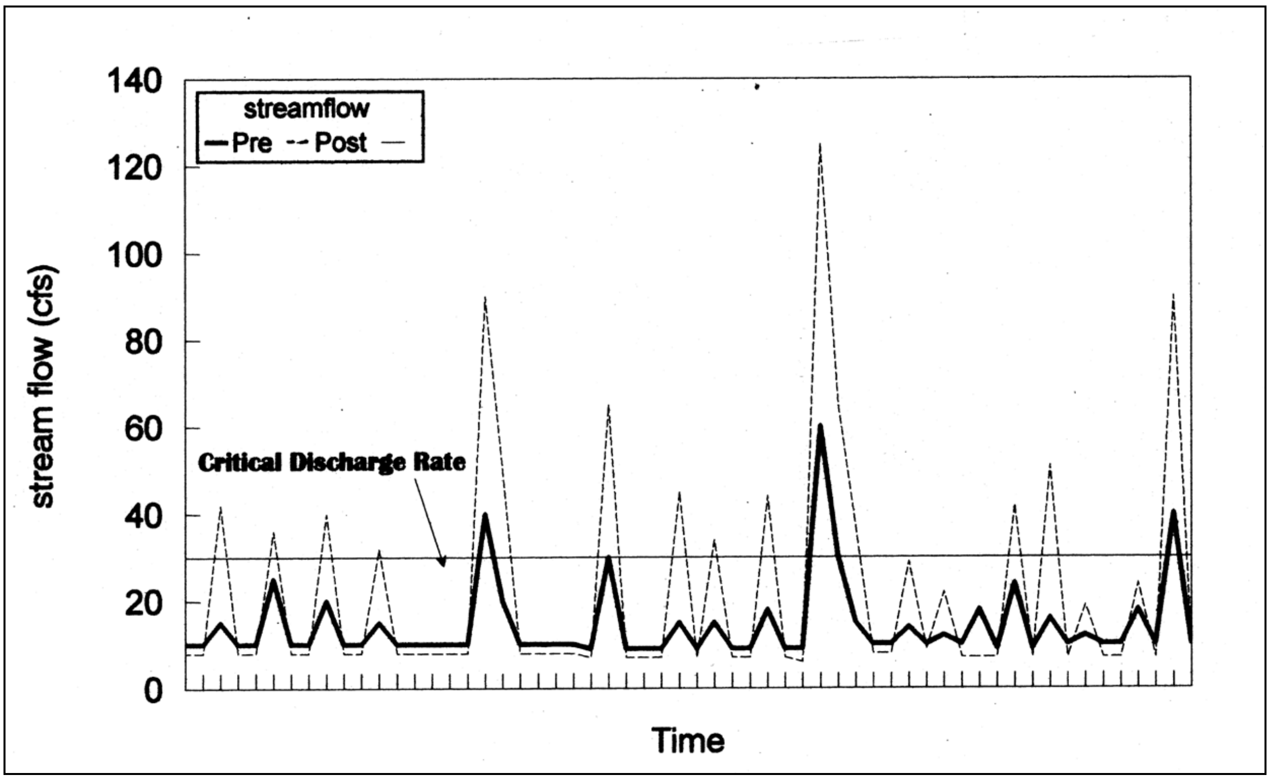

Figure 3. Increased Frequency of Erosive Velocities After Development

Channel enlargement in response to watershed development has been observed for decades, with research indicating that stream channel area expands to between two and five times its original size in response to upland development (Hammer, 1972; Morisawa and LaFlure, 1979; Allen and Narramore, 1985; Booth, 1990). One researcher developed a direct relationship between the level of impervious cover and the "ultimate" channel enlargement, the area a stream will eventually reach over time

(MacRae, 1996; Figure 4). A 1999 Stream Geomorphic Assessment found that this channel enlargement phenomena was also applicable to Vermont streams where channel cross-sectional area increased from approximately 1.25 to 2.0 times more than the pre-developed cross-sectional area for impervious cover between 6 and 22% (CWP, 1999).

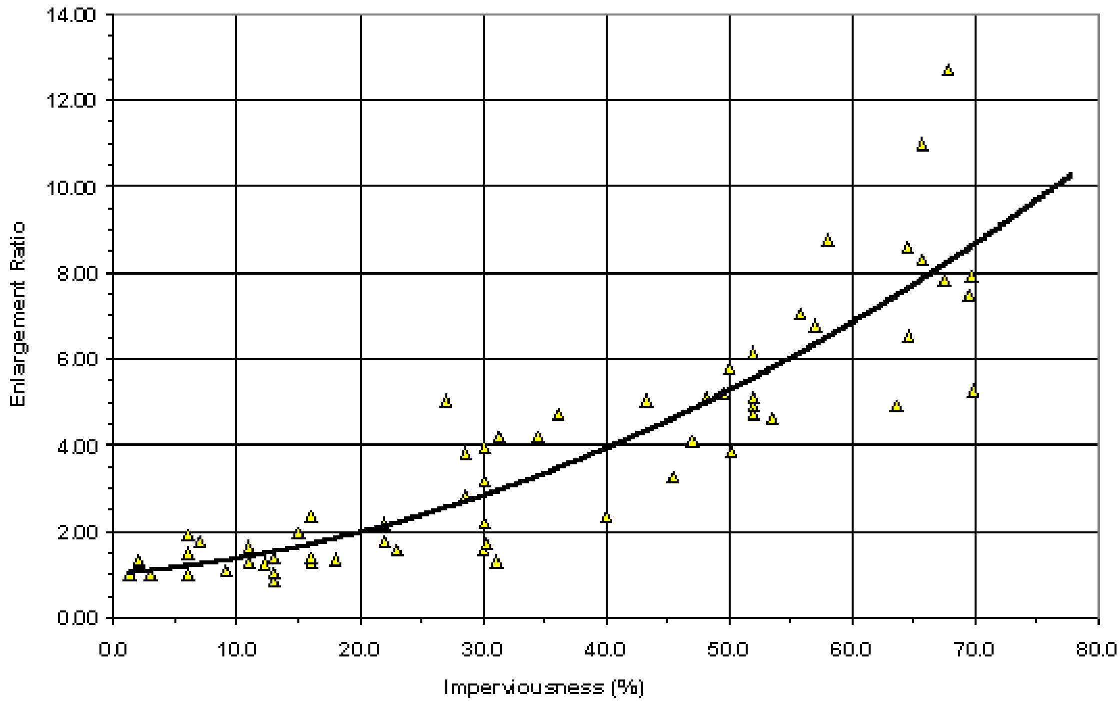

Figure 4. Relationship Between Impervious Cover and Channel Enlargement

Historically, Vermont has used 2-year control (i.e., reduction of the peak flow from the 2-year storm to predeveloped levels) to prevent channel erosion. Research suggests that this measure does not adequately protect stream channels effectively (McCuen and Moglen, 1988, MacRae, 1996). Although the peak flow is lower, it is also extended over a longer period of time, thus increasing the duration of erosive flows. In addition, the bankfull flow event actually becomes more frequent after development occurs. Consequently, capturing the two-year event may not address the channel-forming event.

This stream channel erosion and expansion, combined with direct impacts to the stream system, act to decrease the habitat quality of the stream. As a result, streams experience the following impacts to habitat (Table 4):

- Decline in stream substrate quality (through sediment deposition and embedding of the substrate)
- Loss of pool/riffle structure in the stream channel
- Degradation of stream habitat structure
- Creation of fish barriers by culverts and other stream crossings.
- Loss of “large woody debris,” which is critical to fish habitat.

Table 4. Impacts to Stream Habitat

Stream Channel Impact	Key Finding	Reference	Year
Habitat Characteristics			
Embeddedness	Interstitial spaces between substrate fill with increasing watershed imperviousness	Horner et al.	1996
Large Woody Debris (LWD)	Important for habitat diversity and anadramous fish.	Spence	1996
	Decreased LWD with increases in imperviousness	Booth et al.	1996
Changes in Stream Features	Altered pool/riffle sequence with urbanization	Richey	1982
	Loss of habitat diversity	Scott et al.	1986
Direct Channel Impacts			
Reduction in 1st Order Streams	Replaced by storm drains and pipes increases erosion rate downstream	Dunne and Leopold	1972
Channelization and hardening of stream channels	Increase instream velocities often leading to increased erosion rates downstream	Sauer et al.	1983
Fish Blockages	Fish blockages caused by bridges and culverts	MWCOG	1989

5.4. Increased Overbank Flooding

Flow events that exceed the capacity of the stream channel spill out into the adjacent floodplain. These are termed “overbank” floods, and can damage property and downstream drainage structures.

While some overbank flooding is inevitable and even desirable, the historical goal of drainage design in Vermont has been to maintain pre-development peak discharge rates for the two and prior to this, the ten-year frequency storms, thus keeping the level of overbank flooding the same over time. This prevents costly damage or maintenance for culverts, drainage structures, and swales.

Overbank floods are ranked in terms of their statistical return frequency. For example, a flood that has a 50% chance of occurring in any given year is termed a “two year” flood. In Vermont, about 2.1 to 2.9 inches of rain in a 24-hour period produces a two-year flood.

Similarly, a flood that has a 10% chance of occurring in any given year is termed a “ten-year flood." A ten-year flood occurs when a storm event produces between 3.0 and 4.0 inches of rain in a 24 hour period. Under traditional engineering practice, most channels and storm drains in Vermont are designed with enough capacity to safely pass the peak discharge from the ten-year design storm.

Development increases the peak discharge rate associated with a given design storm, because impervious surfaces generate greater runoff volumes and drainage systems deliver it more rapidly to a stream. The change in post-development peak discharge rates that accompany development is conceptually illustrated in Figure 5.

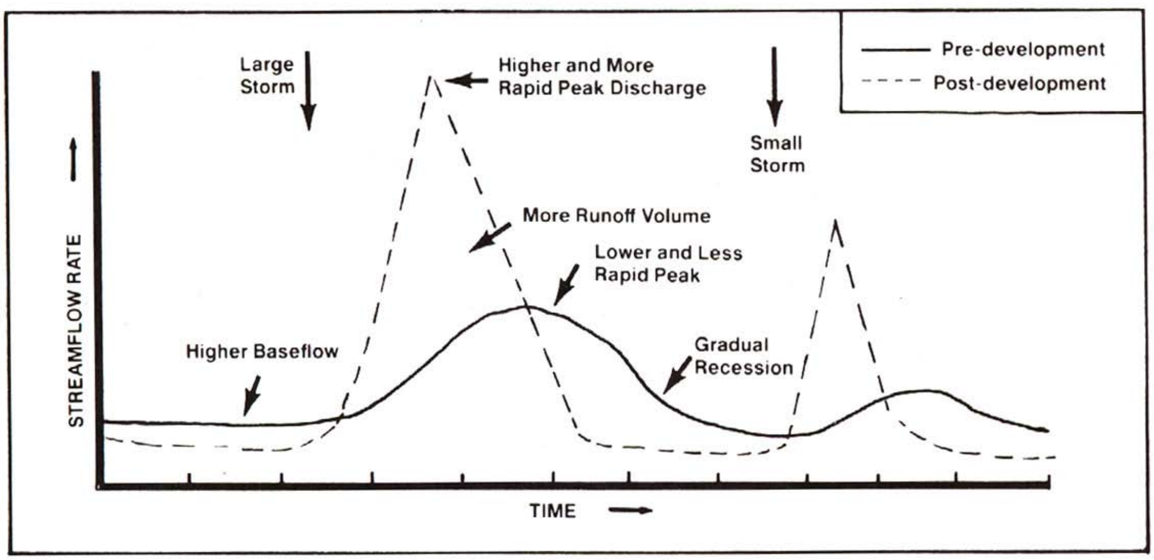

Figure 5. Hydrographs Before and After Development

5.5. Floodplain Expansion

The level areas bordering streams and rivers are known as floodplains. Operationally, the floodplain is usually defined as the land area within the limits of the 100-year storm water elevation. The 100-year storm has a 1% chance of occurring in any given year. In Vermont, a 100-year flood occurs after between 5 and 8 inches of rainfall in a 24-hour period. These floods can be very destructive, and can pose a threat to property and human life.

As with overbank floods, development increases the peak discharge rate associated with the 100-year design storm. As a consequence, the elevation of a stream’s 100-year floodplain becomes higher and the boundaries of its floodplain expand. In some instances, property and structures that had not previously been subject to flooding are now at risk. Additionally, such a shift in a floodplain’s hydrology can degrade wetland and forest habitats.

5.6. Impacts to Aquatic Organisms

The decline in the physical habitat of the stream, coupled with lower base flows and higher stormwater pollutant loads, has a severe impact on the aquatic community. Research suggests that new development impacts aquatic insets, fish, and amphibians at fairly low levels of imperviousness, usually around 10% imperviousness (I) or less (Table 5). New development appears to cause declining **richness** (the number of different species in an area or community), **diversity** (number and relative frequency of different species in an area or community), and **abundance** (number of individuals in a species). This general relationship was also documented in Vermont where the benthic macroinvertebrate community health was related to impervious cover (CWP, 2000). In general, there appears to be a breakpoint at about 7-8% imperviousness where stream biological condition goes from good to fair or poor. This relationship was found to be consistent with the findings of other national studies (Maxted and Shaver, 1996 and Montgomery County DEP, 2000).

6. Integrated Stormwater Management

Integrated stormwater management design involves the integration of site design practices and procedures with the design and layout of stormwater infrastructure to attain stormwater quality and quantity management goals.

The integrated stormwater management concept uses the following elements or steps:

1. **Better Site Design Practices and Techniques** – Designs oriented to utilize natural features of the site to reduce runoff and pollutants.

2. **Unified Design Criteria for Stormwater Control Requirements** – An approach that utilizes the volume of runoff for water quality, channel protection, overbank flood protection, and extreme flood protection management goals.

3. **Downstream Assessment** – A computational approach that ensures that the proposed development is not adversely impacting downstream properties after the volumes calculated above have been controlled.

4. **Stormwater Credits for Site Design** – A methodology that utilizes the principles of better site layout and design to apply stormwater management *design credits* to the unified design criteria calculations to reduce the overall stormwater runoff volume that needs to be controlled.

5. **Selection of Structural Stormwater Controls** – A methodology of selecting structural control measures using a screening process to choose the most appropriate practice or practices for a given site and watershed conditions.

Table 5. Recent Research Examining the Relationship of Urbanization to Aquatic Habitat and Organisms

Watershed Indicator	Key Finding	Reference	Year	Location
Aquatic insects and fish	A comparison of three stream types found urban streams had lowest diversity and richness. Urban streams had substantially lower EPT scores (22% vs 5% as number of all taxa, 65% vs 10% as percent abundance) and IBI scores in the poor range.	Crawford & Lenat	1989	North Carolina
Insects, fish, habitat water quality,	Steepest decline of biological functioning after 6% imperviousness. There was a steady decline, with approx 50% of initial biotic integrity at 45% I.	Horner et al.	1996	Puget Sound Washington
Fish, Aquatic insects	A study of five urban streams found that as land use shifted from rural to urban, fish and macroinvertebrate diversity decreased.	Masterson and Bannerman	1994	Wisconsin
Insects, fish, habitat, water quality, riparian zone	Physical and biological stream indicators declined most rapidly during the initial phase of the urbanization process as the percentage of total impervious area exceeded the 5-10% range.	May et al.	1997	Washington
Aquatic insects and fish	There was significant decline in the diversity of aquatic insects and fish at 10% impervious cover.	MWCOG	1992	Washington, DC
Aquatic insects and fish	Evaluation of the effects of runoff in urban and non-urban areas found that native fish and insect species dominated the non-urban portion of the watershed, but native fish accounted for only 7% of the number of species found in urban areas.	Pitt	1995	California
Wetland plants, amphibians	Mean annual water fluctuation inversely correlated to plant & amphibian density in urban wetlands. Declines noted beyond 10% impervious area.	Taylor	1993	Seattle
Aquatic insects & fish	Residential urban land use in Cuyahoga watersheds created a significant drop in IBI scores at around 8%, primarily due to certain stressors that functioned to lower the non-attainment threshold When watersheds smaller than 100mi^2 were analyzed separately, the level of urban land use for a significant drop in IBI scores occurred at around 15%.	Yoder et. al.	1999	Ohio
Aquatic insects & fish	All 40 urban sites sampled had fair to very poor index of biotic integrity (IBI) scores, compared to undeveloped reference sites.	Yoder	1991	Ohio

IBI: Index of Biotic Integrity: A measure of species diversity for fish and macroinvertebrates
EPT: A measure of the richness of three sensitive macro-invertebrates (may flies, caddis flies, and stone flies), used to indicate the ability of a waterbody to support sensitive organisms.

The aim of this integrated approach is to provide a process that will address the comprehensive stormwater performance goals presented in Section 7 below, while at the same time providing ease of application for the land developer and a streamlined process for the review of a project by the State or local municipality. The integrated design process is illustrated in Figure 6. Each concept or aspect of this process will be described in the subsequent chapters. These steps are provided as guidance to

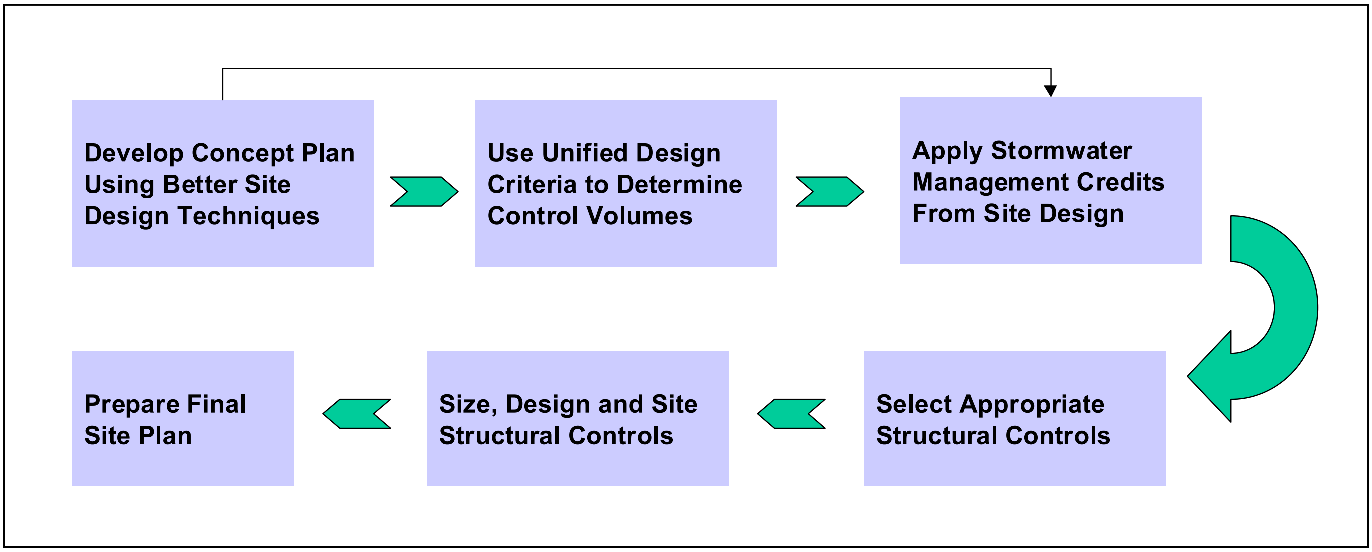

Figure 6. The Integrated Stormwater Management Site Design Process (Source: ARC, 2001)

developers, designers, and reviewers to assist in the often challenging endeavor of providing effective stormwater management at the site level.

The following guidelines should also be kept in mind in using this process and preparing a stormwater management plan for a development site:

- **Site design should utilize an integrated approach to deal with stormwater quantity, quality and streambank protection requirements.**

  The stormwater management infrastructure for a site should be designed to integrate drainage and water quantity control, water quality protection and downstream channel protection. Site design should be done in unison with the design and layout of stormwater infrastructure to attain stormwater management goals. Together, the combination of better site design practices[1] and effective infrastructure layout and design can mitigate most stormwater impacts of many developments while preserving stream integrity and aesthetic attractiveness.

- **Stormwater management practices should strive to utilize the natural drainage system and require as little maintenance as possible.**

  Almost all sites contain natural features that can be used to help manage and mitigate runoff from development. Features on a development site might include natural drainage

1. The reader is referred to the following two references for a more detailed presentation of better site design and low impact development: 1) Center for Watershed Protection. 1998. *Better Site Design A Handbook for Changing Development Rules in Your Community*. Ellicott City, MD; and 2) Prince George's County MD Dept. of Environmental Resources. 1999. *Low Impact Develoment Design Strategies: An Integrated Design Approach*. Largo, MD.

patterns, depressions, permeable soils, wetlands, floodplains and undisturbed vegetated areas that can be used to reduce runoff, provide infiltration and stormwater filtering of pollutants and sediment, recycle nutrients, and maximize on-site storage of stormwater. Site design should seek to improve the effectiveness of natural systems rather than to ignore or replace them. Further, natural systems typically require minimal maintenance, and will continue to function many years into the future.

- **Structural stormwater controls should be implemented in concert with conservation site design and nonstructural options.**

  Operationally, economically, and aesthetically, conservation site design and the use of natural techniques offer complementary benefits to structural stormwater controls. Therefore, all opportunities for utilizing these methods should be explored in conjunction with implementing structural stormwater controls such as engineered wet ponds and sand filters.

- **Structural stormwater solutions should attempt to be multi-purpose and be aesthetically integrated into a site's design.**

  A structural stormwater facility need not be an afterthought or ugly nuisance on a development site. A parking lot, soccer field or city plaza can serve as a temporary storage facility for stormwater. In addition, water features such as ponds and lakes when correctly designed and integrated into a site can increase the aesthetic value of a development.

- **"One size does not fit all" in terms of stormwater management solutions.**

  Although the basic impacts from stormwater runoff and the need for its management remain the same, each project and watershed presents different challenges and opportunities. For instance, an infill development in a highly urbanized town center or downtown area will require a much different set of stormwater management solutions than a low-density residential subdivision in a largely undeveloped watershed. Therefore, local stormwater management needs to take into account differences between development sites, different types of development and land use, various watershed conditions and priorities, the nature of downstream lands and waters, and community desires and preferences.

7. General Performance Goals for Stormwater Management

To minimize adverse impacts of stormwater runoff, the following performance goals are recommended for all new development sites that are subject to the stormwater regulation. The performance goals are intended to be a set of goals that the required criteria strive to achieve. They should be viewed as benchmarks for review of all projects that are covered by the regulation. Some of the goals are inherently qualitative such as minimizing runoff from a site and utilizing pervious areas for treatment (Goal No. 1). Others can be easily quantified (and are direct references to specific criteria in the Manual) such as peak discharge control of the 10-year return frequency event (Goal No. 6).

Goal No.1 Site designs must minimize stormwater runoff and utilize pervious areas for stormwater treatment.

Goal No. 2 Stormwater management should generally be provided through a combination of structural and non-structural practices. Where practical and feasible, non-structural practices should be incorporated into a site's design to reduce the reliance on structural practices.

Goal No. 3 Stormwater runoff generated from new development must be adequately detained and treated prior to discharging into a jurisdictional wetland or waters of the State of Vermont.

Goal No. 4 Annual groundwater recharge rates must be maintained, by promoting infiltration through the use of structural and non-structural methods.

Goal No. 5 For new development, structural stormwater treatment practices (STPs) must be designed to remove 80% of the average annual post development total suspended solids load (TSS) and 40% for total phosphorus (TP). It is presumed that a STP complies with this performance goal if it is:

1. sized to capture the prescribed water quality volume (WQ_v),
2. designed according to the specific performance criteria outlined in this Manual,
3. constructed properly, and
4. maintained regularly.

Goal No. 6 The post-development peak discharge rate must not exceed the pre-development peak discharge rate for the 10-year frequency storm event unless specifically exempted.[2]

Goal No. 7 To protect stream channels from degradation, channel protection volume (Cp_v) must be provided by means of 12 to 24 hours of extended detention storage for the one year storm event or by the Distributed Runoff Control (DRC) method as described in this Manual.

Goal No. 8 All STPs must have an enforceable operation and maintenance agreement to ensure the system functions as designed. In addition, every STP must have an acceptable form of water quality pretreatment[3].

Goal No. 9 Redevelopment and infill projects should maximize the treatment and

2. See Chapter 2 for the definition of Pre- and Post-development

3. Examples of adequate forms of pretreatment include, but are not limited to: forebays, stilling basins/sedimentation chambers, vegetated swales, and filter strips. Most pretreatment is incorporated into the design of each practice (e.g., sedimentation chambers and filters). Pretreatment extends the design life of the facility and makes maintenance operations (a key provision of system performance) easier and more cost effective over the long term. Guidance on appropriate pretreatment measures for specific stormwater treatment practices is provided in Chapter 3.

management of runoff from existing impervious surfaces.

Goal No. 10 Stormwater discharges from certain intensive land uses or activities with higher potential pollutant loadings may be required by ANR to use specific structural STPs and pollution prevention practices. Section 2.6 of the *Vermont Stormwater Management Manual-Volume I* identifies the types of land uses and activities that are defined as hotspots. Stormwater runoff from hotspots should not be allowed to infiltrate where it may contaminate water supplies.

Goal No. 11 To the maximum extent practical, surface discharges from stormwater management practices should be returned to the same drainage catchment or watershed that the majority of the runoff originated in.

Appendix A: Site Design & Landscaping Guidance

Appendix A1: Selecting the Most Effective Stormwater Treatment System

Selecting and Locating the Best Stormwater Treatment Practice (STP)

This section presents a series of matrices that can be used as a screening process for selecting the best STP or group of STPs for a development site. It also provides guidance for locating practices on the site. The matrices presented can be used to screen practices in a step-wise fashion. The six matrices presented here are not exhaustive. Specific additional criteria may be incorporated depending on local design knowledge and resource protection goals. Caveats for the application of each matrix are included in the detailed description of each. Screening factors include:

- Land Use
- Physical Feasibility
- Watershed Factors
- Stormwater Management Capability
- Pollutant Removal
- Community and Environmental Factors

Step 1. Land Use

Which practices are best suited for the proposed land use at this site? In this step, the designer makes an initial screen to select practices that are best suited to a particular land use.

Step 2. Physical Feasibility Factors

Are there any physical constraints at the project site that may restrict or preclude the use of a particular STP? In this step, the designer screens the STP list using Matrix No. 2 to determine if the soils, water table, drainage area, slope or head conditions present at a particular development site might limit the use of a STP.

Step 3. Watershed Factors

What watershed protection goals need to be met in the resource my site drains to? Matrix No.3 outlines STP goals and restrictions based on the nature of the receiving water.

Step 4. Stormwater Management Capability

Can one STP meet all design criteria, or is a combination of practices needed? In this step, designers can screen the STP list using Matrix No. 4 to determine if a particular STP can meet recharge, water quality, channel protection, and flood control storage requirements. At the end of this step, the designer can screen the STP options down to a manageable number and determine if a single STP or a group of STPs are needed to meet stormwater sizing criteria at the site.

Step 5. Pollutant Removal

How do each of the STP options compare in terms of pollutant removal? In this step, the designer views removal of select pollutants to determine the best STP options for addressing specific water quality constraints of a given watershed or receiving water body.

Step 6. Community and Environmental Factors

Do the remaining STPs have any important community or environmental benefits or drawbacks that might influence the selection process? In this step, a matrix is used to compare the STP options with regard to cold climate limitations, maintenance, habitat, community acceptance, cost and other environmental factors.

More detail on the proposed step-wise screening process is provided below:

A1.1. Land Use

This matrix (Table A.1) allows the designer to make an initial screen of practices most appropriate for a given land use.

Rural. This column identifies STPs that are best suited to treat runoff in rural or very low density areas (e.g., typically at a density of less than one dwelling unit per acre).

Residential. This column identifies the best treatment options in medium to high density residential developments.

Roads and Highways. This column identifies the best practices to treat runoff from major roadway and highway systems.

Commercial Development. This column identifies practices that are suitable for new commercial development.

Hotspot Land Uses. This column examines the capability of STPs to treat runoff from designated hotspots. STPs that receive hotspot runoff may have design restrictions, as noted.

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

Table A.1. Land Use Matrix

STP Group	STP Design	Rural	Residential	Roads and Highways	Commercial / High Density	Hotspots	Ultra Urban
Pond	Micropool ED	○	○	○	◗	①	●
	Wet Pond	○	○	○	◗	①	●
	Wet ED Pond	○	○	○	◗	①	●
	Multiple Pond	○	○	◗	◗	①	●
	Pocket Pond	○	◗	○	◗	●	●
Wetland	Shallow Marsh	○	○	◗	◗	①	●
	ED Wetland	○	○	◗	◗	①	●
	Pond/Wetland	○	○	◗	◗	①	●
	Gravel Wetland	○	◗	○	○	①	●
Infiltration	Infiltration Trench	◗	○	○	○	●	◗
	Shallow I-Basin	◗	○	◗	◗	●	◗
Filters	Surface Sand Filter	●	◗	○	○	②	○
	Underground SF	●	●	◗	○	○	○
	Perimeter SF	●	●	◗	○	○	○
	Organic SF	●	◗	○	○	②	○
	Bioretention	○	○	○	○	②	○
Open Channels	Dry Swale	○	◗	○	◗	②	◗
	Wet Swale	○	●	○	●	●	●
	Grass Channel	○	◗	○	◗	②	◗
Detention*	Pond/Vault	○	○	○	○	①	●

○: Yes. Good option in most cases.
◗: Depends. Suitable under certain conditions, or may be used to treat a portion of the site.
●: No. Seldom or never suitable.
①: Acceptable option, but may require a pond liner to reduce risk of groundwater contamination.
②: Acceptable option, if not designed as an exfilter. (An exfilter is a conventional stormwater filter without an underdrain system. The filtered volume ultimately infiltrates into the underlying soils.)
*: The pond/vault is not an acceptable stand-alone water quality STP.

A1.2. Physical Feasibility Factors

This matrix (Table A.2.) allows the designer to evaluate possible options based on physical conditions at the site. More detailed testing protocols are often needed to confirm physical conditions at the site. Therefore, the physical feasibility factors should be viewed as an initial guide and not necessarily as absolute prohibitions against the application of a particular practice or set of practices. Five primary factors are:

Soils. The key evaluation factors are based on an initial investigation of the NRCS hydrologic soil groups at the site. Note that more detailed geotechnical tests are usually required for infiltration feasibility and during design to confirm permeability and other factors (see Section 2 of the *VT Stormwater Management Manual – Volume I* and Appendix D1 of this manual for specific feasibility factors).

Water Table. This column indicates the minimum depth to the seasonally high water table from the bottom elevation, or floor, of an STP.

Drainage Area. This column indicates the minimum or maximum drainage area that is considered optimal for a practice. If the drainage area to a facility is slightly greater than the maximum allowable drainage area for a practice, some leeway is warranted where a practice meets other management objectives. Likewise, the minimum drainage areas indicated for ponds and wetlands should not be considered inflexible limits, and may be increased or decreased depending on water availability (baseflow or groundwater), mechanisms employed to prevent clogging, or the ability to assume an increased maintenance burden.

Slope. This column evaluates the effect of slope on the practice. Specifically, the slope guidance refers to how flat the area where the practice is installed must be and/or how steep the contributing drainage area or flow length can be.

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

Table A.2. Physical Feasibility Matrix

STP GROUP	STP DESIGN	SOILS	WATER TABLE	DRAINAGE AREA (AC)	SITE SLOPE	HEAD (FT)
Pond	Micropool ED	HSG A soils may require pond liner	3 foot* separation if hotspot or aquifer	10 min**	Generally no more than 15%	6 to 8 ft
	Wet Pond			25 min**		
	Wet ED Pond					
	Multiple Pond					
	Pocket Pond	OK	below WT	5 max***		4 to 6 ft
Wetland	Shallow Marsh	HSG A soils may require liner	3 foot* separation if hotspot or aquifer	25 min	Generally no more than 8%	3 to 5 ft
	ED Wetland					
	Pond/Wetland					
	Gravel Wetland	OK	below WT	5 max		2 to 3 ft
Infiltration	Infiltration Trench	f_c > 0.5* inch/hr	3 feet*	5 max	Generally no more than 6%	~1 ft
	Shallow I-Basin			10 max		3 to 5 ft
Filters	Surface Sand Filter	OK	2 feet****	10 max ***	Generally no more than 6%	~5 ft
	Underground SF			2 max ***		5 to 7ft
	Perimeter SF			2 max ***		2 to 3 ft
	Organic SF			10 max***		2 to 4 ft
	Bioretention	Made Soil		5 max***		~5 ft
Open Channels	Dry Swale	Made Soil	2 feet	5 max	No more than 4%	3 to 5 ft
	Wet Swale	OK	below WT	5 max		~1 ft
	Grass Channel	OK	2 feet	5 max	No more than 4%	~1 ft
Detention	Pond/Vault	OK	2 ft* if hotspot or aquifer	OK	Generally no more than 15%	6 to 8 ft

Notes: OK= not restricted, WT= water table, f_c =soil permeability
* denotes a required limit, other elements are planning level guidance and may vary somewhat depending on site conditions ** unless adequate water balance and anti-clogging device installed *** drainage area can be larger in some instances. **** may be less or actually intercept the water table if an adequate liner is provided

A1.3. Watershed Factors

The design of STPs is fundamentally influenced by the nature of the downstream water body that will be receiving the stormwater discharge. Consequently, the designer needs to be cognizant of the goals in the water resource the site drains to. This matrix (Table A.3) includes selection criteria and design guidelines based on the major water quality designations in Vermont. These designations include:

A(1) Ecological Waters -- These are managed to achieve and maintain waters in a natural condition.

A(2) Public Water Supplies --These are managed for public water supply purposes to achieve and maintain waters with a uniformly excellent character and quality.

B WMT 1 Waters – These water are managed so as to have essentially a minimal change from the reference condition consistent with the full support of all aquatic biota and wildlife uses.

B WMT 2 Waters – These are managed so as to have a minor difference from the reference condition consistent with the full support of all aquatic biota and wildlife uses.

B WMT 3 Waters – These are managed so as to have a moderate difference from the reference condition. These waters may be subject to hydrological modifications due to water supply reservoirs or wastewater treatment plants.

The five water quality designations were consolidated into three groups to reflect common management objectives. For the purpose of this matrix all State waters currently listed as " Class B Waters" are considered to fall into the B2 designation.

Table A.3. Watershed Factors Matrix

STP GROUP	(A1, B1, A2)	(B2 or Current B)	(B3)
Ponds	Generally utilize the DRC approach for channel protection. Restrict in-stream practices. Limit use in cold water streams if ANR considers thermal influence to be significant.	Generally utilize the 1- yr, 24-hr ED for channel protection.	Channel protection potentially relaxed with basin plans. Where possible, integrate design with watershed retrofit priorities. Emphasize flood control when local flooding is a concern. Provide long detention times for bacteria control.
Wetlands	Generally utilize the DRC approach for channel protection. Restrict in-stream practices. Limit use in cold water streams if ANR considers thermal influence to be significant.	Generally utilize the 1- yr, 24-hr ED for channel protection.	Channel protection potentially relaxed with basin plans. Where possible, integrate design with watershed retrofit priorities. Design wetland practices to support habitat restoration goals. Emphasize flood control when local flooding is a concern. Consider the gravel wetland in phosphorus-limited watersheds.
Infiltration	Strongly encourage use for groundwater recharge. Combine with a detention facility to provide channel protection generally using the DRC.	Combine with a detention facility to provide channel protection (1- yr, 24-hr ED).	Combine with a detention facility to provide channel protection and flood control where appropriate. Direct infiltration of hotspot runoff is prohibited.
Filtering Systems	Combine with a detention facility to provide channel protection generally using the DRC.	Combine with a detention facility to provide channel protection (1- yr, 24-hr ED).	Combine with a detention facility to provide channel protection and flood control where appropriate. Use as pretreatment prior to an infiltration practice for hotspot runoff.
Open Channels	Combine with a detention facility to provide channel protection generally using the DRC.	Combine with a detention facility to provide channel protection (1- yr, 24-hr ED).	Combine with a detention facility to provide channel protection and flood control where appropriate. Can be restricted due to space limitations.
Detention*	Generally utilize the DRC approach for channel protection. Restrict in-stream practices.	Generally utilize the 1- yr, 24-hr ED for channel protection.	Channel protection potentially relaxed with basin plans. Where possible, integrate design with watershed retrofit priorities. Emphasize flood control when local flooding is a concern.

Notes: For all stream systems, removal of specific pollutants may also be a goal, particularly when a stream does not meet water quality standards, is part of a TMDL watershed, or drains to a waterbody that has specific pollutant reduction targets.
Cold water stream designations are identified in the Vermont Water Quality Standards, Appendix A.
*detention facilities are not acceptable stand-alone water quality practices.

A1.4. Stormwater Management Capability

This matrix (Table A.4) examines the capability of each STP option to meet stormwater management criteria. It shows whether an STP can meet requirements for:

Water Quality. The matrix tells whether each practice can be used to provide water quality treatment effectively as a stand-alone practice. For more detail, consult the pollutant removal matrix.

Recharge. The matrix indicates whether each practice can provide groundwater recharge, in support of recharge requirements by the Percent Volume Method. Note that it may also be possible to meet this requirement using stormwater credits (see Section 3 of the *VT Stormwater Management Manual – Volume I*).

Channel Protection. The matrix indicates whether the STP can typically provide channel protection storage. The designation that a particular STP cannot meet the channel protection requirement does not necessarily imply that the STP should be eliminated from consideration, but is a reminder that more than one practice may be needed at a site (e.g., a bioretention area and a downstream ED pond).

Quantity Control. The matrix shows whether an STP can typically meet the overbank and extreme event flooding criteria for the site. Again, the designation that a particular STP cannot meet these requirements does not necessarily mean that it should be eliminated from consideration, but rather is a reminder that more than one practice may be needed at a site (e.g., a bioretention area and a downstream stormwater detention pond)

Table A.4. STP Selection: Stormwater Management Capability Matrix

STP GROUP	STP DESIGN	WATER QUALITY?	RECHARGE?	CHANNEL PROTECTION?	FLOOD CONTROL?***
Pond	Micropool ED	○	●**	○	○
	Wet Pond	○	●**	○	○
	Wet ED Pond	○	●**	○	○
	Multiple Pond	○	●**	○	○
	Pocket Pond	○	❶	○	○
Wetland	Shallow Marsh	○	●**	○	○
	ED Wetland	○	●**	○	○
	Pond/Wetland	○	●**	○	○
	Gravel Wetland	○	●**	❸	❸
Infiltration	Infiltration Trench	○	○	❸	❹
	Shallow I-Basin	○	○	❸	❹
Filters	Surface SF	○	❷	❸	●
	Underground SF	○	●	●	●
	Perimeter SF	○	●	●	●
	Organic SF	○	❷	❸	●
	Bioretention	○	❷	❸	●
Open Channels	Dry Swale	○	❷	●	●
	Wet Swale	○	●	●	●
	Grass Channel	○	○	●	●
Detention*	Pond/Vault	●	●**	○	○

○ Practice generally meets this stormwater management goal.
● Practice can almost never be used to meet this goal.
❶ Since intercepting groundwater, side slopes contribute.
❷ Provides recharge only if designed as an exfilter system (i.e., native soils have adequate permeability to allow downward movement of filtered or pretreated water through the soil).
❸ Practice may partially meet this goal, or under specific site and design conditions.
❹ Can be used to meet flood control in rare conditions, with highly permeable soils.
* Practice is not an acceptable water quality practice.
** Practices are not acceptable for meeting the Re_v by the Percent Volume Method.
*** Includes both Q_{p10} and Q_{p100}.

A1.5. Pollutant Removal

This matrix (Table A.5) examines the capability of each STP option to remove specific pollutants from stormwater runoff. The matrix includes data for:

- Total Suspended Solids
- Total Phosphorous
- Total Nitrogen
- Metals
- Bacteria
- Hydrocarbons

Table A.5. STP Selection: Pollutant Removal Matrix

Practice	TSS [%]	TP [%]	TN [%]	Metals[1] [%]	Bacteria [%]	Hydrocarbons [%]
Wet Ponds	80	51	33	62	70	81[2]
Stormwater Wetlands	76	49	30	42	78[2]	85[2]
Filtering Practices	86	59	38	69	37[2]	84[2]
Infiltration Practices[3]	95[2]	80	51	99[2]	N/A	N/A
Open Channels[4]	81	34	84[2]	70	N/A	62[2]
Quantity Control Ponds[2, 5]	3	19	5	7.5	78	N/A

1. Average of zinc and copper. Only zinc for infiltration
2. Based on fewer than five data points (i.e., independent monitoring studies)
3. Includes porous pavement, which is not on the list of approved practices for Vermont. At this time, there are no known field studies that have measured sediment removal in infiltration trenches. However, it can logically be presumed that a properly operating infiltration trench will remove nearly 100% of the TSS load associated with the design treatment volume.
4. Higher removal rates for dry swales.
5. Quantity control ponds (a.k.a. dry detention basins or vaults) do not meet the WQ_v requirement and must be used in conjunction with acceptable water quality STPs.

N/A: Data not available

Removals represent median values from Winer (2000)

A1.6. Community and Environmental Factors

The last step assesses community and environmental factors involved in STP selection. This matrix (Table A.6) employs a comparative index approach. An open circle indicates that the STP has a high benefit (or low limitations for cold climate and cold water fish habitat factors) and a dark circle indicates that the particular STP has a low benefit (or high limitations for cold climate and cold water fish habitat factors).

Cold Climate Limitations. This column assesses the relative limitations that each STP may have with respect to its ability perform and be maintained under extreme and prolonged cold climate conditions[1].

Maintenance. This column assesses the relative maintenance effort needed for an STP, in terms of three criteria: frequency of scheduled maintenance, chronic maintenance problems (such as clogging) and reported failure rates. It should be noted that all STPs require routine inspection and maintenance.

Cold Water Fish Habitat Limitations. This column assesses the relative limitations that each STP may have with respect to providing the necessary protection to Cold Water Fish Habitat waters against thermal influences. Where high limitations are identified, a site specific assessment may be warranted by ANR to determine whether or not a specific practice is appropriate. Specific design considerations to minimize the potential of thermal impact for some STPs are provided in Section 2 of the *VT Stormwater Management manual – Volume I*.

Community Acceptance. This column assesses community acceptance, as measured by three factors: market and preference surveys, reported nuisance problems, and visual orientation (i.e., is it prominently located or is it in a discrete underground location). It should be noted that a low rank can often be improved by a better landscaping plan.

Affordability. The STPs are ranked according to their relative capital construction cost per impervious acre treated. These costs exclude design, land acquisition, and other individual costs.

Safety. A comparative index that expresses the relative public safety of an STP. An open circle indicates a reasonably safe STP, while a darkened circle indicates deep pools may create potential public safety risks. The safety factor is included at this stage of the screening process because liability and safety are of paramount concern in many residential settings.

Habitat. STPs are evaluated on their ability to provide wildlife or wetland habitat, assuming that an effort is made to landscape them appropriately. Objective criteria include size, water features, wetland features and vegetative cover of the STP and its buffer.

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1 See Appendix D10 of this manual and Section 2 of the *Vermont Stormwater Treatment Standards Volume I* for specific cold climate guidance with respect to practice selection, design, and maintenance.

Table A.6. STP Selection: Community and Environmental Factors Matrix

STP GROUP	STP LIST	COLD CLIMATE LIMITATIONS	EASE OF MAINTENANCE	COLD WATER FISH HABITAT LIMITATIONS	AFFORDABILITY	COMMUNITY ACCEPTANCE	SAFETY	HABITAT
Ponds	Micropool ED	○	◗	◗	○	◗	◗	◗
	Wet Pond	◗	○	●	○	○	●	○
	Wet ED Pond	○	○	●	○	○	●	○
	Multiple Pond	◗	○	●	◗	○	●	○
	Pocket Pond	◗	●	◗	○	◗	◗	●
Wetlands	Shallow Marsh	◗	◗	●	◗	○	◗	○
	ED Wetland	○	◗	●	◗	◗	◗	○
	Pond/Wetland	◗	○	●	◗	○	●	○
	Gravel Wetland	●	●	◗	●	◗	○	◗
Infiltration	Infiltration Trench	●	●	○	◗	○	○	●
	Shallow I-Basin	◗	●	○	◗	●	○	●
Filters	Surface SF	●	◗	○	●	◗	○	●
	Underground SF	○	●	○	●	○	◗	●
	Perimeter SF	●	●	○	●	○	○	●
	Organic SF	●	◗	○	●	◗	○	●
	Bioretention	◗	◗	○	◗	○	○	◗
Open Channels	Dry Swale	◗	◗	○	◗	○	○	●
	Wet Swale	◗	○	○	○	◗	○	◗
	Grass Channel	○	○	○	○	○	○	●
Detention	Pond/Vault	○	○	○	○	◗	◗	●

○ High Benefit and/or Low Limitations
● Low Benefit and/or High Limitations
◗ Medium Benefit and/or Limitations

Appendix A2: Landscaping Guidance / Plant Lists

A2.1. General Landscaping Guidance for All STPs

- Do not plant trees and shrubs within 15 feet of the toe of slope of a dam.
- Do not plant trees or shrubs known to have long tap roots within the vicinity of the earth dam or subsurface drainage facilities.
- Do not plant trees and shrubs within 15 feet of perforated pipes.
- Do not plant trees and shrubs within 25 feet of a riser structure.
- Provide 15-foot clearance from a non-clogging, low flow orifice.
- Herbaceous embankment plantings should be limited to 10 inches in height, to allow visibility for the inspector who is looking for burrowing rodents that may compromise the integrity of the embankment.
- Provide slope stabilization methods for slopes steeper than 2:1, such as planted erosion control mats. Also, use seed mixes with quick germination rates in this area. Augment temporary seeding measures with container crowns or root mats of more permanent plant material.
- Utilize erosion control mats and fabrics to protect channels that are subject to frequent washouts.
- Stabilize all water overflows with plant material that can withstand strong current flows. Root material should be fibrous and substantial but lacking a tap root.
- Sod channels that are not stabilized by erosion control mats.
- Divert flows temporarily from seeded areas until stabilized.
- Check water tolerances of existing plant materials prior to inundation of area.
- Stabilize aquatic and safety benches with emergent wetland plants and wet seed mixes.
- Do not block maintenance access to structures with trees or shrubs.
- To reduce thermal warming, when possible shade inflow and outflow channels as well as the southern exposures of pond.
- Avoid plantings that will require routine or intensive chemical applications (i.e. turf area).
- Have soil tested to determine if there is a need for amendments.
- Select plants that can thrive with on-site soil with no additional amendments or a minimum of amendments.
- Avoid use of any plants included on ANR's Invasive Exotic Plants of Vermont List and the Agricultural Department's proposed Noxious Weed Quarantine List.
- Decrease the areas where turf is used. Use low maintenance ground cover to absorb run-off.
- When planting a mix of plant species, plant individual of same species in clumps (e.g., groups of three to five) rather than alternating species on a plant by plant basis.

- Plant stream and edge of water buffers with trees, shrubs, ornamental grasses, and herbaceous materials where possible, to stabilize banks and provide shade.
- Maintain and frame desirable views. Be careful not to block views at entrances, exits, or difficult road curves. Screen or buffer unattractive views into the site.
- Use plants to prohibit pedestrian access to pools or slopes that may be unsafe.
- The designer should carefully consider the long term vegetation management strategy for the BMP, keeping in mind the “maintenance” legacy for the future owners. Keep maintenance area open to allow future access for pond maintenance. Provide a planting surface that can withstand the compaction of vehicles using maintenance access roads. Make sure the facility maintenance agreement includes a maintenance requirement of designed plant material.
- Select salt tolerant plant material in areas that may receive wintertime salt applications (roads and parking lots).
- Provide signage for:
  - Stormwater Management Areas to help educate the public when possible.
  - Wildflower areas, when possible, to designate limits of mowing.
- Avoid the overuse of any plant materials.
- Preserve existing natural vegetation when possible.

It is often necessary to test the soil in which you are about to plant in order to determine the following:

- pH; whether acid, neutral, or alkali
- major soil nutrients; Nitrogen, Phosphorus, Potassium
- minerals; such as chelated iron, lime

Have soil samples analyzed by experienced and qualified individuals, such as those at the Agricultural Extension Office, who will explain in writing the results, what they mean, as well as what soil amendments would be required. Certain soil conditions can present serious constraints to the growth of plant materials and may require the involvement of qualified professionals. When poor soils can’t be amended, seed mixes and plant material must be selected to establish ground cover as quickly as possible.

Areas that have recently been involved in construction can become compacted so that plant roots cannot penetrate the soil. The result is that often seeds lie on the surface of compacted soils, allowing them to be washed away or be eaten by birds. Instead, soils should be loosened to a minimum depth of two inches, preferably to a four-inch depth. Hard soils may require discing to a deeper depth. The soil should be loosened regardless of the ground cover. This will improve seed contact with the soil, providing greater germination rates, allowing the roots to penetrate into the soil. If the area is to be sodded; discing will allow the roots to penetrate into the soil. Weak or patchy crops can be prevented by providing good growing conditions.

Whenever possible, topsoil should be spread to a depth of four inches (two inch minimum) over the entire area to be planted. This provides organic matter and important nutrients for the plant material. This also allows the stabilizing materials to become established faster, while the roots are able to penetrate deeper and stabilize the soil, making it less likely that the plants will wash out during a heavy storm.

If topsoil has been stockpiled in deep mounds for a long period of time, it is desirable to test the soil for pH as well as microbial activity. If the microbial activity has been destroyed, it is necessary to inoculate the soil after application.

Remember that newly installed plant material requires water in order to recover from the shock of being transplanted. Be sure that some source of water is provided, should dry periods occur after the initial planting. This will reduce plant loss and provide the new plant materials with a chance to establish root growth.

A2.2. Ponds and Wetlands

For areas that are to be planted within a stormwater management facility it is necessary to determine what type of hydrologic zones will be created within the facility. The six zones presented in Table A.7 describe the different conditions encountered in stormwater management facilities. Every facility does not necessarily reflect all of these zones. The hydrologic zones designate the degree of tolerance the plant exhibits to differing degrees of inundation by water.

Table A.7. Hydrologic Zones

Zone #	Zone Description	Hydrologic Conditions
Zone 1	Deep Water Pool	1-6 feet deep Permanent Pool
Zone 2	Shallow Water Bench	6 inches to 1 foot deep
Zone 3	Shoreline Fringe	Regularly inundated
Zone 4	Riparian Fringe	Periodically inundated
Zone 5	Floodplain Terrace	Infrequently inundated
Zone 6	Upland Slopes	Seldom or never inundated

Each zone has its own set of plant selection criteria based on the hydrology of the zone, the stormwater functions required of the plant and the desired landscape effect. The hydrologic zones are as follows (see Figure A.1 for a schematic):

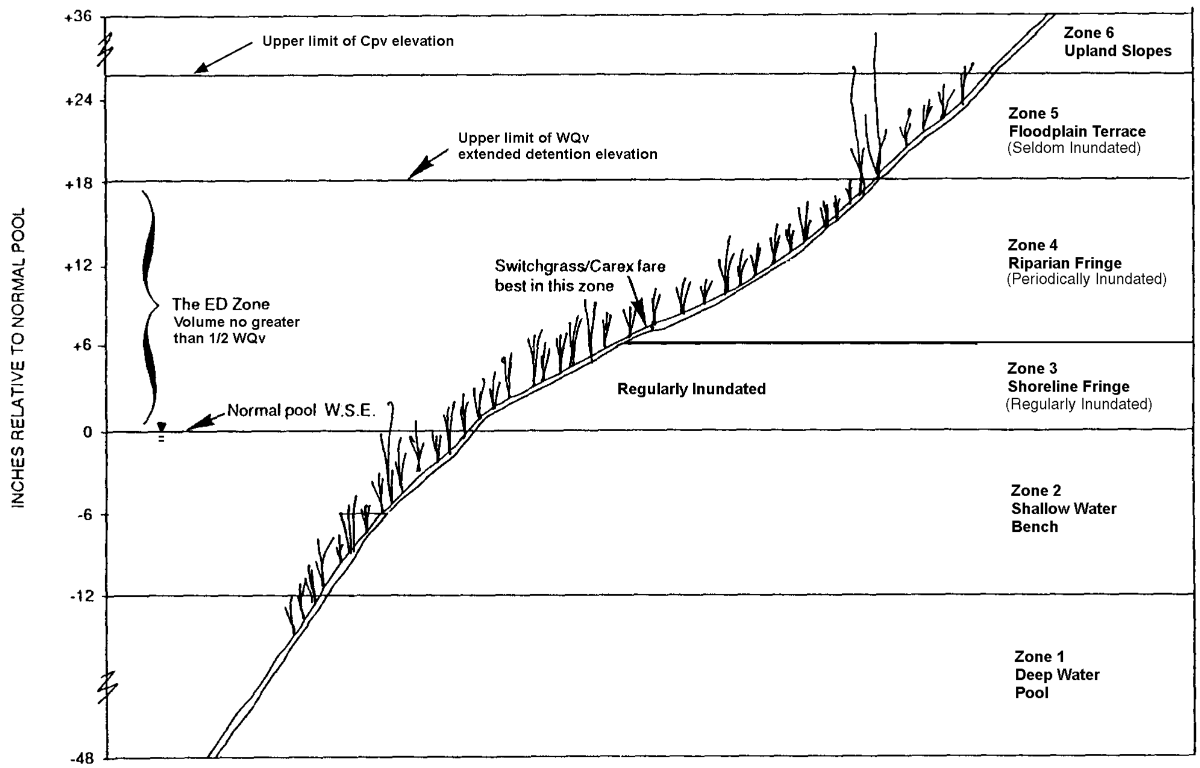

Figure A.1. Schematic of Pondscaping Zones

Zone 1: Deep Water Area (1- 6 Feet)

Ponds and wetlands both have deep pool areas that comprise Zone 1. These pools range from one to six feet in depth, and are best colonized by submergent plants, if at all.

This pondscaping zone has not been routinely planted for several reasons. First, the availability of plant materials that can survive and grow in this zone is limited, and it is also feared that plants could clog the stormwater facility outlet structure. In many cases, these plants will gradually become established through natural recolonization (e.g., transport of plant fragments from other ponds via the feet and legs of waterfowl). If submerged plant material becomes more commercially available and clogging concerns are addressed, this area can be planted. The function of the planting is to reduce resedimentation and improve oxidation while creating a greater aquatic habitat.

- Plant material must be able to withstand constant inundation of water of one foot or greater in depth.
- Plants may be submerged partially or entirely.
- Plants should be able to enhance pollutant uptake.
- Plants may provide food and cover for waterfowl, desirable insects, and other aquatic life.

Zone 2: Shallow Water Bench (Normal Pool To 1 Foot)

Zone 2 includes all areas that are inundated below the normal pool to a depth of one foot, and is the primary area where emergent plants will grow in a stormwater wetlands. Zone 2 also coincides with the aquatic bench found in stormwater ponds (Figure A.2.). This zone offers ideal conditions for the growth of many emergent wetland species. These areas may be located at the edge of the pond or on low mounds of earth located below the surface of the water within the pond. When planted, Zone 2 can be an important habitat for many aquatic and nonaquatic animals, creating a diverse food chain. This food chain includes predators, allowing a natural regulation of mosquito populations.

- Plant material must be able to withstand constant inundation of water to depths between six inches and one foot deep.
- Plants will be partially submerged.
- Plants should be able to enhance pollutant uptake.
- Plants may provide food and cover for waterfowl, desirable insects and other aquatic life.

Plants will stabilize the bottom of the pond, as well as the edge of the pond, absorbing wave impacts and reducing erosion, when water levels fluctuate. Plants also slow water velocities and increase sediment deposition rates. Plants can reduce resuspension of sediments caused by the wind. Plants can also soften the engineered contours of the pond, and can conceal drawdowns during dry weather.

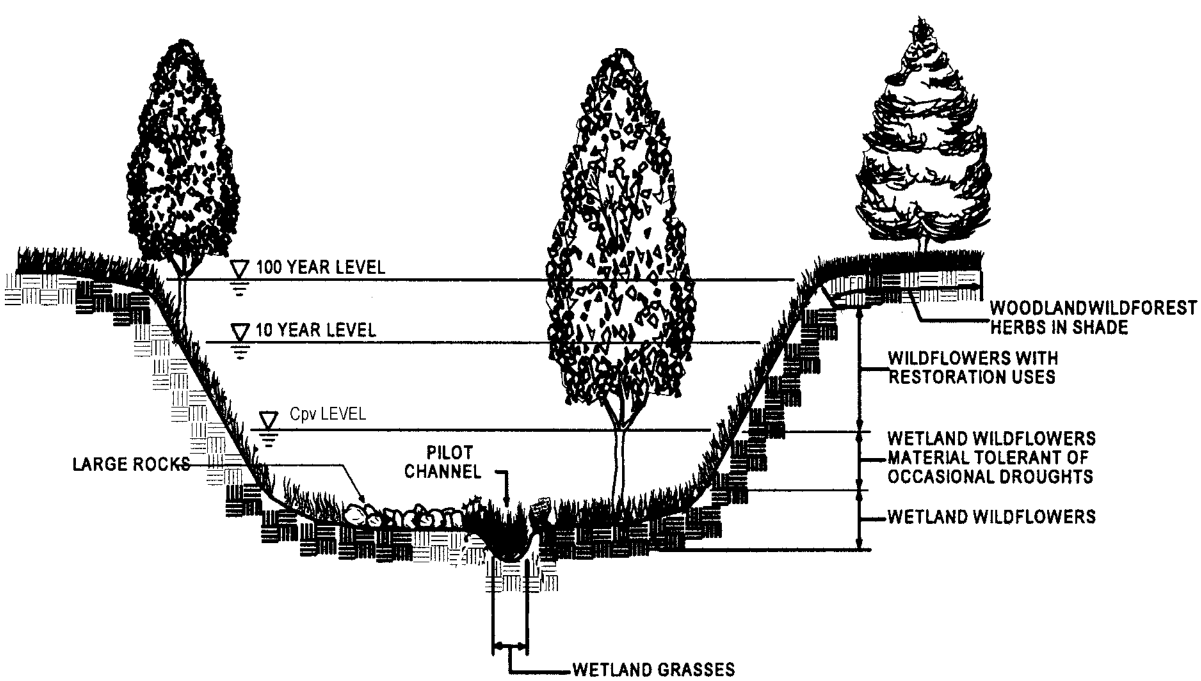

Figure A.2. Schematic Section of Typical Stormwater Management Detention Pond

Zone 3: Shoreline Fringe (Regularly Inundated)

Zone 3 encompasses the shoreline of a pond or wetland, and extends vertically about one foot in elevation from the normal pool. This zone includes the safety bench of a pond, and may also be periodically inundated if storm events are subject to extended detention. This zone occurs in a wet pond or shallow marsh (Figure A.3) and can be the most difficult to establish since plants must be able to withstand inundation of water during storms and prolonged drought conditions. In order to stabilize the soil in this zone, Zone 3 must have a vigorous cover. Planting a diverse mix of appropriate species is desirable, since hydrological conditions within this zone can be highly variable and hard to predict.

- Plants should stabilize the shoreline to minimize erosion caused by wave and wind action or water fluctuation.
- Plant material must be able to withstand occasional inundation of water. Plants will be partially submerged at this time.
- Plant material should, whenever possible, shade the shoreline, especially the southern exposure. This will help to reduce the water temperature.
- Plants should be able to enhance pollutant uptake.
- Plants may provide food and cover for waterfowl, songbirds, and wildlife. Plants can also be selected and located to control waterfowl.
- Plants should be located to reduce human access, where there are potential hazards, but should not block the maintenance access.
- Plants should have very low maintenance requirements, since they may be difficult or impossible to reach.
- Plants should be resistant to disease and other problems, which might require chemical applications.

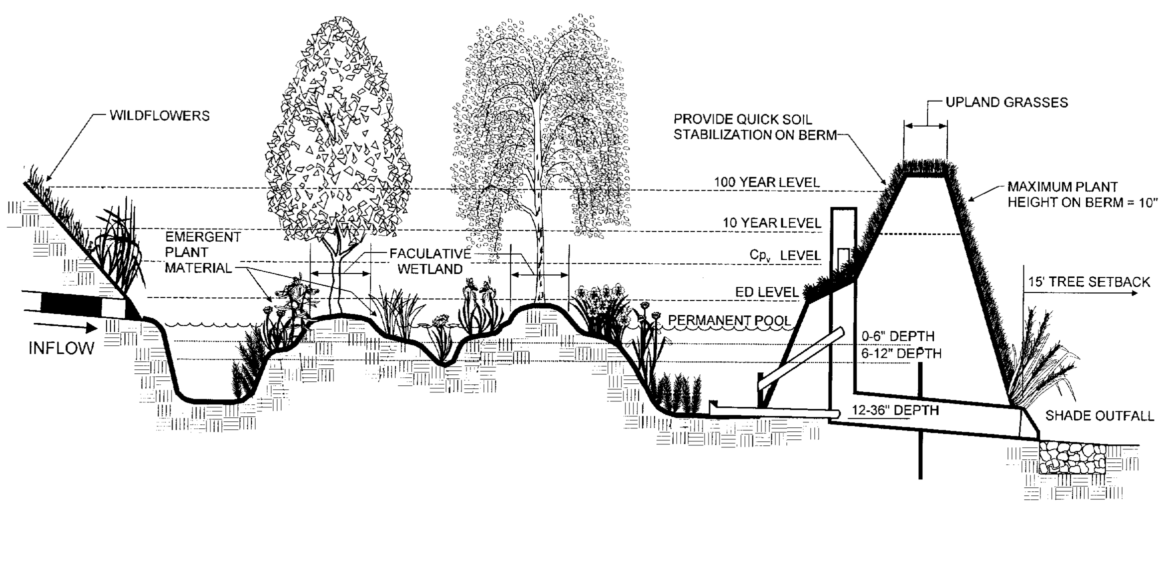

Figure A.3. Schematic Section of Shallow Marsh Wetland System

Zone 4: Riparian Fringe (Periodically Inundated)

Zone 4 extends from one to four feet in elevation above the normal pool. Plants in this zone are subject to periodic inundation during storms, and may experience saturated or partly saturated soils. Nearly all of the extended detention area is included within this zone.

- Plants must be able to withstand periodic inundation during storms, as well as occasional drought.
- Plants should stabilize the ground from erosion caused by run-off.
- Plants should be able to enhance pollutant uptake.
- Plant material should have very low maintenance, since they may be difficult or impossible to access.
- Plants may provide food and cover for waterfowl, songbirds and wildlife. Plants may also be selected and located to control waterfowl.
- Plants should be located to reduce pedestrian access to the deeper pools.

Zone 5: Floodplain Terrace (Infrequently Inundated)

Zone 5 is periodically inundated by flood waters that quickly recede in a day or less. Operationally, Zone 5 extends from the maximum Cp_v water surface elevation up to the 10 or 100 year maximum water surface elevation. Key landscaping objectives for Zone 5 are to stabilize the steep slopes characteristic of this zone, and establish a low maintenance, natural vegetation.

- Plant material should be able to withstand occasional but brief inundation during storms, although typical moisture conditions may be moist, slightly wet, or dry.
- Plants should stabilize the basin slopes from erosion.
- Ground cover should be very low maintenance, since they may be difficult to access on steep slopes or if frequency of mowing is limited. A dense tree cover may help reduce maintenance and discourage resident geese.
- Plants may provide food and cover for waterfowl, songbirds, and wildlife.
- Placement of plant material in Zone 5 is often critical, as it often creates a visual focal point and provides structure and shade for a greater variety of plants.

Zone 6: Upland Slopes (Seldom or Never Inundated)

The last zone extends above the maximum 100-year water surface elevation, and often includes the outer buffer of a pond or wetland. Unlike other zones, this upland area may have sidewalks, bike paths, retaining walls, and maintenance access roads. Care should be taken to locate plants so they will not overgrow these routes or create hiding places that might make the area unsafe.

- Plant material is capable of surviving the particular conditions of the site. Thus, it is not necessary to select plant material that will tolerate any inundation. Rather, plant selections should be made based on soil condition, light, and function within the landscape.

- Ground covers should emphasize infrequent mowing to reduce the cost of maintaining this landscape.
- Placement of plants in Zone 6 is important since they are often used to create a visual focal point, frame a desirable view, screen undesirable views, serve as a buffer, or provide shade to allow a greater variety of plant materials. Particular attention should be paid to seasonal color and texture of these plantings.

The plant list in Appendix A2.7. provides guidance on each plant's appropriate zones. The typical zones associated with each plant are shown in brackets "[ ]." In addition, there may be other zones listed outside of these brackets, which indicates that the plant materials may occur within these zones, but are not typically found in them.

A2.3. Infiltration and Sand Filters

Infiltration systems include Infiltration Trenches (I-1) and Infiltration Basins (I-2). Filter systems include sand and organic filters (F-1 and F-3). Properly planted, these systems blend into natural surroundings. If unplanted or improperly planted, they can become eyesores and liabilities.

Design Constraints:

- Do not plant trees or provide shade within 15 feet of infiltration or filtering area or where leaf litter will collect and clog infiltration area.
- Determine depth of water table to determine standing water conditions and depth to constant soil moisture.
- Planting turf over sand filters is allowed with prior approval of the reviewing public agency, on a case-by-case basis.
- Do not locate plants to block maintenance access to structures.
- Sod areas with heavy flows that are not stabilized with erosion control mats.
- Divert flows temporarily from seeded areas until stabilized.
- Planting of peat filters or any filter requiring a filter fabric should include material selected with care to insure that no taproots will penetrate the filter fabric.

A2.4. Bioretention

Planting Soil Bed Characteristics

The characteristics of the soil for the bioretention facility are perhaps as important as the facility location, size, and treatment volume. The soil must be permeable enough to allow runoff to filter through the media, while having characteristics suitable to promote and sustain a robust vegetative cover crop. In addition, much of the nutrient pollutant uptake (nitrogen and phosphorus) is accomplished through adsorption and microbial activity within the soil profile. Therefore, the soils must balance soil chemistry and physical properties to support biotic communities above and below ground.

The planting soil should be a sandy loam, loamy sand, loam, or a loam/sand mix (i.e., should contain a minimum 35 to 60% sand, by volume). The clay content for these soils should by less than 25% by volume (ETAB, 1993). Soils should fall within the SM, or ML classifications of the Unified Soil Classification System (USCS). A permeability of at least 1.0 foot per day (0.5"/hr) is required (a conservative value of 0.5 feet per day is used for design). The soil should be free of stones, stumps, roots, woody material over 1" in diameter, and brush or seeds from noxious weeds. Placement of the planting soil should be in lifts of 12" to 18", loosely compacted (tamped lightly with a dozer or backhoe bucket). The specific characteristics are presented in Table A.8.

Table A.8. Planting Soil Characteristics (Source MDE, 2000)

Parameter	Value
PH range	5.2 to 7.00
Organic matter	1.5 to 4.0%
Magnesium	35 lbs. per acre, minimum
Phosphorus (P_2O_5)	75 lbs. per acre, minimum
Potassium (K_2O)	85 lbs. per acre, minimum
Soluble salts	500 ppm
Clay	10 to 25%
Silt	30 to 55%
Sand	35 to 60%

Mulch Layer

The mulch layer plays an important role in the performance of the bioretention system. The mulch layer helps maintain soil moisture and avoids surface sealing, which reduces permeability. Mulch helps prevent erosion, and provides a micro-environment suitable for soil biota at the mulch/soil interface. It also serves as a pretreatment layer, trapping the finer sediments that remain suspended after the primary pretreatment.

The mulch layer should be standard landscape style, single or double, shredded hardwood mulch or chips. The mulch layer should be well aged (stockpiled or stored for at least 12 months), uniform in color, and free of other materials, such as weed seeds, soil, roots, etc. The mulch should be applied to a maximum depth of three inches. Grass clippings should not be used as a mulch material.

Planting Plan Guidance

Plant material selection should be based on the goal of simulating a terrestrial forested community of native species. Bioretention simulates an ecosystem consisting of an upland-oriented community dominated by trees, but having a distinct community, or sub-canopy, of understory trees, shrubs and herbaceous materials. The intent is to establish a diverse, dense plant cover to treat stormwater runoff and withstand urban stresses from insect and disease infestations, drought, temperature, wind, and exposure.

The proper selection and installation of plant materials is key to a successful system. There are essentially three zones within a bioretention facility (Figure A.4). The lowest elevation supports plant species adapted to standing and fluctuating water levels. The middle elevation supports a slightly drier group of plants, but still tolerates fluctuating water levels. The outer edge is the highest elevation and generally supports plants adapted to dryer conditions.

A sample of appropriate plant materials for bioretention facilities is included in Table A.9. For a more extensive bioretention plant list, consult ETAB, 1993 or Claytor and Schueler, 1997.

The layout of plant material should be flexible, but should follow the general principals described in Table A.10. The objective is to have a system which resembles a random and natural plant layout, while maintaining optimal conditions for plant establishment and growth.

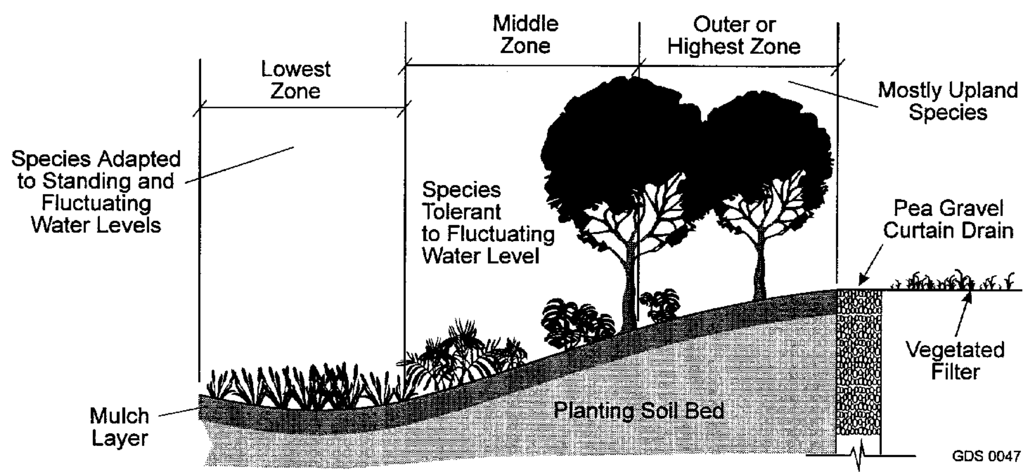

Figure A.4. Planting Zones for Bioretention Facilities

Table A.9. Native Plant Guide for Stormwater Bioretention Areas

Trees	Shrubs	Herbaceous Species
Acer rubrum Red Maple	*Hamemelis virginiana* Witch Hazel	*Iris versicolor* Blue Flag
Juniperus virginiata Eastern Red Cedar	*Ilex verticillata* Winterberry	*Lobelia cardinalis* Cardinal Flower
Platanus occidentalis Sycamore	*Viburnum dentatum* Arrowwood	*Rudbeckia laciniata* Cutleaf Coneflower
Salix nigra Black Willow	*Alnus serrulata* Brook-side Alder	*Scirpus cyperinus* Woolgrass
Pinus rigida Pitch Pine	*Cornus stolonifera* Redosier Dogwood	*Scirpus pungens* Three Square Bulrush
Note 1: For more options on plant selection for bioretention, consult Bioretention Manual (ETAB, 1993) or the Design of Stormwater Filtering Systems (Claytor and Schueler, 1996).		

Table A.10. Planting Plan Design Considerations

Native plant species should be specified over exotic or foreign species.
Appropriate vegetation should be selected based on the zone of hydric tolerance (see Figure A.4).
Species layout should generally be random and natural.
A canopy should be established with an understory of shrubs and herbaceous materials.
Woody vegetation should not be specified in the vicinity of inflow locations.
Trees should be planted primarily along the perimeter of the bioretention area.
Urban stressors (e.g., wind, sun, exposure, insect and disease infestation, drought) should be considered when laying out the planting plan.
Noxious weeds should not be specified (See ANR's Invasive Exotic Plants of Vermont List and the Agricultural Department's proposed Noxious Weed Quarantine List).
Aesthetics and visual characteristics should be a prime consideration.
Traffic and safety issues must be considered.
Existing and proposed utilities must be identified and considered.

Plant Material Guidance

Plant materials should conform to the American Standard Nursery Stock, published by the American Association of Nurserymen, and should be selected from certified, reputable nurseries. Planting specifications should be prepared by the designer and should include a sequence of construction, a description of the contractor's responsibilities, a planting schedule and installation specifications, initial maintenance, and a warranty period and expectations of plant survival. Table A.11 presents some typical issues for planting specifications.

Table A.11. Planting Specification Issues for Bioretention Areas

Specification Element	Elements
Sequence of Construction	Describe site preparation activities, soil amendments, etc.; address erosion and sediment control procedures; specify step-by-step procedure for plant installation through site clean-up.
Contractor's Responsibilities	Specify the contractors responsibilities, such as watering, care of plant material during transport, timeliness of installation, repairs due to vandalism, etc.
Planting Schedule and Specifications	Specify the materials to be installed, the type of materials (e.g., B&B, bare root, containerized); time of year of installations, sequence of installation of types of plants; fertilization, stabilization seeding, if required; watering and general care.
Maintenance	Specify inspection periods; mulching frequency (annual mulching is most common); removal and replacement of dead and diseased vegetation; treatment of diseased trees; removal of invasives; watering schedule after initial installation (once per day for 14 days is common); repair and replacement of staking and wires.
Warranty	Specify the warranty period, the required survival rate, and expected condition of plant species at the end of the warranty period.

A2.5. Open Channels

Consult Table A.12 for grass species that perform well in the stressful environment of an open channel.

Table A.12. Common Grass Species for Dry and Wet Swales & Grass Channels

Common Name	Scientific Name	Notes
Spreading Bentgrass	*Agrostis stolonifera*	Cool,
Red Fescue	*Festuca rubra*	Cool, not for wet swales
Bluejoint Reed grass	*Calamagrostis canadensis*	Cool, wet swales
Redtop	*Agrostis alba*	Cool
Notes: These grasses are sod-forming and can withstand frequent inundation, and are thus ideal for the swale or grass channel environment. Most are salt-tolerant, as well. Cool refers to cool season grasses. Where possible, one or more of these grasses should be in the seed mixes.		

A2.6. Other Considerations in Stormwater STP Landscaping

Use or Function

In selecting plants, consider their desired function in the landscape. Is the plant needed as ground cover, soil stabilizer, or a source of shade? Will the plant be placed to frame a view, create focus, or provide an accent? Does the location require that you provide seasonal interest to neighboring properties? Does the adjacent use provide conflicts or potential problems and require a barrier, screen, or buffer? Nearly every plant and plant location should be provided to serve some function in addition to any aesthetic appeal.

Plant Characteristics

Certain plant characteristics are so obvious, they may actually be overlooked in the plant selection. These are:

- Size
- Shape

For example, tree limbs, after several years, can grow into power lines. A wide growing shrub may block an important line of sight to oncoming vehicular traffic. A small tree could strategically block the a view from a second story window. Consider how these characteristics can work for you or against you, today and in the future.

Other plant characteristics must be considered to determine how the plant provides seasonal interest and whether the plant will fit with the landscape today and through the seasons and years to come. Some of these characteristics are:

- Color
- Texture
- Seasonal interest, i.e., flowers, fruit, leaves, stems/bark
- Growth rate

If shade is required in large amounts, quickly, a Planetree might be chosen over an Oak. In urban or suburban settings, a plant's seasonal interest may be of greater importance. Residents living next to a stormwater system may desire that the facility be appealing or interesting to look at throughout the year. Aesthetics is an important factor to consider in the design of these systems. Failure to consider the aesthetic appeal of a facility to the surrounding residents may result in reduced value to nearby lots. Careful attention to the design and planting of a facility can result in maintained or increased values of a property.

Availability and Cost

Often overlooked in plant selection is the availability from wholesalers and the cost of the plant material. There are many plants listed in landscape books that are not readily available from the nurseries. Without knowledge of what is available, time spent researching and finding the one plant that meets all the needs will be wasted. Some plants may require shipping, therefore, making it more costly than the budget may allow. Some planting requirements may require a special effort to find the specific plant that fulfills the needs of the site and the function of the plant in the landscape.

A2.7. Stormwater Plant List

The following pages present a detailed list of trees, shrubs, and herbaceous plants native to Vermont and suitable for planting in stormwater management facilities. The list is intended as a general guide for planning considerations. Local landscape architects/designers and nurseries may provide additional information for successful plant establishment.

The plant list is broken out into an herbaceous list and a woody list. Species are listed in alphabetical order, according to the common name. Scientific name and plant form (e.g., annual, perennial, grass, shrub, or tree) are also provided.

The recommended hydrologic zone(s) for each plant is provided to provide guidance on planting location. The most common zones are listed in brackets, “[ ]”, with additional zones listed indicating that a plant may survive over a range of hydrologic conditions.

A wetland indicator status is also listed to illustrate the likelihood of a species occurring in wetlands versus uplands (Reed, 1998). The indicator categories are defined as follows:

Obligate wetland (OBL): plants, which nearly always (more than 99% of the time) occur in wetlands under natural conditions.

Facultative wetland (FACW): plants, which usually (from 67% to 99% of the time) occur in wetlands, but occasionally found in nonwetlands.

Facultative (FAC): plants, which are equally likely to occur in wetlands and nonwetlands and are found in wetlands from 34% to 66% of the time.

Facultative upland (FACU): plants, which usually occur in nonwetlands (from 67% to 99% of the time), but occasionally found in wetlands (from 1% to 33% of the time).

Upland (UPL): plants, which almost always (more than 99% of the time) occur under natural conditions in nonwetlands.

Indicators with a "+" or "-" mean that the species is more (+) or less (-) often found in wetlands than other plants with the same indicator status without the "+" or "-" designation.

An inundation tolerance indicator is provided to provide guidance on the sensitivity of plants to a depth and duration of flooding. Plants that can withstand a period of standing water are indicated with a "yes". Additional information is provided for depth of inundation and tolerance for seasonal inundation, saturated soil conditions, pollution, and salt. Additional research may be warranted to ensure successful plant establishment.

STORMWATER PLANT LIST A- HERBACEOUS VEGETATION

					TOLERANCE		
COMMON	SCIENTIFIC	FORM	ZONE	INDICATOR	INUNDATION	P0LLUTION	SALT
ARROW-HEAD,BROAD-LEAF	Sagittaria latifolia	Perennial	[1,2],3	OBL	0-2'	No	No
ARROW-HEAD,GRASS-LEAF	Sagittaria graminea	Perennial	[1,2],3	OBL	0-1'	No	No
ARROW-HEAD,NORTHERN	Sagittaria cuneata	Perennial	[1,2],3	OBL	Yes	No	No
ARROW-HEAD,WAPATO DUCK POTATO	Sagittaria latifolia	Perennial	[1,2],3	OBL	0-2'	No	No
ASTER,CALICO	Aster lateriflorus	Perennial	[2,3,4]	FACW-	Seasonal	No	No
ASTER,FLAT-TOP WHITE	Aster umbellatus	Perennial	[2,3],4	FACW	Yes	No	No
ASTER,NEW ENGLAND	Aster novae-angliae	Perennial	[2,3],4	FACW	Yes	No	No
ASTER,NEW YORK	Aster novi-belgii	Perennial	[2,3],4	FACW+	Yes	No	No
ASTER,SWAMP	Aster puniceus	Perennial	1,[2,3]	OBL	Yes	No	No
ASTER,TRADESCANT	Aster tradescanti	Perennial	[2,3],4	FACW	Yes	No	No
ASTER,WHITE HEATH	Aster ericoides	Perennial	3,[4,5,6]	FACU	No	No	No
BEARDTONGUE	Penstemon digitalis	Perennial	3,4,5	FAC	No	No	No
BENTGRASS,PERENNIAL	Agrostis perennans	Grass	[4,5],6	FACU	Yes	No	No
BENTGRASS,SPREADING	Agrostis stolonifera	Grass	[2,3],4	FACW	Yes	No	No
BENTGRASS,WINTER	Agrostis hyemalis	Grass	[3,4],5	FAC	No	No	No
BERGAMOT,WILD	Monarda fistulosa	Perennial	[4,5,6]	UPL	No	No	No
BLACK-EYED SUSAN	Rudbeckia hirta (yellow)	Perennial	4,5,6	FACU-	No	No	No
BLOODROOT	Sanguinaria canadensis	Perennial	4,[5,6]	UPL,FACU-	No	No	No
BLUEGRASS,GROVE	Poa alsodes	Grass	2,[3,4],5	FACW-	Seasonal	No	No
BLUESTEM,BIG	Andropogon gerardii	Grass	[4,5],6	FAC	No	No	No
BULRUSH, HARDSTEMMED	Scirpus acutus	Perennial	[1,2],3	OBL	0-3'	No	No
BULRUSH, SOFTSTEM	Scirpus validus	Perennial	[1,2,],3	OBL	0-1'	No	No
BULRUSH,RIVER	Scirpus fluviatilis	Grass	[1,2],3	OBL	0-1'	No	No
BULRUSH,THREE-SQUARE	Scirpus pungens	Grass	[2,3],4	FACW+	0-6"	No	No
BURREED,AMERICAN	Sparganium americanum	Grass	[1,2],3	OBL	0-1'	No	No

						TOLERANCE	
COMMON	**SCIENTIFIC**	**FORM**	**ZONE**	**INDICATOR**	**INUNDATION**	**P0LLUTION**	**SALT**
BURREED,GIANT	Sparganium eurycarpum	Grass	[1,2],3	OBL	Yes	No	No
CARDINAL FLOWER	Lobelia cardinalis	Perennial	1,[2,3],4	FACW+	Yes	No	No
CHOKEBERRY,BLACK	Aronia melanocarpa	Shrub	[4,5]	FAC	No	No	No
COLUMBINE,WILD	Aquilegia canadensis	Perennial	[3,4],5	FAC	No	No	No
CONEFLOWER,CUT-LEAF	Rudbeckia laciniata	Perennial	[2,3],4	FACW	Yes	No	No
CORDGRASS,PRAIRIE	Spartina pectinata	Grass	[1,2],3	OBL	Salt, Edge	No	Yes
CRANBERRY,HIGH BUS	Viburnum trilobum	Shrub	[3,4]	FACW	Yes	No	No
CUTGRASS,RICE	Leersia oryzoides	Grass	[1,2],3	OBL	0-6"	No	No
DOGWOOD,RED-OSIER	Cornus stolonifera	Shrub	[4]	FACW	Seasonal	No	No
DUCKWEED,LESSER	Lemna minor	Perennial	[1,2],3	OBL	Free Float	No	No
ELDERBERRY	Sambucus candensis	Shrub	[4]	FACW	Seasonal	No	No
FALSE-HELLEBORE,AMERICAN	Veratrum viride	Perennial	[2,3,4]	FACW+	Yes	No	No
FALSE-SOLOMON'S-SEAL,FEATHER	Smilacina racemosa	Perennial	[4,5],6	FACU-	No	No	No
FERN,CINNAMON	Osmunda cinnamomea	Fern	[2,3],4	FACW	Saturated	No	No
FERN,NEW YORK	Thelypteris noveboracensis	Fern	[3,4],5	FAC	Saturated	No	No
FERN,ROYAL	Osmunda regalis	Fern	[1,2],3	OBL	Saturated	No	No
FERN,SENSITIVE	Onoclea sensibilis	Fern	[2,3],4	FACW	Saturated	No	No
FESCUE,RED	Festuca rubra	Groundcover	[4,5]	FACU	No	No	No
GRASS,CANADA MANNA	Glyceria canadensis	Grass	[1,2],3	OBL	0-1'	No	No
GRASS,FOWL MANNA	Glyceria striata	Grass	[1,2],3	OBL	Seasonal	No	No
GRASS,ROUGH BARNYARD	Echinochloa muricata	Grass	[2,3],4	FACW+	Yes	No	No
HAREBELL	Campanula rotundifolia	Perennial	[5,6]	FACU		No	No
HOBBLEBUSH	Viburnum alnifolium	Shrub	[5]	FAC	No	No	No
HONEYSUCKLE,BUSH	Diervilla lonicera	Shrub	[6]	UPL	No	No	No
HORNWORT,COMMON	Ceratophyllum demersum	Perennial	[1,2],3	OBL	1-5'	No	No
HORSETAIL,ROUGH	Equisetum hyemale	Grass	[2,3],4	FACW	Yes	No	No
INDIAN-TOBACCO	Lobelia inflata	Perennial	[4,5,6]	FACU	No	No	No

						TOLERANCE	
COMMON	SCIENTIFIC	FORM	ZONE	INDICATOR	INUNDATION	P0LLUTION	SALT
IRIS, BLUE WATER	Iris versicolor	Perennial	[1,2],3	OBL	0-6"	No	No
JACK-IN-THE-PULPIT,SWAMP	Arisaema triphyllum	Perennial	[2,3],4	FACW	Seasonal	No	No
LILY,CANADA	Lilium canadense	Perennial	2,[3,4]	FAC+	Yes	No	No
LOBELIA,BROOK	Lobelia kalmii	Perennial	[1,2],3	OBL	Yes	No	No
LOBELIA,PALE-SPIKE	Lobelia spicata	Perennial	[3,4,5]	FAC-	No	No	No
LOBELIA,WATER	Lobelia dortmanna	Perennial	[1,2],3	OBL	Yes	No	No
LOVEGRASS,PURPLE	Eragrostis pectinacea	Grass	[4,5],6	FAC	No	No	No
MARSH MARIGOLD	Caltha palustris	Perennial	3,4	OBL	6"Saturate	No	No
MARSH SMARTWEED	Polygonum puntatum	Perennial	2,3	OBL	Saturated	No	No
MONKEY-FLOWER	Mimulus ringens	Perennial	[1,2],3	OBL	Yes	No	No
MUHLY,MARSH	Muhlenbergia glomerata	Grass	[2,3],4	FACW	Yes	No	No
PARTRIDGE-BERRY	Mitchella repens	Groundcover	[4,5],6	FACU	No	No	No
PENNSYLVANIA SMARTWEED	Polygonum pensylvanicum	Annual	[2,3]	FACW	0-6"	No	No
PICKERELWEED	Pontederia cordata	Perennial	2,3	OBL	0-1'	No	No
PITCHER PLANT	Sarracenia purpurea	Perennial	[3,4]	OBL	Yes	No	No
PONDWEED,CLASPING-LEAF	Potamogeton perfoliatus	Perennial	[1,2],3	OBL	1' Min-6'	No	No
PONDWEED,LONG-LEAF	Potamogeton nodosus	Perennial	[1,2]	OBL	1' Min-6'	No	No
PONDWEED,SAGO	Potamogeton pectinatus	Perennial	[1,2]	OBL	1' Min-24'	No	No
REEDGRASS,BLUE-JOINT	Calamagrostis canadensis	Grass	[1,2],3	FACW+	6"Saturate	No	No
ROSE,VIRGINIA	Rosa virginiana	Shrub	[5]	FAC	No	No	No
RUSH,NARROW-PANICLE	Juncus brevicaudatus	Grass	[1,2],3	OBL	Yes	No	No
RUSH,SOFT	Juncus effusus	Grass	[2,3],4	FACW+	0-1'	No	No
SAXIFRAGE,SWAMP	Saxifraga pensylvanica	Perennial	[1,2],3	OBL	Yes	No	No
SAXIFRAGE,VIRGINIA	Saxifraga virginiensis	Perennial	[4,5]	FAC-	No	No	No
SEDGE,BEARDED	Carex comosa	Grass	[1,2],3	OBL	6"Saturate	No	No
SEDGE,CRESTED	Carex cristatella	Grass	[1,2],3,4	FACW	Yes	No	No
SEDGE,FOX	Carex vulpinoidea	Grass	[1,2],3	OBL	Sat. 0-6"	No	No
SEDGE,FRINGED	Carex crinita	Grass	[1,2],3	OBL	Yes	No	No
SEDGE,GRACEFUL	Carex gracillima	Grass	[4,5],6	FACU	No	No	No

						TOLERANCE	
COMMON	SCIENTIFIC	FORM	ZONE	INDICATOR	INUNDATION	P0LLUTION	SALT
SEDGE,HOARY	Carex canescens	Grass	[1,2],3	OBL	Yes	No	No
SEDGE,INLAND	Carex interior	Grass	1,[2,3]	OBL	Yes	No	No
SEDGE,LAKEBANK	Carex lacustris	Grass	[1,2],3	OBL	Sat.. 0-2'	No	No
SEDGE,LOOSE-FLOWERED	Carex laxiflora	Grass	[4,5,6]	FACU	No	No	No
SEDGE,RETRORSE	Carex retrorsa	Grass	[2,3],4	FACW+	Sat. 0-6"	No	No
SEDGE,SHALLOW	Carex lurida	Grass	[1,2],3	OBL	Yes	No	No
SEDGE,SWAN'S	Carex swanii	Grass	[4,5,6]	FACU	No	No	No
SEDGE,TUSSOCK	Carex stricta	Grass	[2,3,4]	OBL	Seasonal	No	No
SEDGE,UPTIGHT	Carex stricta	Grass	[1,2],3	OBL	Sat.0-6"	No	No
SEDGE,YELLOW-FRUIT	Carex annectens	Grass	[2,3,]4	FACW+	Yes	No	No
SPIKERUSH,BLUNT	Eleocharis obtusa	Grass	[1,2],3	OBL	0-6"	No	No
SPIKERUSH,CREEPING	Eleocharis palustris	Grass	[1,2],3	OBL	Seasonal	No	No
ST. JOHN'S-WORT,MARSH	Triadenum fraseri	Perennial	[1,2],3	OBL	Yes	No	No
STEEPLEBUSH	Spirea tomentosa	Shrub	[4]	FACW	Seasonal	No	No
SWAMP MILKWEED	Asclepias incarnata	Perennial	2,3	OBL	Saturated	No	No
SWAMP-LOOSESTRIFE,HAIRY	Decodon verticillatus	Perennial	[1,2],3	OBL	Yes	No	No
SWEETFLAG	Acorus calmus	Perennial	[2]	OBL	Yes	No	No
TRILLIUM,RED	Trillium erectum	Annual	[5,6]	FACU	No	No	No
TRILLIUM,WHITE	Trillium grandiflorum	Annual	[6]	UPL	No	No	No
TURTLEHEAD,WHITE	Chelone glabra	Perennial	[1,2],3	OBL	Yes	No	No
VERVAIN,BLUE	Verbena hastata	Perennial	2,3,4	FACW+	Yes	No	No
VIRGINIA WILD RYE	Elymus virginicus	Grass	2,[3,4]	FACW-	Yes	No	No
WATER ARUM	Calla palustris	Perennial	[2]	OBL		No	No
WATER SMARTWEED	Polygonum amphibium	Perennial	2,3	OBL	6"-Sat	No	No
WATER-LILY,WHITE	Nymphaea tuberosa	Perennial	[1,2],3	OBL	1-3'	No	No
WATER-LILY,YELLOW/ SPATTERDOCK	Nuphar advena/luteum	Perennial	[1,2],3	OBL	1-3'	No	No
WILD-LILY-OF-THE-VALLEY	Maianthemum canadense	Perennial	[4,5],6	FAC-	No	No	No
WINTERGREEN	Gaultheria procumbens	Shrub	[5,6]	FACU	No	No	No
WOOD-REEDGRASS,SLENDER	Cinna latifolia	Grass	[2,3,4]	FACW	Yes	No	No
WOODRUSH,COMMON	Luzula multiflora	Grass	[4,5,6]	FACU	No	No	No
WOOL-GRASS	Scirpus cyperinus	Grass	[2,3],4	FACW+	Seasonal	No	No

STORMWATER PLANT LIST B - WOODY VEGETATION

						TOLERANCE	
COMMON	SCIENTIFIC	FORM	ZONE	INDICATOR	INUNDATION	POLLUTION	SALT
ALDER,BROOK-SIDE	Alnus serrulata	Tree	[1,2],3	OBL	0-3"	No	No
ARROW-WOOD	Viburnum dentatum	Shrub	[3,4],5	FAC	Seasonal	No	Yes
ASH,BLACK	Fraxinus nigra	Tree	[2,3],4	FACW	Saturated	No	No
ASH,GREEN	Fraxinus pennsylvanica	Tree	[2,3],4	FACW	Seasonal	No	Yes
ASH,WHITE	Fraxinus americana	Tree	[4,5],6	FACU	No	No	No
ASPEN,BIG-TOOTH	Populus grandidentata	Tree	[4,5,6]	FACU	No	No	No
ASPEN,QUAKING	Populus tremuloides	Tree	[4,5],6	FACU	Yes	No	No
AZALEA,EARLY	Rhododendron	Shrub	[2,3,4],5	FAC,FAC+	Yes	No	No
BASSWOOD,AMERICAN	Tilia americana	Tree	3,[4,5],6	FACU	No	No	No
BEECH,AMERICAN	Fagus grandifolia	Tree	[4,5],6	FACU	No	No	No
BIRCH,GRAY	Betula populifolia	Tree	[3,4],5	FAC	Seasonal	No	No
BIRCH,PAPER	Betula papyrifera	Tree	[5,6]	FACU	No	No	No
BIRCH,YELLOW	Betula alleghaniensis	Tree	[3,4],5	FAC	Yes	No	No
BLADDERNUT, AMERICAN	Staphylea trifolia	Shrub-Tree	[3,4],5	FAC	Yes	No	No
BLUEBERRY,LOWBUSH	Vaccinium angustifolium	Shrub	3,[4,5,6]	FACU-,FACU	No	No	No
BLUEBERRY,VELVET-LEAF	Vaccinium myrtilloides	Shrub	1,2,[3,4,5]	FACU,FACW-	Yes	No	No
BOX-ELDER	Acer negundo	Tree	2,[3,4]	FAC+	Seasonal	No	No
BUFFALO-BERRY,CANADA	Shepherdia canadensis	Shrub	6	NI	No	No	Yes
BUTTERNUT	Juglans cinerea	Tree	[3,4,5,6]	FACU-,FACU+	Yes	No	No
BUTTONBUSH,COMMON	Cephalanthus occidentalis	Shrub	[1,2],3	OBL	0-3'	No	No
CEDAR,EASTERN RED	Juniperus virginiana	Shrub	4,5,6	FACU	No	Yes	No
CEDAR,NORTHERN WHITE	Thuja occidentalis	Tree	[2,3],4	FACW	Seasonal	No	No
CHERRY,BLACK	Prunus serotina	Tree	[4,5],6	FACU	No	No	No
CHERRY,CHOKE	Prunus virginiana	Tree	4,5,6	FACU	Yes	No	No
CHERRY,FIRE	Prunus pensylvanica	Tree	4,5,6	FACU	No	No	No

					TOLERANCE		
COMMON	SCIENTIFIC	FORM	ZONE	INDICATOR	INUNDATION	POLLUTION	SALT
CHERRY,PIN	Prunus pensylvanica	Tree	[5]	FACU	No	No	No
COTTON-WOOD,EASTERN	Populus deltoides	Tree	[3,4],5	FAC	Seasonal	Yes	Yes
CRANBERRY,SMALL	Vaccinium oxycoccos	Shrub	[1,2],3	OBL	Yes	No	No
DOGWOOD,REDOSIER	Cornus Stolonifera	Shrub	3,4		Yes	No	No
DOGWOOD,SILKY	Cornus amomum	Shrub	[2,3],4	FACW	Seasonal	No	No
ELDER,EUROPEAN RED	Sambucus racemosa	Shrub	[3,4,5],6	FACU,FACU+	Yes	No	No
ELM,SLIPPERY	Ulmus rubra	Tree	[3,4],5	FAC	Yes	No	No
FIR,BALSAM	Abies balsamea	Tree	[5]	FAC	Seasonal	No	No
GERMANDER,AMERICAN	Teucrium canadense	Shrub	1,[2,3,4],5	FAC+,FACW	Yes	No	No
HACKBERRY,COMMON	Celtis occidentalis	Shrub-Tree	4,5,6	FACU	Seasonal	Yes	No
HAWTHORN,COCKSPUR	Crataegus crus-galli	Tree	2,[3,4,5],6	FACU,FAC	Yes	Yes	No
HAZEL-NUT,BEAKED	Corylus cornuta	Shrub	3,[4,5,6]	UPL,FACU	No	No	No
HEMLOCK,EASTERN	Tsuga canadensis	Tree	4,5,6	FACU	No	No	No
HICKORY,BITTER-NUT	Carya cordiformis	Tree	4,5,6	FACU+	No	No	No
HICKORY,SHAG-BARK	Carya ovata	Tree	[3,4,5,6]	FACU-,FACU+	Yes	No	No
HOP-HORNBEAM,EASTERN	Ostrya virginiana	Shrub-Tree	[3,4,5,6]	FACU-,FACU+	Yes	No	No
HORNBEAM,AMERICAN	Carpinus caroliniana	Tree	[3,4],5	FAC	Some	No	No
HUCKLEBERRY,BLACK	Gaylussacia baccata	Shrub	3,[4,5],6	FACU	No	No	No
LARCH/TAMARACK	Larix laricina	Tree	[4,5]	FACW	No	No	No
MAPLE,MOUNTAIN	Acer spicatum	Tree	4,5,6	FACU	No	No	No
MAPLE,RED	Acer rubrum	Tree	[3,4],5	FAC	Seasonal	No	No
MAPLE,SILVER	Acer saccharinum	Tree	[2,3],4	FACW	Seasonal	No	No
MAPLE,STRIPED	Acer pensylvanicum	Shrub-Tree	3,[4,5,6]	FACU-,FACU	No	No	No
MAPLE,SUGAR	Acer saccharinum	Tree	[5,6]	FACU	No	No	No
MEADOW-SWEET,NARROW-LEAF	Spiraea alba	Shrub	[1,2,3,4],5	FACW,FACW+	Yes	No	No
NANNYBERRY	Viburnum lentago	Shrub	[3,4],5	FAC	Seasonal	No	No
OAK, SCARLET	Quercus coccinea	Tree	6		No	No	No

					TOLERANCE		
COMMON	**SCIENTIFIC**	**FORM**	**ZONE**	**INDICATOR**	**INUNDATION**	**POLLUTION**	**SALT**
OAK,BUR	Quercus macrocarpa	Tree	3,[4,5],6	FAC-	Yes	Yes	No
OAK,CHESTNUT	Quercus prinus	Tree	4,5,6	FACU	No	No	No
OAK,CHINKAPIN	Quercus muhlenbergii	Tree	[3,4],5	FAC	Yes	No	No
OAK,RED	Quercus rubra	Tree	6		No	Yes	No
OAK,SWAMP WHITE	Quercus bicolor	Tree	1,[2,3]	FACW+	Seasonal	No	No
OAK,WHITE	Quercus alba	Tree	[4,5,6]	FACU	Yes	No	No
PINE,EASTERN WHITE	Pinus strobus	Tree	4,5,6	FACU	No	Yes	Yes
PINE,PITCH	Pinus rigida	Tree	4,5,6	FACU	Seasonal	No	Yes
PLUM,CANADA	Prunus nigra	Tree	[6]	UPL	No	No	No
POPLAR,BALSAM	Populus balsamifera	Tree	[4]	FACW	Seasonal	No	No
RHODODENDRON	Rhododendron canadense	Shrub	1,[2,3,4],5	FACW	Yes	No	No
ROSEMARY,BOG	Andromeda polifolia	Shrub	[1,2],3	OBL	Yes	No	No
SASSAFRAS	Sassafras albidum	Tree	3,[4,5,6]	FACU-,FACU	No	No	No
SERVICE-BERRY,DOWNY	Amelanchier arborea	Shrub-Tree	2,[3,4,5],6	FAC-	Yes	No	No
SHEEP-LAUREL	Kalmia angustifolia	Shrub	3,[4,5],6	FAC	Yes	No	No
SPRUCE,WHITE	Picea glauca	Tree	[5,6]	FACU	No	No	No
STEEPLE-BUSH	Spiraea tomentosa	Shrub	1,[2,3,4],5	FACW	Yes	No	No
SUMAC,STAGHORN	Rhus typhina	Tree	[6]	UPL	No	No	No
SYCAMORE,AMERICAN	Platanus occidentalis	Tree	[2,3],4	FACW-	Saturated	No	No
TEABERRY	Gaultheria procumbens	Shrub	3,[4,5],6	FACU	No	No	No
VIBURNUM,MAPLE-LEAF	Viburnum acerifolium	Shrub	3,[4,5,6]	UPL,FACU	No	No	No
WILLOW,BLACK	Salix nigra	Tree	[2,3]	FACW+	Seasonal	No	No
WILLOW,PUSSY	Salix discolor	Tree	[3,4]	FACW	Seasonal	No	No
WILLOW,SILKY	Salix sericea	Shrub	[1,2],3	OBL	Yes	No	No
WILLOW,TALL PRAIRIE	Salix humilis	Shrub	3,[4,5],6	FACU	No	No	No
WINTERBERRY,COMMON	Ilex verticillata	Shrub	1,[2,3]	FACW+	Seasonal	No	No
WITCH-HAZEL, AMERICAN	Hamamelis virginiana	Shrub-Tree	3,[4,5],6	FAC-	No	Yes	No
WITHE-ROD	Viburnum cassinoides	Shrub	1,[2,3,4],5	FACW	Yes	No	No
YEW,AMERICAN	Taxus canadensis	Shrub	2,[3,4,5],6	FACU,FAC	Yes	No	No

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Appendix B: STP Construction Specifications

Appendix B1: USDA/NRCS Conservation Practice Standard

POND CONSTRUCTION SPECIFICATION GUIDANCE

Definition

A water impoundment made by constructing a dam or an embankment or by excavating a pit or dugout.

In this standard, ponds constructed by the first method are referred to as embankment ponds, and those constructed by the second method are referred to as excavated ponds. Ponds constructed by both excavation and the embankment methods are classified as embankment ponds if the depth of water impounded against the embankment at the principal spillway storm design high water elevation is 3 feet or more (See Table B.1).

This 3 feet must be measured from the low point on the upstream toe of the embankment to the design high water.

Purpose

To provide water for livestock, fish and wildlife, recreation, fire control, crop and orchard spraying, and other related uses, and to maintain or improve water quality. This standard also applies to stormwater management ponds.

Conditions where practice applies

General - This practice applies where it is determined that stormwater management, water supply, or temporary storage is justified and it is feasible and practicable to build a pond which will meet local and state law requirements.

This standard establishes the minimum acceptable quality for the design and construction of ponds if:

1. Failure of the dam will not result in loss of life; in damage to homes, commercial or industrial buildings, main highways, or railroads; or interruption of the use or service of public utilities.

Note: This document was adapted from Maryland Code 378 Pond Specifications. Conservation practice standards are reviewed periodically, and updated if needed. To obtain the current version of this standard, contact the NRCS.

2. The product of the storage times the effective height of the dam is less than 3,000. Storage is the volume, in acre-feet, in the reservoir below the elevation of the crest of the emergency spillway.

   The effective height of the dam is the difference in elevation, in feet, between the emergency spillway crest and the lowest point on a profile taken along the centerline of the dam, excluding the cutoff trench. If there is no emergency spillway, the top of the dam becomes the upper limit for determining the storage and the effective height.

3. For dams in rural areas, the effective height of the dam (as defined above) is 35 feet or less and the dam is hazard class "a". For dams in urban areas, the effective height of the dam is 20 feet or less and the dam is hazard class "a".

   Ponds exceeding any of the above conditions must be designed and constructed according to the requirements of Technical Release 60.

Exemptions - Soil Conservation District small pond approval is not required for small class "a" structures where the following exists:

1. Ponds or other structures have less than four (4) feet of embankment, or

2. The storage at emergency spillway design high water elevation according to Table B.1. does not exceed 40,000 cubic feet, and the height of the embankment is 6 feet or less.

The height of the embankment must be measured from the top of the dam to the lowest point of excavation, excluding the cutoff trench, along the centerline of the dam.

In addition, an embankment pond that meets the criteria below must be considered an excavated pond and is also exempt from small pond approval.

1. The calculation of 10H+20=L, where H=height from the pond bottom to the top of the dam, is provided, and

2. The projection of L horizontally downstream from the pond bottom is below the existing or proposed ground, and

3. The existing or proposed downstream ground slope within the projection of L is less than 10% at any point.

The review and design of such class "a" structures must be based on sound engineering judgment assuring a stable outfall for the ten (10) year, 24-hour storm event.

Table B.1. Hydrologic Criteria for Ponds

Structure Class	Storage Height Product[1]	Watershed Area (Acres)	Height To Emergency Spwy Crest (Feet)	Normal Surface Area (Acres)	Spillway Capacity[5] Principal[2] Rural	Spillway Capacity[5] Principal[2] Urban	Spillway Capacity[5] Emergency[3, 4] Rural	Spillway Capacity[5] Emergency[3, 4] Urban	Freeboard[6] Rural & Urban
“c” & “b”	Any	Any	Any	Any	TR 60	TR 60	TR 60	TR 60	TR 60
“a”	3,000 or more	Any	Any	Any	TR 60	TR 60	TR 60	TR 60	TR 60
		320	>20 - 35	≥12	25 YR	TR 60	100 YR	100 YR	2.0’ above E.S. Design Storm
	Less	and	≤20	≥12	10 YR	25 YR	100 YR	100 YR	
		Larger	<15	<12	5 YR	10 YR	50 YR	100 YR	
		100	>20 - 35	≥12	10 YR	TR 60	100 YR	100 YR	2.0’ above E.S. Design Storm
“a”	than	to	≤20	≥12	5 YR	10 YR	50 YR	100 YR	1.0’ above E.S. Design Storm
		320	<15	<12	2 YR	5 YR	25 YR	100 YR	1.0’ above E.S. Design Storm
		Less	>20 - 35	≥12	5 YR	TR 60	50 YR	100 YR	1.0’ above E.S. Design Storm
	3,000	Than	≤20	≥12	2 YR	5 YR	25 YR	100 YR	
		100	<15	<12	10% of 25 YR Peak	5 YR	25 YR	100 YR	

NOTES

1) The storage is defined as the original capacity of the reservoir in acre-feet at the elevation of the crest of the emergency spillway. The effective height is the difference in elevation in feet between the emergency spillway crest and the lowest point on a profile taken along the centerline of the dam, excluding the cutoff trench. If there is no emergency spillway, this height must be to the top of the dam.

2) Principal - minimum storm to be contained below the crest of the emergency spillway including any combination of temporary storage and principal spillway discharge.

3) Emergency - minimum storm used to proportion the emergency spillway to meet the limitations for shape, size, velocity and exit channel. This storm can be handled by any combination of principal spillway discharge, emergency spillway discharge and storage.

4) For ponds without a separate emergency spillway, the principal spillway functions as the emergency spillway. In this situation, the principal spillway must comply with the emergency spillway hydrologic criteria.

5) All ponds, which are being designed to meet local stormwater requirements, will be required to use the urban criteria. Storm duration used must be 24 hours except where TR-60 is specified.

6) For ponds without a functioning open channel emergency spillway, minimum freeboard will be 2 feet.

Site Conditions - Site Conditions must be such that runoff from the design storm can be safely passed through (1) a natural or constructed emergency spillway, (2) a combination of a principal spillway and an emergency spillway, or (3) a principal spillway.

Drainage Area - The drainage area above the pond must be protected against erosion to the extent that expected sedimentation will not shorten the planned effective life of the structure.

For ponds whose primary purpose is to trap sediment for water quality, adequate storage should be provided to trap the projected sediment delivery from the drainage area for the life of the pond.

If the intent is to maintain a permanent pool, the drainage area should be at least 4 acres for each acre-foot of permanent storage. These recommendations may be reduced if a dependable source of ground water or diverted surface water contributes to the pond. The water quality must be suitable for its intended use.

Soils Investigation - A soils investigation is required on all ponds. As a minimum it must include information along the centerline of the proposed dam, in the emergency spillway location, and the planned borrow area. The type of equipment used and the extent of the investigation will vary from site to site. All investigations must be logged using the Unified Soil Classification System.

Road Embankments - Where road embankments are being designed to impound a specific volume of water, either as a permanent pool or temporary stormwater storage, special design and evaluation criteria may be required.

Considerations

Water Quantity - The following items should be considered for water quantity:

1. Effects upon components of the water budget, especially effects on volumes and rates of runoff, infiltration, evaporation, transpiration, deep percolation, and ground water recharge.

2. Variability of effects caused by seasonal or climatic changes.

3. Effects on the downstream flows or aquifers that could affect other water uses or users.

4. Potential for multiple use.

5. Effects on the volume of downstream flow to prohibit undesirable environmental, social or economic effects.

Water Quality - The following items should be considered for water quality:

1. Effects on erosion and the movement of sediment, pathogens, and soluble and sediment attached substances that are carried by runoff.

2. Effects on the visual quality of on-site and downstream water resources.

3. Short-term and construction-related effects of this practice on the quality of downstream water courses.

4. Effects of water level control on the temperatures of downstream waters to prevent undesired effects on aquatic and wildlife communities.

5. Effects on wetlands and water-related wildlife habitats.

6. Effects of water levels on soil nutrient processes such as plant nitrogen use or denitrification.

7. Effects of soil water level control on the soil chemistry, soil water, or downstream water.

8. Potential for earth moving to uncover or redistribute sulfidic bearing soils.

Criteria

Embankment Ponds

Structure Hazard Classification - Documentation of the classification of dams is required. Documentation is to include but is not limited to location and description of dam, configuration of the valley, description of existing development (houses, utilities, highways, railroads, farm or commercial buildings, and other pertinent improvements), potential for future development, and recommended classification. It is also to include results obtained from breach routings, if breach routings are used as part of the classification process. The class (“a”, “b”, and “c”) as contained in this document is related to the potential hazard to life and property that might result from a sudden major breach of the earth embankment. Structure classification and land use for runoff determination must take into consideration the anticipated changes in land use throughout the expected life of the structure. The classification of a dam is the responsibility of the designer, and subject to review and concurrence of the approving authority.

The classification of a dam is determined only by the potential hazard from failure, not by the criteria. Classification factors in the National Engineering Manual, as supplemented, are given below:

Class “a” - Structures located in rural, agricultural or urban areas dedicated to remain in flood tolerant usage where failure may damage non-inhabited buildings, agricultural land, floodplains or county roads.

Class “b” - Structures located in rural, agricultural, or urban areas where failure may damage isolated homes, main highways or minor railroads or cause interruption of use or service of relatively important public utilities.

Class “c” - Structures located where failure may cause loss of life or serious damage to homes, industrial and commercial buildings, important public utilities, main highways, or railroads.

“Rural areas” is defined as those areas in which residents live on farms, in unincorporated settlements, or in incorporated villages or small towns. It is where agriculture, including woodland activities, and extractive industries, provides the primary employment base for residents and where such enterprises are dependent on local residents for labor.

Non-rural areas must be classified as urban.

Peak Breach Discharge Criteria - Breach routings are used to help delineate the area potentially impacted by inundation should a dam fail and can be used to aid dam classification. The breach hydrograph is the outflow hydrograph attributed to the sudden release of water in reservoir storage. This is due to a dam breach during non-storm conditions.

Stream routings made of the breach hydrograph are to be based upon topographic data and hydraulic methodologies mutually consistent in their accuracy and commensurate with the risk being evaluated.

The minimum peak discharge of the breach hydrograph, regardless of the techniques used to analyze the downstream inundation area, is as follows:

$$Q_{max} = 3.2\ H_w^{2.5}$$

where,

Q_{max} = the peak breach discharge, cfs.

H_w = depth of water at the dam at the time of failure, feet.

This is measured to the crest of the emergency spillway or to design high water, if no emergency spillway exists. Use “nonstorm” conditions downstream of the dam.

Where breach analysis has indicated that only overtopping of downstream roads will occur, the following guidelines will be used:

Class	**Depth of Flow (d) ft.**
“a”	$d \leq 1.5$
“b” & “c”	$d > 1.5$

Use and importance of the roadway must be considered when making a classification.

Hydrology - Principal and emergency spillways will be designed within the limitations shown on Table B.1. The storm duration used must be 24 hours except where TR-60 is specified. The pond must be designed to safely pass the base flow along with volume and peak rates of runoff from design storms, specified in Table 1. All storm water management ponds must be designed using urban criteria. This can be done by using principal and emergency spillways. The following must be used to determine runoff rates and volumes:

1. NRCS "Engineering Field Handbook, Part 650" or;
2. NRCS, NEH, Section 4, Hydrology" or;
3. NRCS, TR-55, "Urban Hydrology for Small Watersheds" or;
4. NRCS, TR-20, "Computer Program for Project Formulation" or,
5. Computer programs using NRCS hydrology methods with identifiable inputs and outputs as approved by the reviewing agency.

Earth Embankment

Top Width - The minimum top width of the dam is shown in Table B.2. When the embankment top is to be used as a public road, the minimum width is to be 16 feet for one-way and 26 feet for two-way traffic. If the embankment is to be used for infrequent vehicle crossings, the minimum top width must be 10 feet. Guardrails or other safety measures are to be used where necessary and are to meet the requirements of the responsible road authority.

Side Slopes - The combined upstream and downstream side slopes of the settled embankment must not be less than five horizontal to one vertical (5:1) with neither slope steeper than 2:1. If the dam is used as a road crossing with a top width greater than 26 feet, then the combined side slopes of the settled embankment must not be less than 4 horizontal to one vertical (4:1) with neither slope steeper than 2:1. Slopes must be designed to be stable in all cases, even if flatter side slopes are required.

Earth Cuts - If cuts in an existing fill or in natural ground are required for the rehabilitation of an existing pond spillway or the construction of a new pond, the slope of the bonding surfaces between the existing material in place and the fill to be placed must not be steeper than a ratio of two horizontal to one vertical (2:1).

Table B.2. Embankment

Total Height Of Embankment (Feet)	Minimum Top Width (Feet)
10 or less	6
11 - 14	8
15 - 19	10
20 - 24	12
25 - 34	14
35 or more	15

Foundation Cutoff - A cutoff trench of relatively impervious material must be provided under the entire length of the dam and must be located at or upstream from the centerline of the dam. The cutoff trench must have a bottom width adequate to accommodate the equipment used for excavation, backfill and compaction operations, with the minimum width being 4 feet, and must have side slopes no steeper than one horizontal to one vertical. Minimum depth must be 4 feet.

Impervious Core - Any impervious core within the embankment must be located at or upstream from the centerline of the dam, and must extend up the abutments to the 10-year water surface elevation. The impervious core must extend vertically from the cutoff trench up to the 10-year water surface elevation throughout the embankment.

Seepage Control - Seepage control is to be included: (1) if pervious layers are not intercepted by the cutoff; (2) if seepage from the abutments may create a wet embankment; (3) if the phreatic line intersects the downstream slope; or (4) if special conditions require drainage to insure a stable dam. The phreatic line must be drawn on a 4:1 slope starting on the inside slope at the normal pool elevation. For stormwater management ponds, normal pool must be considered as the 10-year water surface elevation.

Seepage may be controlled by (1) foundation abutment or embankment drains; (2) reservoir blanketing; or (3) a combination of these measures. Foundation drains may control seepage encountered in the cutoff trench during construction. These drains must be located downstream of the dam centerline and outside the limits of the proposed cutoff trench. All drains must be designed according to the section *Principal Spillway*, *Conduit Piping and Seepage Control*.

Wave Erosion Protection - Where needed to protect the face of the dam, special wave protection measures such as a bench, rock riprap, sand-gravel, soil cement or special vegetation must be provided. (Reference NRCS Technical Releases 56 & 69)

Freeboard - The top elevation of the settled embankment must be determined in accordance with minimum criteria established in Table B.1.

Allowance for Settlement - The design height of the dam must be increased by the amount needed to insure that the design top of fill elevation will be maintained after all settlement has taken place. This increase must not be less than 5 percent, except where detailed soil testing and lab analyses indicate a lesser amount is adequate.

Principal Spillway

Capacity - A conduit, with needed appurtenances, must be placed under or through the dam, except where a weir type structure is used. The minimum capacity of the principal spillway must be that required in Table B.1.

Crest Elevation of Inlet - The crest elevation of the principal spillway must be no less than 1.0 foot below the crest of the emergency spillway. The crest elevation is the invert elevation of the lowest opening 6 inches or larger in any direction.

The inlet or riser size for the pipe drops must be such that the flow through the structure goes from weir-flow control to pipe-flow control without going into orifice-flow control in the riser. The inlets and outlets must be designed and analyzed to function satisfactorily for the full range of flow and hydraulic head anticipated.

The riser must be analyzed for flotation assuming all orifices and pipes are plugged. The factor of safety against flotation must be 1.2 or greater.

Pipe Conduits - Pipe conduits under or through the dam must meet the following requirements:

1. All pipes must be circular in cross section except for cast-in-place reinforced concrete box culverts.

2. Pipe must be capable of withstanding the external loading without yielding, buckling, or cracking.

3. Pipe strength must be not less than those shown on Tables B.3, B.4 and B.5 for corrugated steel, aluminum, and plastic pipes and applicable ASTM's for other materials.

4. Where inlet or outlet flared sections are used, they must be made from materials compatible with the pipe.

5. All pipe joints must be made watertight by the use of flanges with gaskets, coupling bands with gaskets, bell and spigot ends with gaskets, or by welding. See Section *Construction Specifications* for details.

Table B.3. Minimum Gages Steel[12]

CORRUGATED STEEL PIPE
2 - 2/3 inches x 1/2 inch Corrugations

Fill Height Over Pipe (Feet)	Pipe Diameter in Inches 24 & Less	30	36	42	48
1 - 15	16	16	14	10	10
15 - 20	16	12	10	*	*
20 - 25	16	10	*	*	*

* Not Permitted

CORRUGATED STEEL PIPE
3 inches x 1 inch or 5 inch x 1 inch Corrugations

Fill Height Over Pipe (Feet)	Pipe Diameter (Inches)			Flowable Fill			
	36	42	48	54[3]	60[3]	66[3]	72[3]
1 - 15	16	16	16	14	14	14	14
15 - 20	16	16	12	14	14	14	14
20 - 25	14	14	10	14	14	14	14

Table B.4. Minimum Gages Aluminum[1,2]

CORRUGATED ALUMINUM PIPE
2 - 2/3 inches x 1/2 inch Corrugations

Fill Height Over Pipe (Feet)	Pipe Diameter in Inches 21 & Less	24	30
1 - 15	16	14	10
15 - 20	12	10	*
20 - 25	10	*	*

* Not Permitted

CORRUGATED ALUMINUM PIPE
3 inches x 1 inch Corrugations

Fill Height Over Pipe (Feet)	Pipe Diameter in Inches 30	36	42	48	54[3]
1 - 15	16	16	14	10	14
15 - 20	16	12	*	*	*
20 - 25	12	*	*	*	*

* Not Permitted

1. Coatings for corrugated metal must be as specified in the Construction Specifications.

2. Tables 3 and 4 were developed using the modified Spangler equation. Sizes other than those shown above are not permitted.

3. Must use flowable backfill as specified in the Construction Specifications and the pipe must be bituminous coated.

Table B.5. Acceptable Plastic Pipe for use in Earth Dam[1,2]

Nominal Pipe Size (inches)	Schedule or Standard Dimension Ratio (SDR)	Maximum Depth of Fill Over [3]
6 - 24	PVC Schedule 40	10
6 - 24	PVC Schedule 80	15
6 – 24	PVC SDR 26	10
6 - 24	Corrugated HDPE	10

1. See Specifications, Plastic Pipe
2. All designs based on Technical Release 77, Reference 20. Other diameters and / or fill heights may be used that meet all the requirements of TR-77.
3. Larger fill heights may be permitted when using flowable fill

6. The joints between sections of pipe must be designed to remain watertight after joint rotation and elongation caused by foundation consolidation.

The capacity of the pipe conduit must be adequate to discharge long duration, continuous or frequent flows without flow through the emergency spillway. The diameter of the pipe must be not less than 6 inches.

For dams 20 feet or less in effective height, the following pipe materials are acceptable: cast-iron, ductile iron, steel, corrugated steel or aluminum, concrete with rubber gaskets, plastic, and cast-in-place reinforced concrete box culverts. Plastic pipe that will be exposed to direct sunlight should be made of ultraviolet resistant materials and protected by coating or shielding. Connections of pipe to less flexible pipe or structures must be designed to avoid stress concentrations that could rupture the pipe.

For dams over 20 feet in effective height, conduits are to be reinforced concrete pipe, cast-in-place reinforced concrete box culverts, corrugated steel, ductile iron, welded steel or aluminum pipe. The maximum height of fill over any principal spillway steel, aluminum, or plastic pipe must not exceed 25 feet.

Concrete pipe must have a concrete cradle extending up the sides of the pipe at least 50% of its outside diameter with minimum thickness of 6 inches. Where a concrete cradle is not needed for structural reasons, flowable fill may be used as described in the *Construction Specifications* section of this standard. Gravel bedding is not permitted. Cantilever outlet sections, if used, must be designed to withstand the cantilever load. Pipe supports must be provided when needed. Other suitable devices such as plunge basin, stilling basin, impact basin, or rock riprap spreader should be used to provide a safe outlet. Cathodic protection is to be provided for welded steel and corrugated steel pipe where the need and importance of the structure warrant. Cathodic protection should normally be provided for corrugated steel pipe where the saturated soil resistivity is less than 4,000 ohms-cm or the pH is lower than 5. The National Handbook of Conservation Practices, Irrigation Water

Conveyance, Steel Pipeline Standard (430-FF), provides criteria for cathodic protection of welded steel pipes.

Multiple Conduits - Where multiple conduits are used, there must be sufficient space between the conduits and the installed anti-seep collars to allow for backfill material to be placed between the conduits by the earth moving equipment and for easy access by hand operated compaction equipment. This distance between conduits must be equal to or greater than half the pipe diameter but not less than 2 feet.

Conduit Piping and Seepage Control - Seepage along pipe conduit spillways extending through the embankment must be controlled by use of (1) anti-seep collars, or (2) filter and drainage diaphragm. Seepage control will not be required on pipes 6 inches in diameter or less.

Anti-seep collars must be installed around all conduits through earth fills according to the following criteria:

1. Sufficient collars must be placed to increase the seepage length along the conduit by a minimum of 15 percent of the pipe length located within the saturation zone.

2. The assumed normal saturation zone must be determined by projecting a line at a slope (4) horizontal to (1) vertical from the point where the normal water elevation meets the upstream slope to a point where this line intersects the invert of the pipe conduit or bottom of the cradle, whichever is lower. For Stormwater Management ponds, the phreatic line starting elevation must be the 10-year water elevation.

3. Maximum collar spacing must be 14 times the required projection above the pipe. The minimum collar spacing must be 5 times the required minimum projection.

4. Anti-seep collars should be placed within the saturated zone. In cases where the spacing limit will not allow this, at least one collar will be in the saturated zone.

5. All anti-seep collars and their connections to the conduit must be watertight and made of material compatible with the conduit.

6. Collar dimensions must extend a minimum of 2 feet in all directions around the pipe.

7. Anti-seep collars must be placed a minimum of two feet from pipe joints except where flanged joints are used.

8. For pipes with concrete cradles, the projection must be measured from the cradle.

Filter and drainage diaphragms are always recommended, but are required when the following conditions are encountered:

1. The pond requires design according to TR-60.

2. Embankment soils with high piping potential such as Unified Classes GM, SM, and ML.

Filter and drainage diaphragms must be designed in accordance with procedures from NRCS TR-60, Earth Dams and Reservoirs, Section 6, Principal Spillways, as described below.

The drainage diaphragm must usually consist of sand, meeting the fine concrete aggregate requirements (ASTM C-33). A design analysis must be made using Part 633 of the National Engineering Manual, Chapter 26, Gradation Design of Sand and Gravel Filters.

The drainage diaphragm must be a minimum of 3 ft thick and extend vertically upward and horizontally at least three times the conduit outside diameter or the width of the cradle, whichever is greater except that:

1. The vertical extension need be no higher than the maximum potential reservoir water level

2. The horizontal extension need be no further than 5 feet beyond the sides and slopes of any excavation made to install the conduit.

3. The minimum soil cover over any portion of the filter-drainage diaphragm measured normal to the nearest embankment surface must be at least 2 feet.

It must extend vertically downward at least 2 ft beneath the conduit outside diameter or bottom of the cradle, whichever is greater. The drainage diaphragm must be located immediately downstream of the cutoff trench, approximately parallel to the centerline of the dam but no further upstream than the centerline of the dam.

The drainage diaphragm must outlet at the embankment downstream toe, preferably using a drain backfill envelope continuously along the pipe to where it exits the embankment. Protecting drain fill from surface erosion will be necessary.

It is required that the outlet for the filter diaphragm is sized to safely discharge the design flow. Where a drain backfill envelope is used as the outlet, it is recommended that it be designed so the hydraulic head does not exceed the depth of the drain outlet. The exposed area of the drain outlet must also be protected from external attack such as surface erosion and slope instability due to horizontal seepage pressures. A weighted toe cover such as riprap can be effective if protected with a properly designed filter between the sand drain material and the riprap cover.

If pipe drain outlets are used, consideration must be given to the structural design of the conduit in resisting external loading and the design life of the pipe must be consistent with the design life of the dam and physical conditions of the site. Also, the pipe must be designed for capacity and size of perforations as outlined in NEH Part 633, Chapter 26 and Soil Mechanics Note 3. If the pipe corrodes, is crushed by exterior loading, or is otherwise damaged, the outlet of the filter diaphragm is lost and a piping failure may occur.

The design quantity (Q) used to size the outlet can be calculated by Darcy's Law,

$$Q = kiA$$

where:

k = permeability of the embankment or drain outlet material (ft/day)

i = hydraulic gradient where $i = h/l$

h = head differential (ft)

l = seepage path (ft)

A = area of flow (diaphragm or outlet) (ft^2)

Anti-vortex Devices - Drop inlet spillways are to have adequate anti-vortex devices. Splitter type anti-vortex devices must be placed in line with the barrel. An anti-vortex device is not required if weir control is maintained in the riser through all flow stages.

Trash Racks - All pipe and inlet structures must have a trash rack. Openings for trash racks must be no larger than 1/2 of the barrel conduit diameter, but in no case less than 6 inches.

Flush grates for trash racks are not acceptable. Inlet structures that have flow over the top must have a non-clogging trash rack such as a hood-type inlet extending a minimum of 8 inches below the weir openings, which allows passage of water from underneath the trash rack into the riser.

For inlet structures with solid covered tops, the bottom of the cover slab must be set at an elevation to prevent orifice flow control before pipe flow control governs.

Low stage releases, where the opening is larger than 6 inches, must have a non-clogging trash rack with openings no larger than half the low flow dimension.

For all low stage releases 6 inches or smaller in any direction, the emergency spillway design storm must be routed assuming the release has failed, using storage and discharge only above the elevation of the next opening larger than 6 inches in all directions. This design storm routing must not overtop the dam.

Drain Pipe - A pipe with a suitable valve must be provided to drain the pool area, where needed for proper pond management. The principal spillway conduit may serve as a pond drain, when so located, to accomplish this function.

Water Supply Pipes or Utilities - All pipes through the dam must have an inside diameter of not less than 1 1/4 inches. Pipes / utilities not parallel to the axis of the dam must meet all principal spillway requirements (i.e. filter diaphragm, embankment soils, etc.). Pipes / utilities parallel to the axis of the dam must be constructed with no granular bedding.

Earth Emergency Spillways

Emergency spillways are provided to convey large flood flows safely past earth embankments. An emergency spillway must be provided for each dam, unless the principal spillway is large enough to pass the routed design hydrograph peak discharge and any trash without overtopping the dam. The only design that may be utilized without an emergency spillway is: a principal spillway with a cross-sectional area of 3 square feet or more and an inlet that will not clog, such as a hood-type inlet which allows passage of water from underneath the trash rack into the riser.

Capacity - The minimum capacity of emergency spillways must be that required to pass the peak flow expected from a design storm of the frequency and duration shown in Table B.1 less any reduction creditable to conduit discharge and detention storage.

The emergency spillway must (1) safely pass the storm design peak or (2) the storm runoff must be routed through the reservoir. The routing must start with the water surface at the elevation of the crest of the principal spillway, or at the water surface after 10 days drawdown, whichever is higher. The 10-day drawdown must be computed from the crest of the emergency spillway or from the elevation that would be attained had the entire design storm been impounded, whichever is lower. Emergency spillways are to provide for passage of the design flow at a non-erosive velocity to a point downstream where the dam will not be endangered.

Component Parts - Earth spillways are open channels and usually consist of an inlet channel, level section, and an exit channel. The minimum difference in elevation between the crest of the emergency spillway and the settled top of dam must be 2.0 feet.

Cross-Section - Earth spillways must be trapezoidal and must be located in undisturbed earth. The side slopes must be stable for the material in which the spillway is to be constructed, but not steeper than 2:1. The emergency spillway must have a bottom width of not less than 8 feet.

The inlet channel may be curved to fit existing topography; however, it should be flared to allow unrestricted flow to the level section. The level section should be located as near the centerline of dam as possible. The level section must be 25 feet in length, and must be rectangular or square.

Exit channel centerline must be perpendicular to the level section downstream edge and must be straight for a distance beyond the downstream toe, so that discharges will not reach the earth embankment. The grade of the exit channel must fall within the range established by discharge requirement and permissible velocities.

The crest of any “token” spillway will be located at or above the 100-year storm elevation in undisturbed earth and have a minimum depth of one foot and bottom width of 8 feet.

Permissible Velocities - Earth spillways must be designed for non-erosive velocities through the control section and to a point downstream where the dam will not be endangered. The maximum permissible velocity for the grass and grass mixture to be used must be selected from Table B.6. Velocities exceeding these values will require use of linings other than vegetation.

Infiltration / Water Quality Basins – Ponds, either excavated or embankment, that are designed solely for infiltration or as water quality basins will have an emergency spillway. The capacity of the spillway will be determined by the following procedure:

Pass the routed 100-Year Storm with 1 foot of freeboard to the top of dam elevation. Routing will begin at the emergency spillway crest.

Structural Emergency Spillways

Chutes or drops, when used for principal spillways or principal-emergency or emergency spillways, must be designed in accordance with the principals set forth in the National Engineering Handbook, Section 5 “Hydraulics”; Section 11 “Drop Spillways”; and Section 14 “Chute Spillways”. The minimum capacity of a structural spillway must be that required to pass the peak flow expected from a design storm of the frequency and duration shown in Table B.1 less any reduction creditable to conduit discharge and detention storage.

Table B.6. Permissible Velocities (Ft/Sec) For Emergency Spillways Lined with Vegetation

	Slope Of Exit Channel	
Type of Cover	**0 - 5%**	**5 - 10%**
Bermudagrass	6	5
Reed Canarygrass	5	4
Tall Fescue	5	4
Kentucky Bluegrass	5	4
Grass-legume mixture	4	3

Visual Resource Design

The visual design of ponds must be carefully considered in areas of high public visibility and those associated with recreation. The underlying criterion for all visual design is appropriateness. The shape and form of ponds, excavated material, and plantings are to relate visually to their surroundings and to their functions.

The embankment may be shaped to blend with the natural topography. The edge of the pond should be shaped so that it is generally curvilinear rather than rectangular. Excavated material must be shaped so that the final form is smooth, flowing, and fitting to the adjacent landscape rather than angular geometric mounds. If feasible, islands may be added for visual interest and to attract wildlife.

Trees and Shrubs

Non-Roadway Embankments - Trees and/or shrubs will not be allowed on any embankment, will not be allowed within the buffer zone (15 feet from the toe of the dam), and will not be allowed within a 25-foot radius around the inlet structure.

Roadway Embankments - Trees and/or shrubs will not be allowed on any embankment, except for dry stormwater management structures that will be utilized as a roadway under all the following conditions:

1. Plantings may only be on top of the dam along the roadway and/or sidewalks.
2. The top of the dam must have a minimum of 50-foot top width.
3. Plantings will not be allowed on the side slopes of the embankment.
4. Plantings will not be allowed within the buffer zone (15 feet from the toe of the dam).
5. Plantings will only be shallow rooted (roots less than 3' deep) trees or shrubs.
6. The pond is a "dry" structure (normal pool not exceeding 18 inches).
7. A landscape plan showing type and location of planting must be prepared by a Landscape Architect certifying shallow rooted plants (roots less than 3' deep) under mature conditions.
8. A minimum of 3 feet freeboard above the 100-year water surface elevation must be maintained.
9. The structure is a low hazard (Class "a") pond.

Safety

Special considerations should be made for safety and access during the design of a pond. Measures to be considered may include fencing, slope benching, access roads, flattened side slopes, etc. When fencing a structure, the fence will be located so it will not interfere with the operation of the emergency spillway.

Excavated Ponds

General - Excavated ponds that create a failure potential through a constructed or created embankment will be designed as embankment ponds. Excavated ponds that include a pipe or weir outlet control system for urban stormwater management must be designed using the principal and emergency spillway hydrologic criteria for Embankment Ponds, Table B.1.

Side Slopes - Side slopes of excavated ponds must be such that they will be stable and must not be steeper than 1 horizontal to 1 vertical. Flatter slopes are to be utilized where safety for children, livestock watering, etc. is a design factor.

Perimeter Form - Where the structures are used for recreation or are located in high public view, the perimeter or edge should be shaped to a curvilinear form.

Inlet Protection - When the excavated pond is a bypass type and water is being diverted from a stream, the minimum size inlet line must be a 4-inch diameter pipe. All state laws concerning water use and downstream rights must be strictly adhered to.

Where surface water enters the pond in a natural or excavated channel, the side slope of the pond must be protected against erosion.

Outlet Protection - An excavated pond with a low embankment (combination excavation / embankment pond) must be designed to ensure a stable outfall for the 10-year, 24-hour frequency storm.

Placement of Excavated Material - The material excavated from the pond must be placed in one of the following ways so that its weight will not endanger the stability of the pond side slopes and where it will not be washed back into the pond by rainfall:

1. Uniformly spread to a height not exceeding 3 feet with the top graded to a continuous slope away from the pond;

2. Uniformly placed or shaped reasonably well with side slopes no steeper than 2 to 1. The excavated material will be placed at a distance equal to the depth of the pond, but not less than 12 feet from the edge of the pond;

3. Shaped to a designed form that blends visually with the landscape;

4. Used for low embankment and leveling; or

5. Hauled away.

Reservoir Area for Wet Ponds

For most ponds, the topography of the site must permit storage of water at a depth and volume that ensures a dependable supply, considering beneficial use, sedimentation, season of use, and evaporation and seepage losses. Soils in the reservoir must be impervious enough to minimize seepage losses or must be of a type that sealing is practical.

Excavation and shaping required to permit the reservoir area to suitably serve the planned purpose must be included in the construction plans.

Construction Specifications

These specifications are appropriate to all ponds within the scope of this specification. All references to ASTM and AASHTO specifications apply to the most recent version.

Site Preparation

Areas designated for borrow areas, embankment, and structural works must be cleared, grubbed and stripped of topsoil. All trees, vegetation, roots and other objectionable material must be removed. Channel banks and sharp breaks must be sloped to no steeper than 1:1. All trees must be cleared and grubbed within 15 feet of the toe of the embankment.

Areas to be covered by the reservoir will be cleared of all trees, brush, logs, fences, rubbish and other objectionable material unless otherwise designated on the plans. Trees, brush, and stumps must be cut approximately level with the ground surface. For dry stormwater management ponds, a minimum of a 25-foot radius around the inlet structure must be cleared.

All cleared and grubbed material must be disposed of outside and below the limits of the dam and reservoir as directed by the owner or his representative. When specified, a sufficient quantity of topsoil will be stockpiled in a suitable location for use on the embankment and other designated areas.

Earth Fill

Material - The fill material must be taken from approved designated borrow areas. It must be free of roots, stumps, wood, rubbish, stones greater than 6”, frozen or other objectionable materials. Fill material for the center of the embankment, and cut off trench must conform to Unified Soil Classification GC, SC, CH, or CL and must have at least 30% passing the #200 sieve. Consideration may be given to the use of other materials in the embankment if designed by a geotechnical engineer. Such special designs must have construction supervised by a geotechnical engineer.

Materials used in the outer shell of the embankment must have the capability to support vegetation of the quality required to prevent erosion of the embankment.

Placement - Areas on which fill is to be placed must be scarified prior to placement of fill. Fill materials must be placed in maximum 8 inch thick (before compaction) layers which are to be continuous over the entire length of the fill. The most permeable borrow material must be placed in the downstream portions of the embankment. The principal spillway must be installed concurrently with fill placement and not excavated into the embankment.

Compaction - The movement of the hauling and spreading equipment over the fill must be controlled so that the entire surface of each lift must be traversed by not less than one tread track of heavy equipment or compaction must be achieved by a minimum of four complete passes of a sheepsfoot, rubber tired or vibratory roller. Fill material must contain sufficient moisture such that the required degree of compaction will be obtained with the equipment used. The fill material must contain sufficient moisture so that if formed into a ball it will not crumble, yet not be so wet that water can be squeezed out.

When required by the reviewing agency the minimum required density must not be less than 95% of maximum dry density with a moisture content within ±2% of the optimum. Each layer of fill must be compacted as necessary to obtain that density, and is to be certified by the Engineer at the time of construction. All compaction is to be determined by AASHTO Method T-99 (Standard Proctor).

Cut Off Trench - The cutoff trench must be excavated into impervious material along or parallel to the centerline of the embankment as shown on the plans. The bottom width of the trench must be governed by the equipment used for excavation, with the minimum width being four feet. The depth must be at least four feet below existing grade or as shown on the plans. The side slopes of the trench must be 1 to 1 or flatter. The backfill must be compacted with construction equipment, rollers, or hand tampers to assure maximum density and minimum permeability.

Embankment Core - The core must be parallel to the centerline of the embankment as shown on the plans. The top width of the core must be a minimum of four feet. The height must extend up to at least the 10 year water elevation or as shown on the plans. The side slopes must be 1 to 1 or flatter. The core must be compacted with construction equipment, rollers, or hand tampers to assure maximum density and minimum permeability. In addition, the core must be placed concurrently with the outer shell of the embankment.

Structure Backfill

Backfill adjacent to pipes or structures must be of the type and quality conforming to that specified for the adjoining fill material. The fill must be placed in horizontal layers not to exceed four inches in thickness and compacted by hand tampers or other manually directed compaction equipment. The material needs to fill completely all spaces under and adjacent to the pipe. At no time during the backfilling operation must driven equipment be allowed to operate closer than four feet, measured horizontally, to any part of a structure. Under no circumstances must equipment be driven over any part of a concrete structure or pipe, unless there is a compacted fill of 24" or greater over the structure or pipe.

Structure backfill may be flowable fill meeting the requirements of Vermont Agency of Transportation Standard Specifications for Construction. The mixture must have a 100-200 psi; 28 day unconfined compressive strength. The flowable fill must have a minimum pH of 4.0 and a minimum resistivity of 2,000 ohm-cm. Material must be placed such that a minimum of 6" (measured perpendicular to the outside of the pipe) of flowable fill must be under (bedding), over and, on the sides of the pipe. It only needs to extend up to the spring line for rigid conduits. Average slump of the fill must be 7" to assure flowability of the material. Adequate measures must be taken (sand bags, etc.) to prevent floating the pipe. When using flowable fill, all metal pipe must be bituminous coated. Any adjoining soil fill must be placed in horizontal layers not to exceed four inches in thickness and compacted by hand tampers or other manually directed compaction equipment. The material must completely fill all voids adjacent to the flowable fill zone. At no time during the backfilling operation must driven equipment be allowed to operate closer than four feet, measured horizontally, to any part of a structure. Under no circumstances must equipment be driven over any part of a structure or pipe unless there is a compacted fill of 24" or greater over the structure or pipe. Backfill material outside the structural backfill (flowable fill) zone must be of the type and quality conforming to that specified for the core of the embankment or other embankment materials.

Pipe Conduits

All pipes must be circular in cross section.

Corrugated Metal Pipe - All of the following criteria must apply for corrugated metal pipe:

1. Materials - (Polymer Coated steel pipe) - Steel pipes with polymeric coatings must have a minimum coating thickness of 0.01 inch (10 mil) on both sides of the pipe. This pipe and its appurtenances must conform to the requirements of AASHTO Specifications M-245 & M-246 with watertight coupling bands or flanges.

   Materials - (Aluminum Coated Steel Pipe) - This pipe and its appurtenances must conform to the requirements of AASHTO Specification M-274 with watertight coupling bands or flanges. Aluminum Coated Steel Pipe, when used with flowable fill or when soil and/or water conditions warrant the need for increased durability, must be fully bituminous coated per requirements of AASHTO Specification M-190 Type A. Any aluminum coating damaged or otherwise removed must be replaced with cold applied bituminous coating compound. Aluminum surfaces that are to be in contact with concrete must be painted with one coat of zinc chromate primer or two coats of asphalt.

Materials - (Aluminum Pipe) - This pipe and its appurtenances must conform to the requirements of AASHTO Specification M-196 or M-211 with watertight coupling bands or flanges. Aluminum Pipe, when used with flowable fill or when soil and/or water conditions warrant for increased durability, must be fully bituminous coated per requirements of AASHTO Specification M-190 Type A. Aluminum surfaces that are to be in contact with concrete must be painted with one coat of zinc chromate primer or two coats of asphalt. Hot dip galvanized bolts may be used for connections. The pH of the surrounding soils must be between 4 and 9.

2. Coupling bands, anti-seep collars, end sections, etc., must be composed of the same material and coatings as the pipe. Metals must be insulated from dissimilar materials with use of rubber or plastic insulating materials at least 24 mils in thickness.

3. Connections - All connections with pipes must be completely watertight. The drain pipe or barrel connection to the riser must be welded all around when the pipe and riser are metal. Anti-seep collars must be connected to the pipe in such a manner as to be completely watertight. Dimple bands are not considered to be watertight.

   All connections must use a rubber or neoprene gasket when joining pipe sections. The end of each pipe must be re-rolled an adequate number of corrugations to accommodate the bandwidth. The following type connections are acceptable for pipes less than 24 inches in diameter: flanges on both ends of the pipe with a circular 3/8 inch closed cell neoprene gasket, pre-punched to the flange bolt circle, sandwiched between adjacent flanges; a 12-inch wide standard lap type band with 12-inch wide by 3/8-inch thick closed cell circular neoprene gasket; and a 12-inch wide hugger type band with o-ring gaskets having a minimum diameter of 1/2 inch greater than the corrugation depth. Pipes 24 inches in diameter and larger must be connected by a 24 inch long annular corrugated band using a minimum of 4 (four) rods and lugs, 2 on each connecting pipe end. A 24-inch wide by 3/8-inch thick closed cell circular neoprene gasket will be installed with 12 inches on the end of each pipe. Flanged joints with 3/8 inch closed cell gaskets the full width of the flange is also acceptable.

   Helically corrugated pipe must have either continuously welded seams or have lock seams with internal caulking or a neoprene bead.

4. Bedding - The pipe must be firmly and uniformly bedded throughout its entire length. Where rock or soft, spongy or other unstable soil is encountered, all such material must be removed and replaced with suitable earth compacted to provide adequate support.

5. Backfilling must conform to the *Structure Backfill* section of this standard.

6. Other details (anti-seep collars, valves, etc.) must be as shown on the drawings.

Reinforced Concrete Pipe - All of the following criteria must apply for reinforced concrete pipe:

1. Materials - Reinforced concrete pipe must have bell and spigot joints with rubber gaskets and must equal or exceed ASTM C-361.

2. Bedding - Reinforced concrete pipe conduits must be laid in a concrete bedding / cradle for their entire length. This bedding / cradle must consist of high slump concrete placed under the pipe and up the sides of the pipe at least 50% of its outside diameter with a minimum thickness of 6 inches. Where a concrete cradle is not needed for structural reasons, flowable fill may be used as described in the *Structure Backfill* section of this standard. Gravel bedding is not permitted.

3. Laying pipe - Bell and spigot pipe must be placed with the bell end upstream. Joints must be made in accordance with recommendations of the manufacturer of the material. After the joints are sealed for the entire line, the bedding must be placed so that all spaces under the pipe are filled. Care must be exercised to prevent any deviation from the original line and grade of the pipe. The first joint must be located within 4 feet from the riser.

4. Backfilling must conform to the *Structure Backfill* section of this standard.

5. Other details (anti-seep collars, valves, etc.) must be as shown on the drawings.

Plastic Pipe - The following criteria must apply for plastic pipe:

1. Materials - PVC pipe must be PVC-1120 or PVC-1220 conforming to ASTM D-1785 or ASTM D-2241. Corrugated High Density Polyethylene (HDPE) pipe, couplings and fittings must conform to the following: 4” – 10” inch pipe must meet the requirements of AASHTO M252 Type S, and 12” through 24” inch must meet the requirements of AASHTO M294 Type S.

2. Joints and connections to anti-seep collars must be completely watertight.

3. Bedding - The pipe must be firmly and uniformly bedded throughout its entire length. Where rock or soft, spongy or other unstable soil is encountered, all such material must be removed and replaced with suitable earth compacted to provide adequate support.

4. Backfilling must conform to the *Structure Backfill* section of this standard.

5. Other details (anti-seep collars, valves, etc.) must be as shown on the drawings.

Drainage Diaphragms - When a drainage diaphragm is used, a registered professional engineer will supervise the design and construction inspection.

Concrete

Concrete must meet the requirements of Vermont Agency of Transportation Standard Specifications for Construction.

Rock Riprap

Rock riprap must meet the requirements of Vermont Agency of Transportation Standard Specifications for Construction.

Geotextile must be placed under all riprap and must meet the requirements of Vermont Agency of Transportation Standard Specifications for Construction.

Care of Water During Construction

All work on permanent structures must be carried out in areas free from water. The Contractor must construct and maintain all temporary dikes, levees, cofferdams, drainage channels, and stream diversions necessary to protect the areas to be occupied by the permanent works. The contractor must also furnish, install, operate, and maintain all necessary pumping and other equipment required for removal of water from various parts of the work and for maintaining the excavations, foundation, and other parts of the work free from water as required or directed by the engineer for constructing each part of the work. After having served their purpose, all temporary protective works must be removed or leveled and graded to the extent required to prevent obstruction in any degree whatsoever of the flow of water to the spillway or outlet works and so as not to interfere in any way with the operation or maintenance of the structure. Stream diversions must be maintained until the full flow can be passed through the permanent works. The removal of water from the required excavation and the foundation must be accomplished in a manner and to the extent that will maintain stability of the excavated slopes and bottom required excavations and will allow satisfactory performance of all construction operations. During the placing and compacting of material in required excavations, the water level at the locations being refilled must be maintained below the bottom of the excavation at such locations which may require draining the water sumps from which the water must be pumped.

Stabilization

All borrow areas must be graded to provide proper drainage and left in a sightly condition. All exposed surfaces of the embankment, spillway, spoil and borrow areas, and berms must be stabilized by seeding, liming, fertilizing and mulching in accordance with the Natural Resources Conservation Service Standards and Specifications for Critical Area Planting or as shown on the accompanying drawings.

Erosion and Sediment Control

Construction operations will be carried out in such a manner that erosion will be controlled and water and air pollution minimized. State and local laws concerning pollution abatement will be followed. Construction plans must detail erosion and sediment control measures.

Operation and Maintainance

An operation and maintenance plan in accordance with Local or State Regulations will be prepared for all ponds. As a minimum, the attached dam inspection checklist must be included as part of the operation and maintenance plan and performed at least annually. Written records of maintenance and major repairs needs to be retained in a file. The issuance of a Maintenance and Repair Permit for any repairs or maintenance that involves the modification of the dam or spillway from its original design and specifications is required. A permit is also required for any repairs or reconstruction that involve a substantial portion of the structure. All indicated repairs are to be made as soon as practical.

Supporting Data and Documentation

Field Data and Survey Notes

The following is a list of the minimum data needed:

1. Profile along centerline of structure.
2. Profile along centerline of principal spillway.
3. Profile along centerline of emergency spillway.
4. Survey of storage area to develop topography and storage volumes.
5. Soil investigation logs and notes.

Design Data

Record on appropriate engineering paper. The following is a list of the minimum required design data:

1. Determine pond class and list appropriate spillway design criteria, including map.
2. Determine peak runoff from the contributing area for the design storms selected, including topo map.
3. Develop a stage-storage/discharge curve for the site.
4. Determine the pipe spillway by storm routing using the procedure in Chapter 11, EFH; Chapter 6, TR-55; or TR-20.
5. Design emergency spillway using EFH 11-61.
6. Drawings should show the following as a minimum: profile along centerline of dam; profile along centerline of emergency spillway; cross section through dam at principal spillway; cross section through emergency spillway; plan view; and construction details & notes and soil logs.
7. Compute earth fill (if needed).

8. Special design feature details; watering, fire hydrants, fish management, irrigation, outfall stabilization, etc.; structural details with design loadings, if applicable, should be shown on the drawings.

9. Record seeding plan on drawings.

10. A written *Operation and Maintenance Plan.*

Construction Check Data/As-built

Record on survey notepaper, SCS-ENG-28. Survey data for ponds will be plotted in red. All construction inspection visits must be recorded on appropriate documentation paper. The documentation must include the date, who performed the inspection, specifics as to what was inspected, all alternatives discussed, and decisions made and by whom. The following is a list of the minimum data needed for As-Builts:

1. A profile of the top of the dam.

2. A cross-section of the emergency spillway at the control section.

3. A profile along the centerline of the emergency spillway.

4. A profile along the centerline of the principal spillway extending at least 100 feet downstream of the fill.

5. The elevation of the principal spillway crest.

6. The elevation of the principal spillway conduit invert (inlet and outlet).

7. The diameter, length, thickness and type of material for the riser.

8. The diameter, length, and type of material for the conduit.

9. The size and type of anti-vortex and trash rack device and its elevations in relation to the principal spillway crest.

10. The number, size and location of the anti-seep collars.

11. The diameter and size of any low stage orifices or drain pipes.

12. Show the length, width, and depth of contours of the pool area so that design volume can be verified.

13. Notes and measurements to show that any special design features were met.

14. Statement on seeding and fencing.

15. Notes on site clean up and disposal.

16. Sign and date check notes to include statement that practice meets or exceeds plans and specifications.

REFERENCES

AWWA Standards, American Water Works Association, Denver, Colorado.

ASTM Standards, American Society for Testing and Materials, Philadelphia, Pennsylvania.

Engineering Field Handbook, Part 650, USDA, Soil Conservation Service.

Handbook of PVC Pipe Design and Construction, First Edition, Uni-Bell Plastic Pipe Association, Dallas, Texas, 1980.

Handbook of Steel Drainage and Highway Construction Products, Third Edition, American Iron and Steel Institute, Washington, D.C., 1983.

Maryland Dam Safety Manual, Maryland Department of Natural Resources, Water Resources Administration, Annapolis, Maryland, June 1993.

Maryland Technical Guide, Section IV, Standards and Specifications, USDA, Natural Resources Conservation Service.

National Engineering Handbook, Section 4, Hydrology, USDA, Natural Resources Conservation Service, March 1985.

National Engineering Handbook, Section 5, Hydraulics, USDA, Natural Resources Conservation Service, August 1956.

National Engineering Handbook, Section 11, Drop Spillways, USDA, Natural Resources Conservation Service, April 1968.

National Engineering Handbook, Section 14, Chute Spillways, USDA, Natural Resources Conservation Service, October 1977.

National Engineering Handbook, Part 633, Chapter 26, Gradation Design of Sand and Gravel Filters, USDA, Natural Resources Conservation Service, October 1994.

National Handbook of Conservation Practices, USDA, Natural Resources Conservation Service.

Standard Specifications for Materials and Methods of Sampling and Testing, Nineteenth Edition, American Association of State Highway and Transportation Officials, Washington D.C., 1998.

Standard Specifications for Construction, Vermont Agency of Transportation Standard Specifications for Construction, 2001.

Technical Release No. 20, *Computer Programs for Project Formulation Hydrology*, USDA, Natural Resources Conservation Service, 1992.

Technical Release No. 55, *Urban Hydrology for Small Watersheds*, USDA, Natural Resources Conservation Service, 1986.

Technical Release No. 56, *A Guide for Design and Layout of Vegetative Wave Protection for Earth Dam Embankments*, USDA, Natural Resources Conservation Service, 1974.

Technical Release No. 60, *Earth Dams and Reservoirs*, USDA, Natural Resources Conservation Service, 1985.

Technical Release 69, *Riprap for Slope Protection Against Wave Action*, USDA, Natural Resources Conservation Service, 1983.

Technical Release No. 77, *Design and Installation of Flexible Conduits*, USDA, Natural Resources Conservation Service, 1990.

ADOPTED FROM MARYLAND DAM SAFETY MANUAL (1993)

DAM INSPECTION CHECKLIST

To help the dam owner perform periodic safety inspections of the structure, a checklist is provided. Each item of the checklist should be completed. **Repair** is required when obvious problems are observed. **Monitoring** is recommended if there is potential for a problem to occur in the future. **Investigation** is necessary if the reason for the observed problem is not obvious.

A brief description should be made of any noted irregularities, needed maintenance, or problems. Abbreviations and short descriptions are recommended. Space at the bottom of the form should be used for any items not listed.

DAM ____________________ **DATE** ____________________
OWNER ____________________ **WEATHER** ____________________
INSPECTED BY ____________________ **POOL LEVEL** ____________________

Item	Comments	Y/N	MONITOR	REPAIR	INVESTIGATE
1. CREST					
a. Visual settlement?					
b. Misalignment?					
c. Cracking?					
2. UPSTREAM SLOPE					
a. Erosion?					
b. Ground cover in good condition?					
c. Trees, shrubs, or other woody vegetation?					
d. Longitudinal/Vertical cracks?					
e. Adequate riprap protection?					
f. Stone deterioration?					
g. Settlements, depressions, or bulges?					
3. DOWNSTREAM SLOPE					
a. Erosion?					
b. Ground cover in good condition?					
c. Trees, shrubs, or other woody vegetation?					
d. Longitudinal/Vertical cracks?					
e. Riprap protection adequate?					
f. Settlements, depressions, or bulges?					
g. Soft spots or boggy areas?					
h. Movement at or beyond toe?					
i. Boils at toe?					
4. DRAINAGE-SEEPAGE CONTROL					
a. Internal drains flowing?	Est. Left ______ gpm Est. Right ______ gpm				
b. Seepage at toe?	Estimated ______ gpm				
c. Does seepage contain fines?					

INSPECTION CHECKLIST - PAGE 2

INSPECTED BY ________________ **DATE** ____________________

Item	Comments	Y/N	MONITOR	REPAIR	INVESTIGATE
5. ABUTMENT CONTACTS					
a. Erosion?					
b. Differential movement?					
c. Cracks?					
d. Seepage?	Estimated ______ gpm				
e. Adequate erosion protection for ditches?					
6. INLET STRUCTURE	Concrete or Metal Pipe (circle one)				
a. Seepage into structure?					
b. Debris or obstructions?					
c. If concrete, do surfaces show:					
1. Spalling?					
2. Cracking?					
3. Erosion?					
4. Scaling?					
5. Exposed reinforcement?					
6. Other?					
d. If metal, do surfaces show:					
1. Corrosion?					
2. Protective Coating deficient?					
3. Misalignment or split seams?					
e. Do the joints show:					
1. Displacement or offset?					
2. Loss of joint material?					
3. Leakage?					
f. Are the trash racks:					
1. Broken or bent?					
2. Corroded or rusted?					
3. Obstructed?					
4. Operational?					
g. Sluice/Drain gates:					
1. Broken or bent?					
2. Corroded or rusted?					
3. Leaking?					
4. Not seated correctly?					
4. Periodically maintained?					
5. Operational?					

INSPECTION CHECKLIST - PAGE 3

INSPECTED BY ________________ **DATE** ____________________

Item	Comments	Y/N	MONITOR	REPAIR	INVESTIGATE
7. PRINCIPAL SPILLWAY PIPE	Concrete or Metal Pipe (circle one)				
a. Seepage into conduit?					
b. Debris present?					
c. Do concrete surfaces show:					
1. Spalling?					
2. Cracking?					
3. Erosion?					
4. Scaling?					
5. Exposed reinforcement?					
6. Other?					
d. Do the joints show:					
1. Displacement or offset?					
2. Loss of joint material?					
3. Leakage?					
8. STILLING BASIN/POOL	Riprap or Concrete (circle one)				
a. If concrete, condition of surfaces?					
b. Deterioration or displacement of joints?					
c. Outlet channel obstructed?					
d. Is released water:					
1. Undercutting the outlet?					
2. Eroding the embankment?					
3. Displacing riprap?					
4. Scouring the plunge pool?					
e. Tailwater elevation and flow condition:					
9. EMERGENCY SPILLWAY					
a. Is the channel:					
1. Eroding or backcutting?					
2. Obstructed?					
b. Trees or shrubs in the channel?					
c. Seepage present?					
d. Soft spots or boggy areas?					
e. Channel slopes eroding or sloughing?					
10. RESERVOIR					
a. High water marks?					
b. Erosion/Slides into pool area?					
c. Sediment accumulation?					
d. Floating debris present?					
e. Adequate riprap protection for ditches?					

Appendix B2: Construction Specifications for Infiltration Practices

Infiltration Trench General Notes and Specifications

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

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

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

3. A Class "C" geotextile or better should interface between the trench side walls and between the stone reservoir and gravel filter layers. A partial list of non-woven filter fabrics that meet the Class "C" criteria is contained below. Any alternative filter fabric must be approved by the review agency.

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

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

4. A 6 inch sand filter layer may be placed on the bottom of the infiltration trench in lieu of filter fabric, and should be compacted using plate compactors. The sand for the infiltration trench should be washed and meet AASHTO Std. M-43, Size No. 9 or No. 10. Any alternative sand gradation must be approved by the Engineer or the review agency.

5. The stone aggregate should be placed in lifts and compacted using plate compactors. A maximum loose lift thickness of 12 inches is recommended. The

aggregate for infiltration trenches should consist of clean, washed aggregate between 2 and 5 inches in diameter. The aggregate should be graded such that there will be few aggregates smaller than the selected size.

6. Following the stone aggregate placement, the filter fabric should be folded over the stone aggregate to form a 6-inch minimum longitudinal lap. The desired fill soil or stone aggregate should be placed over the lap at sufficient intervals to maintain the lap during subsequent backfilling.

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

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

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

10. PVC distribution pipes should be Schedule 40 and meet ASTM Std. D 1784. All fittings and perforations (1/2 inch in diameter) should meet ASTM Std. D 2729. A perforated pipe should be provided only within the infiltration trench and should terminate 1 foot short of the infiltration trench wall. The end of the PVC pipe should be capped.

11. The corrugated metal distribution pipes should conform to AASHTO Std. M-36, and should be aluminized in accordance with AASHTO Std. M-274. Coat aluminized pipe in contact with concrete with an inert compound capable of affecting isolation of the deleterious effect of the aluminum on the concrete. Perforated distribution pipe should be provided only within the infiltration trench and should terminate 1 foot short of the infiltration trench wall. An aluminized metal plate should be welded to the end of the pipe.

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

13. The observation well is to consist of 6-inch diameter PVC Schedule 40 pipe (ASTM Std. D 1784) with a cap set flush with the ground level and located near the longitudinal center of the infiltration trench. The pipe should be perforated (1/2 inch in diameter) and placed vertically within the gravel portion of the

infiltration trench and a cap provided at the bottom of the pipe. The bottom of the cap should rest on the infiltration trench bottom. Preferably the observation well will not be located in vehicular traffic areas. The pipe should have a plastic collar with ribs to prevent rotation when removing cap. The screw top lid should be a “Panella” type cleanout with a locking mechanism or special bolt to discourage vandalism.

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

NOTE: PVC pipe with a wall thickness classification of SDR-35 meeting ASTM standard D3034 is an acceptable substitution for PVC Schedule 40 pipe.

Infiltration Basins Notes and Specifications

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

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

2. Initial basin excavation should be carried to within 1 foot of the final elevation of the basin floor. Final excavation to the finished grade should be deferred until all disturbed areas on the watershed have been stabilized or protected. The final phase excavation should remove all accumulated sediment. Relatively light tracked equipment is recommended for this operation to avoid compaction of the basin floor. After the final grading is completed, the basin provide a well-aerated, highly porous surface texture.

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

4. Establishing dense vegetation on the basin side slopes and floor is recommended. A dense vegetative stand will not only prevent erosion and sloughing, but will also provide a natural means of maintaining relatively high infiltration rates. Erosion protection of inflow points to the basin should also be provided.

5. Selection of suitable vegetative materials for the side slope and all other areas to be stabilized with vegetation and application of required lime, fertilizer, etc. should be done in accordance with the NRCS Standards and Specifications or your local Standards and Specifications for Soil Erosion and Sediment Control.

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

Appendix B3 : Construction Specifications for Sand Filters, Bioretention, & Open Channels

SAND FILTER SPECIFICATIONS

Material Specifications for Sand Filters

The allowable materials for sand filter construction are detailed in Table B.7.

Sand Filter Testing Specifications

Underground sand filters, facilities within sensitive groundwater aquifers, and filters designed to serve urban hot spots are to be tested for water tightness prior to placement of filter layers. Entrances and exits should be plugged and the system completely filled with water to demonstrate water tightness.

All overflow weirs, multiple orifices and flow distribution slots to be field-tested as to verify adequate distribution of flows.

Sand Filter Construction Specifications

Provide sufficient maintenance access; 12-foot-wide road with legally recorded easement. Vegetated access slopes to be a maximum of 10%; gravel slopes to 15%; paved slopes to 25%.

Absolutely no runoff is to enter the filter until all contributing drainage areas have been stabilized.

The surface of the filter bed should be completely level.

All sand filters should be clearly delineated with signs so that they may be located when maintenance is due.

Surface sand filters should be planted with appropriate grasses as specified in local NRCS Standards and Specifications guidance or other comparable guidance.

Pocket sand filters (and residential bioretention facilities treating areas larger than an acre) should be sized with an ornamental stone window covering approximately 10% of the filter area. This surface should be 2” to 5” size stone on top of a pea gravel layer (3/4 inch stone) approximately 4” to 6” in depth.

Table B.7. Sand Material Specifications

Parameter		Size	Notes
Sand	clean AASHTO M-6 or ASTM C-33 concrete sand	0.02" to 0.04"	Sand substitutions such as Diabase and Graystone #10 are not acceptable. No calcium carbonated or dolomitic sand substitutions are acceptable. Rock dust cannot be substituted for sand.
Peat	ash content: < 15% pH range: 5.2 to 4.9 loose bulk density 0.12 to 0.15 g/cc	n/a	The material must be Reed-Sedge Hemic Peat, shredded, uncompacted, uniform, and clean.
Underdrain Gravel	AASHTO M-43	0.375" to 0.75"	
Geotextile Fabric (if required)	ASTM D-4833 (puncture strength - 125 lb.) ASTM D-1117 (Mullen Burst Strength - 400 psi) ASTM D-4632 (Tensile Strength - 300 lb.)	0.08" thick equivalent opening size of #80 sieve	Must maintain 125 gpm per sq. ft. flow rate. Note: a 4" pea gravel layer may be substituted for geotextiles meant to separate sand filter layers.
Impermeable Liner (if required)	ASTM D-4833 (thickness) ASTM D-412 (tensile strength 1,100 lb., elongation 200%) ASTM D-624 (Tear resistance - 150 lb./in) ASTM D-471 (water adsorption: +8 to -2% mass)	30 mil thickness	Liner to be ultraviolet resistant. A geotextile fabric should be used to protect the liner from puncture.
Underdrain Piping	ASTM D-1785 or AASHTO M-278	6" rigid schedule 40 PVC	3/8" perf. @ 6" on center, 4 holes per row; minimum of 3" of gravel over pipes; not necessary underneath pipes
Concrete (Cast-in-place)	See local AOT Standards and Specs. f'c = 3,500 psi, normal weight, air-entrained; re-enforcing to meet ASTM 615-60	n/a	on-site testing of poured-in-place concrete required: 28 day strength and slump test; all concrete design (cast-in-place or pre-cast) *not using previously approved State or local standards* requires design drawings sealed and approved by a licensed professional structural engineer.
Concrete (pre-cast)	per pre-cast manufacturer	n/a	SEE ABOVE NOTE
Non-Rebar Steel	ASTM A-36	n/a	structural steel to be hot-dipped galvanized ASTM A-123

Specifications Pertaining to Underground Sand Filters

Provide manhole and/or grates to all underground and below grade structures. Manholes should be in compliance with standard specifications for each jurisdiction but diameters should be 30" minimum (to comply with OSHA confined space requirements) but not too heavy to lift. Aluminum and steel louvered doors are also acceptable. Ten-inch long (minimum) manhole steps (12" o.c.) should be cast in place or drilled and mortared into the wall below each manhole. A 5' minimum height clearance (from the top of the sand layer to the bottom of the slab) is required for all permanent underground structures. Lift rings are to be supplied to remove/replace top slabs. Manholes may need to be grated to allow for proper ventilation; if required, place manholes away from areas of heavy pedestrian traffic.

Underground sand filters should be constructed with a dewatering gate valve located just above the top of the filter bed should the bed clog.

Underground sand beds should be protected from trash accumulation by a wide mesh geotextile screen to be placed on the surface of the sand bed. The screen is to be rolled up, removed, cleaned and re-installed during maintenance operations.

SPECIFICATIONS FOR BIORETENTION

Material Specifications

The allowable materials to be used in bioretention area are detailed in Table B.8.

Planting Soil

The soil should be a uniform mix, free of stones, stumps, roots or other similar objects larger than two inches. No other materials or substances should be mixed or dumped within the bioretention area that may be harmful to plant growth, or prove a hindrance to the planting or maintenance operations. The planting soil should be free of noxious weeds.

The planting soil should be tested and should meet the following criteria:

pH range	5.2 - 7.0
organic matter	1.5 - 4%
magnesium	35 lb./ac
phosphorus P_2O_5	75 lb./ac
potassium K_2O	85 lb./ac
soluble salts	not to exceed 500 ppm

All bioretention areas should have a minimum of one test. Each test should consist of both the standard soil test for pH, phosphorus, and potassium and additional tests of organic matter, and soluble salts. A textural analysis is required from the site's

Table B.8. Materials Specifications for Bioretention

Parameter	Specification		Notes
Plantings	see your local NRCS Standards and Specifications guidance.	n/a	plantings are site-specific
Planting Soil [2.5' to 4' deep]	sand 35 - 60% silt 30 - 55% clay 10 - 25%	n/a	USDA soil types loamy sand, sandy loam or loam
Mulch	shredded hardwood		aged 6 months, minimum
Pea gravel diaphragm and curtain drain	pea gravel: ASTM D-448 ornamental stone: washed cobbles	pea gravel: No. 6 stone: 2" to 5"	
Geotextile	Class "C" apparent opening size (ASTM-D-4751) grab tensile strength (ASTM-D-4632) burst strength (ASTM-D-4833)	n/a	for use as necessary beneath underdrains only
Underdrain gravel	AASHTO M-43	0.375" to 0.75"	
Underdrain piping	ASTM D 1785 or AASHTO M-278	6" rigid schedule 40 PVC	3/8" perf. @ 6" on center, 4 holes per row; minimum of 3" of gravel over pipes; not necessary underneath pipes
Poured in place concrete (if required)	See local AOT Standards and Specs.; f'c = 3,500 lb. @ 28 days, normal weight, air-entrained; re-enforcing to meet ASTM 615-60	n/a	on-site testing of poured-in-place concrete required: 28 day strength and slump test; all concrete design (cast-in-place or pre-cast) *not using previously approved State or local standards* requires design drawings sealed and approved by a licensed professional structural engineer.
Sand [1' deep]	AASHTO M-6 or ASTM C-33	0.02" to 0.04"	Sand substitutions such as Diabase and Graystone #10 are not acceptable. No calcium carbonated or dolomitic sand substitutions are acceptable. No rock dust can be used for sand.

stockpiled topsoil. If topsoil is imported, then a texture analysis should be performed for each location where the topsoil was excavated.

Since different labs calibrate their testing equipment differently, all testing results should come from the same testing facility.

Should the pH fall out of the acceptable range, it may be modified (higher) with lime or (lower) with iron sulfate plus sulfur.

Compaction

It is very important to minimize compaction of both the base of the bioretention area and the required backfill. When possible, use excavation hoes to remove original soil. If bioretention area is excavated using a loader, the contractor should use wide track or marsh track equipment, or light equipment with turf type tires. Use of equipment with narrow tracks or narrow tires, rubber tires with large lugs, or high pressure tires will cause excessive compaction resulting in reduced infiltration rates and storage volumes and is not acceptable. Compaction will significantly contribute to design failure.

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

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

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

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

Plant Installation

Mulch around individual plants only. Shredded hardwood mulch is the only accepted mulch. Pine mulch and wood chips will float and move to the perimeter of the bioretention area during a storm event and are not acceptable. Shredded mulch must be well aged (6 to 12 months) for acceptance.

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

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

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

The topsoil specifications provide enough organic material to adequately supply nutrients from natural cycling. The primary function of the bioretention structure is to improve water quality. Adding fertilizers defeats, or at a minimum, impedes this goal. Only add fertilizer if wood chips or mulch is used to amend the soil. Rototill urea fertilizer at a rate of 2 pounds per 1,000 square feet.

Underdrains

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

The main collector pipe for underdrain systems should be constructed at a minimum slope of 0.5%. Observation wells and/or clean-out pipes must be provided (one minimum per every 1,000 square feet of surface area).

Miscellaneous

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

SPECIFICATIONS FOR OPEN CHANNELS AND FILTER STRIPS

Material Specifications

The recommended construction materials for open channels and filter strips are detailed in Table B.9.

Dry Swales

Roto-till soil/gravel interface approximately 6" to avoid a sharp soil/gravel interface.

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

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

System to have 6" of freeboard, minimum.

Table B.9. Open Vegetated Swale and Filter Strip Materials Specifications

Parameter	Specification	Size	Notes
Dry swale soil	USCS; ML, SM, SC	n/a	soil with a higher percent organic content is preferred
Dry Swale sand	ASTM C-33 fine aggregate concrete sand	0.02" to 0.04"	
Check Dam (pressure treated)	AWPA Standard C6	6" by 6" or 8" by 8"	*do not* coat with creosote; embed at least 3' into side slopes
Check Dam (natural wood)	Black Locust, Red Mulberry, Cedars, Catalpa, White Oak, Chestnut Oak, Black Walnut	6" to 12" diameter; notch as necessary	*do not* use the following, as these species have a predisposition towards rot: Ash, Beech, Birch, Elm, Hackberry, Hemlock, Hickories, Maples, Red and Black Oak, Pines, Poplar, Spruce, Sweetgum, Willow
Filter Strip sand/gravel pervious berm	sand: per dry swale sand gravel; AASHTO M-43	sand: 0.02" to 0.04" gravel: 1/2" to 1"	mix with approximately 25% loam soil to support grass cover crop; see Bioretention planting soil notes for more detail.
Pea gravel diaphragm and curtain drain	ASTM D 448	varies (No. 6) or (1/8" to 3/8")	use clean bank-run gravel
Underdrain gravel	AASHTO M-43	0.375" to 0.75"	
Underdrain	ASTM D-1785 or AASHTO M-278	6" rigid Schedule 40 PVC	3/8" perf. @ 6" o.c.; 4 holes per row
Geotextile	See local AOT Standards and Specs	n/a	
Rip rap	per local AOT criteria	size per Vermont AOT requirements based on 10-year design flows	

Side slopes to be 3:1 minimum; (4:1 or greater preferred).

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

Bottom of facility to be above the seasonably high water table.

Seed with flood/drought resistant grasses; see your local NRCS Standards and Specifications guidance.

Longitudinal slope to be 1 to 2%, maximum [up to 4% with check dams].

Bottom width to be 6' maximum to avoid braiding; larger widths may be used if proper berming is supplied. Width to be 2' minimum.

Wet Swales

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

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

Filter Strips

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

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

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

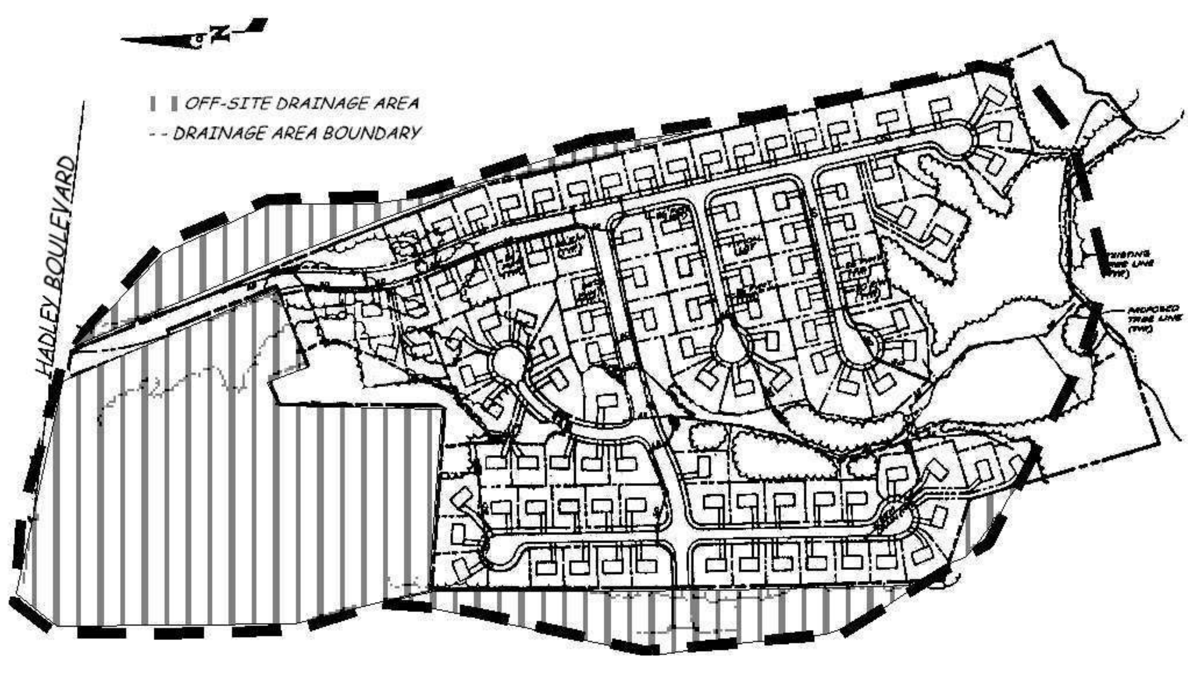
Appendix C: Step-by-Step Design Examples

Appendix C1: Stormwater Wet Pond Design Example

Sizing Example – Cole's Colony

The following sizing example is provided to illustrate how the storage volumes are calculated for a hypothetical development project. For the illustrative purposes of this example, it is assumed that all five criteria are applicable.

Cole's Colony is a hypothetical medium density, single family, residential development located in Brandon, VT. The site area is 45.1 acres and 108 lots are proposed. The site drains approximately 20 acres of offsite area for a total drainage area to the downstream property line of 65.1 acres. On-site soils consist of a mix of HSG "C" and "D" soils. The measured on-site imperviousness is 12 acres or 26.6% of the site (see Figure C.1). The following calculations illustrate the sizing and storage requirements for water quality, recharge, channel protection, and overbank and extreme flood management.



Base Data
Location: Brandon, VT
Site Area = 45.1 ac; Offsite Area = 20.0 ac (meadow)
Total Drainage Area (A) = 65.1
Measured Impervious Area=12.0 ac; or I=12.0/45.1=26.6%
Site Soils Types: 78% "C", 22% "D"
Offsite Soil Type: 100% "C"
Zoning: Residential (½ acre lots)

Hydrologic Data

	Pre	Post	Ult.
CN	72	78	82
tc (hr)	.46	.35	.35

Figure C.1. Cole's Colony Site Plan

Computation of Preliminary Stormwater Storage Volumes and Peak Discharges

Step 1. Water Quality Volume, WQv

- Compute Runoff Coefficient, Rv

$$Rv = 0.05 + (I)(0.009)$$
$$= 0.05 + (26.6)(0.009) = 0.29$$

- Compute WQv (Offsite area does not need to be considered when determining the water quality volume)

Use the 90% capture rule with P = 0.9” of rainfall.

$$WQv = (0.9")(Rv)(A)$$
$$= (0.9")(0.29)(45.1 \text{ ac})(1\text{ft}/12\text{in})$$
$$= 0.98 \text{ ac-ft}$$

Step 2. Recharge Volume, Rev

- Volume-based approach

$$Rev = (F)(A)(I)/12$$
$$\text{Composite F} = (.78)(.1) + (.22)(0) = 0.08 \text{ ft}$$

(Note: no recharge required for D soils which comprise 22% of site)

$$Rev = (0.08 \text{ ft})(45.1 \text{ ac})(.266)/12 = 0.08 \text{ acre-feet}$$

- Area-based approach

$$Rea = (F)(A)(I)$$
$$= (0.08)(45.1 \text{ ac})(.266) = 0.96 \text{ acres}$$

Step 3. Compute Channel Protection Volume, Cpv

In order to calculate the Cpv, the runoff from the post-development 1-year storm must first be calculated. Since a designer will ultimately need the discharge rates for the 10- and 100-year storms, the next step is to compute the hydrologic variables for all storms for pre-and post-development conditions. (Note the 1-year pre-development flow rate is not needed for Cpv). The Natural Resources Conservation Service (NRCS) model, “Urban Hydrology for Small Watersheds”—Technical Release 55 (1986) (hereafter referred to as TR-55) was used to compute runoff volumes and peak discharge for the 1-, 2-, 10-, and 100-year storms. Tables C.1, C.2, and C.3 illustrate pre-development, post-development, and ultimate development conditions, respectively. Ultimate conditions assume build-out of the off-site meadow at a density of one-quarter acre lots and would be used to design outlet spillways (both principal and emergency) for stormwater detention facilities.

PEAK DISCHARGE SUMMARY				
JOB:	COLE'S COLONY			EWB
DRAINAGE AREA NAME:	PRE DEVELOPMENT			30-Apr-01
COVER DESCRIPTION	SOIL NAME	GROUP A,B,C,D?	CN from TABLE 2-2	AREA (In acres)
MEADOW		C	71	20.25 Ac.
MEADOW		D	78	7.95 Ac.
WOOD		C	70	15.09 Ac.
WOOD		D	77	1.81 Ac.
OFF-SITE MEADOW		C	71	20.00 Ac.
		AREA SUBTOTALS:		65.10 Ac.
Time of Concentration 2-Yr 24 Hr Rainfall = 2.5 In	Surface Cover Cross Section	Manning 'n' Wetted Per	Flow Length Avg Velocity	Slope Tt (Hrs)
Sheet Flow	dense grass	'n'=0.24	150 Ft.	3.80% 0.29 Hrs
Shallow Flow	UNPAVED		1300 Ft. 2.65 F.P.S.	2.70% 0.14 Hrs.
Channel Flow Hydraulic Radius =1.26	22.0 SqFt	'n'=0.040 17.5 Ft.	1100 Ft. 7.14 F.P.S.	2.70% 0.04 Hrs.
Total Area in Acres =	65.10 Ac.	Total Sheet Flow=	Total Shallow Flow=	Total Channel Flow =
Weighted CN =	72			
Time Of Concentration =	0.47 Hrs.	0.29 Hrs.	0.14 Hrs.	0.04 Hrs.
Pond Factor =	1	RAINFALL TYPE II		
STORM	*Precipitation* (P) inches	*Runoff* (Q)	Qp, PEAK DISCHARGE	TOTAL STORM Volumes
1 Year	2.3 In.	0.4 In.	18.3 CFS	101,195 Cu. Ft.
2 Year	2.5 In.	0.5 In.	23.9 CFS	124,916 Cu. Ft.
10 Year	3.7 In.	1.3 In.	63 CFS	296,276 Cu. Ft.
100 Year	5.9 In.	2.9 In.	158 CFS	688,060 Cu. Ft.

Table C.1. Cole's Colony Pre-Development Conditions—TR-55 Output

PEAK DISCHARGE SUMMARY				
JOB:	COLE'S COLONY			EWB
DRAINAGE AREA NAME:	POST DEVELOPMENT			30-Apr-01
COVER DESCRIPTION	SOIL NAME	GROUP A,B,C,D?	CN from TABLE 2-2	AREA (In acres)
MEADOW		C	71	0.16 Ac.
MEADOW		D	78	0.14 Ac.
WOOD		C	70	3.09 Ac.
WOOD		D	77	1.81 Ac.
IMPERVIOUS			98	12.00 Ac.
GRASS		C	74	20.09 Ac.
GRASS		D	80	7.81 Ac.
OFFSITE MEADOW		C	71	20.00 Ac.
			AREA SUBTOTALS:	65.10 Ac.
Time of Concentration	Surface Cover	Manning 'n'	Flow Length	Slope
2-Yr 24 Hr Rainfall = 2.5 In	Cross Section	Wetted Per	Avg Velocity	Tt (Hrs)
Sheet Flow	dense grass	'n'=0.24	100 Ft.	3.80% 0.21 Hrs
Shallow Flow (a)	UNPAVED		100 Ft. 1.98 F.P.S.	1.50% 0.01 Hrs.
(b)	PAVED		400 Ft. 2.03 F.P.S.	1.00% 0.05 Hrs.
Channel Flow (a) Hydraulic Radius =0.50	1.6 SqFt	'n'=0.013 3.2 Ft.	1550 Ft. 7.22 F.P.S.	1.00% 0.06 Hrs.
(b) Hydraulic Radius =1.42	12.0 SqFt	'n'=0.030 8.5 Ft.	350 Ft. 13.01 F.P.S.	4.30% 0.01 Hrs.
(c) Hydraulic Radius =1.26	22.0 SqFt	'n'=0.040 8.5 Ft.	300 Ft. 7.89 F.P.S.	3.30% 0.01 Hrs.
Total Area in Acres =	65.10 Ac.	Total Sheet	Total Shallow	Total Channel
Weighted CN =	78	Flow=	Flow=	Flow =
Time Of Concentration =	0.35 Hrs.	0.21 Hrs.	0.07 Hrs.	0.08 Hrs.
Pond Factor =	1	RAINFALL TYPE II		
STORM	*Precipitation* (P) inches	*Runoff* (Q)	Qp, PEAK DISCHARGE	TOTAL STORM Volumes
1 Year	2.3 In.	0.7 In.	37.2 CFS	156,283 Cu. Ft.
2 Year	2.5 In.	0.8 In.	45.1 CFS	186,197 Cu. Ft.
10 Year	3.7 In.	1.7 In.	101 CFS	390,146 Cu. Ft.
100 Year	5.9 In.	3.5 In.	222 CFS	824,904 Cu. Ft.

Table C.2. Cole's Colony Post-Development Conditions—TR-55 Output

PEAK DISCHARGE SUMMARY				
JOB:	COLE'S COLONY			EWB
DRAINAGE AREA NAME:	ULTIMATE BUILDOUT			30-Apr-01
COVER DESCRIPTION	SOIL NAME	GROUP A,B,C,D?	CN from TABLE 2-2	AREA (In acres)
MEADOW		C	71	0.16 Ac.
MEADOW		D	78	0.14 Ac.
WOOD		C	70	3.09 Ac.
WOOD		D	77	1.81 Ac.
IMPERVIOUS			98	12.00 Ac.
GRASS		C	74	20.09 Ac.
GRASS		D	80	7.81 Ac.
OFFSITE ULTIMATE				
SF RES (0.25 AC LOTS)		C	83	20.00 Ac.
			AREA SUBTOTALS:	65.10 Ac.

Time of Concentration 2-Yr 24 Hr Rainfall = 2.5 In	Surface Cover Cross Section	Manning 'n' Wetted Per	Flow Length Avg Velocity	Slope Tt (Hrs)
Sheet Flow	dense grass	'n'=0.24	100 Ft.	3.80% 0.21 Hrs
Shallow Flow (a)	UNPAVED		100 Ft. 1.98 F.P.S.	1.50% 0.01 Hrs.
(b)	PAVED		400 Ft. 2.03 F.P.S.	1.00% 0.05 Hrs.
Channel Flow (a) Hydraulic Radius =0.50	1.6 SqFt	'n'=0.013 3.2 Ft.	1550 Ft. 7.22 F.P.S.	1.00% 0.06 Hrs.
(b) Hydraulic Radius =1.42	12.0 SqFt	'n'=0.030 8.5 Ft.	350 Ft. 13.01 F.P.S.	4.30% 0.01 Hrs.
(c) Hydraulic Radius =1.26	22.0 SqFt	'n'=0.040 8.5 Ft.	300 Ft. 7.89 F.P.S.	3.30% 0.01 Hrs.
Total Area in Acres =	65.10 Ac.	Total Sheet Flow= 0.21 Hrs.	Total Shallow Flow= 0.07 Hrs.	Total Channel Flow = 0.08 Hrs.
Weighted CN =	82			
Time Of Concentration =	0.35 Hrs.			
Pond Factor =	1	RAINFALL TYPE II		

STORM	*Precipitation* (P) inches	*Runoff* (Q)	Qp, PEAK DISCHARGE	TOTAL STORM Volumes
1 Year	2.3 In.	0.9 In.	50.3 CFS	201,772 Cu. Ft.
2 Year	2.5 In.	1.0 In.	59.6 CFS	235,842 Cu. Ft.
10 Year	3.7 In.	1.9 In.	122 CFS	460,575 Cu. Ft.
100 Year	5.9 In.	3.9 In.	248 CFS	920,493 Cu. Ft.

Table C.3. Cole's Colony Ultimate Buildout Conditions—TR-55 Output

Summary of Hydrologic Input Parameters and Calculations

Condition	Area	CN	tc
	Ac		hrs
Pre-developed	65.1	72	0.46
Post-developed	65.1	78	0.35
Ultimate buildout	65.1	82	0.35

Condition	Q1-yr	Q1-yr	Q10-yr	Q100-yr
Runoff	Inches	cfs	cfs	cfs
Pre-developed	0.4	18	63	158
Post-developed	0.7	37	101	222
Ultimate buildout	NA	NA	NA	248

For stream channel protection, provide 24 hours of extended detention (T = 24) for the 1-year event (See methodology in Appendix D6).

Utilize NRCS approach to Compute Channel Protection Storage Volume (from TR-55 Chapter 4).

- Initial abstraction (Ia) for CN of 78 is 0.564: [Ia = (200/CN - 2)]
- Ia/P = (0.564)/ 2.3 inches = 0.245 (P = 2.3 inches for Brandon—see Table 1.2 of the *VT Stormwater Management Manual – Volume I*)
- tc = 0.35 hours
- qu = 570 csm/in (Type II Storm) (Exhibit 4-11 in TR-55 and also in Appendix D6)

Knowing qu and detention time, T (assume 24 hrs of extended detention time), find qo/qi using Figure C.2 (adopted from Harrington, 1987).

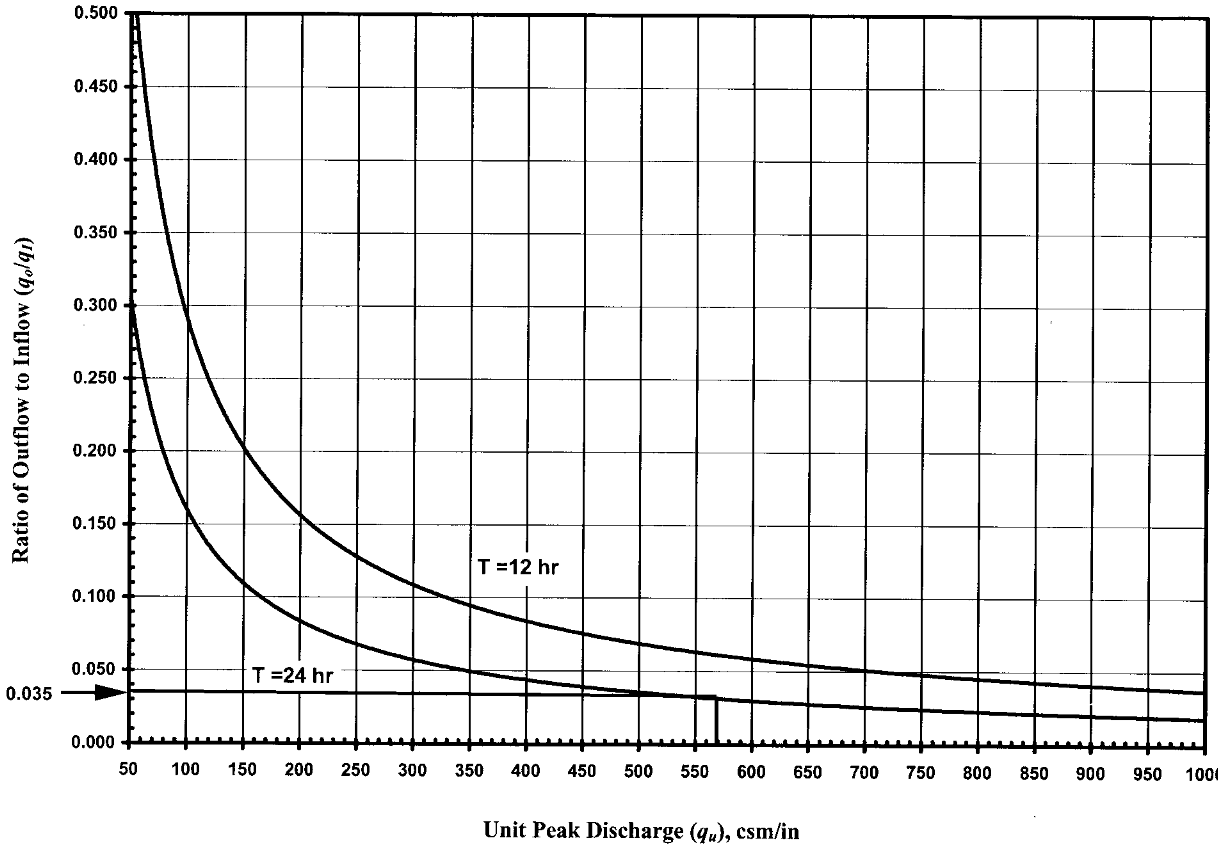


Figure C.2. Detention Time vs. Discharge Ration (Source: adopted from Harrington, 1987)

- Peak outflow discharge/peak inflow discharge (q_o/q_i) = 0.035
- Vs/Vr = 0.682 - 1.43(q_o/q_i) +1.64(q_o/q_i) ^{2} - 0.804(q_o/q_i) ^{3} (From TR-55, Appendix F—equation for Figure 6.1)

  Where Vs equals volume of storage; in this case, channel protection storage (Cp_v) and Vr equals the volume of runoff from the 1-year storm in inches (see column labeled "Runoff (Q)" at bottom of Table C.2).

- Vs/Vr = 0.63
- Therefore, Vs = Cp_v = 0.63(0.7")(1 ft/12")(65.1 ac) = 2.4 ac-ft (104,214 cubic feet)

<u>Define the average ED Release Rate</u>

- The above volume, 2.4 ac-ft, is to be released over 24 hours.
- (2.4 ac-ft × 43,560 ft^2/ac) / (24 hrs × 3,600 sec/hr) = 1.2 cfs

Step 4. Compute Overbank Flood Protection Volume, (Q_{p10})

For both the overbank flood protection volume and the extreme flood protection volume, the size is determined using the TR-55 "Short-Cut Method" (TR-55, Chapter 6), which relates the storage volume to the required reduction in peak flow and storm inflow volume (Figure C.3).

- For a q_i of 101 cfs (post-developed—see Table C.2), and an allowable q_o of 63 cfs (pre-developed—see Table C.1), the value of $(q_o)/(q_i)$ is 0.62
- Using Figure C.3, and based on a Type II rainfall distribution, the value of Vs/Vr is 0.24
- Using a runoff volume of 1.7 inches, or 390,146 cubic feet (see Table C.2 "Runoff (Q)" for 10-year), the required storage (Vs) is (0.24 x 390,140 ft^3)/(43,560 ft^2/ac) = 2.15 acre-feet

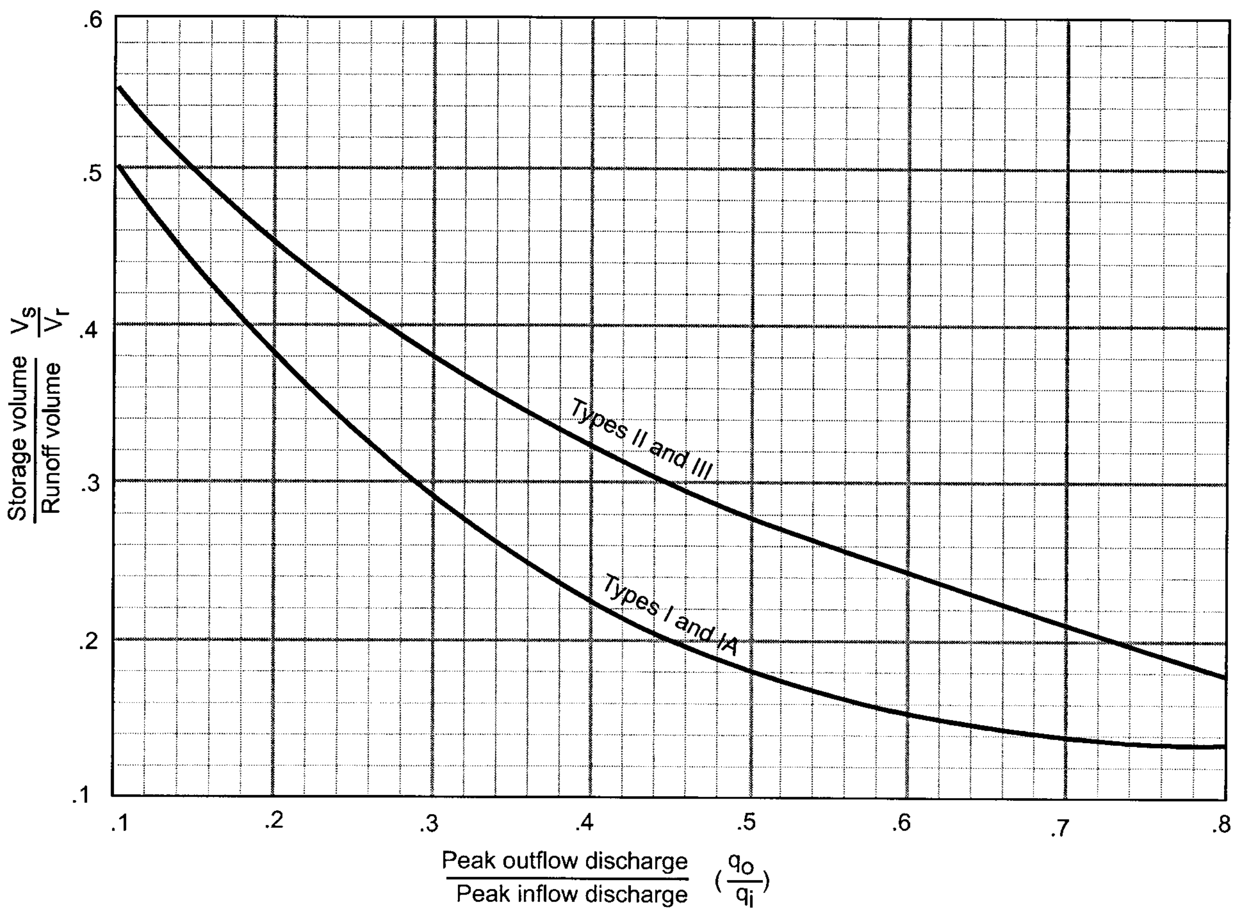


Figure C.3. Approximate Detention Basin Routing For Rainfall Types I, IA, II, and III. Source: NRCS, 1986

While the TR-55 short-cut method reports to incorporate multiple stage structures, experience has shown that an additional 10-15% storage is required when multiple levels of extended detention are provided inclusive with the 10-year storm. So, for preliminary sizing purposes, add 15% to the required volume for the 10-year storm. Q_{p-10} = 2.15 ac-ft × 1.15 = 2.47 ac-ft.

Step 5. Compute Extreme Flood Protection Volume, (Q_{p100})

Extreme flood protection is calculated using the same methodology as overbank protection.

- For a q_i of 222 cfs (post-developed—see Table C.2), and an allowable q_o of 158 cfs (pre-developed—see Table C.1), the value of $(q_o)/(q_i)$ is 0.71
- Using Figure C.3, and based on a Type II rainfall distribution, the value of Vs/Vr is 0.205
- Using a runoff volume of 3.5 inches, or 824,904 cubic feet (see Table C.2 "Runoff (Q)" for 100-year), the required storage (Vs) is (0.205 x 824,904 ft^3)/(43,560 ft^2/ac) = 3.9 acre-feet
- While the TR-55 short-cut method reports to incorporate multiple stage structures, experience has shown that an additional 10-15% storage is required when multiple levels of extended detention are provided inclusive with the 100-year storm. So, for preliminary sizing purposes add 15% to the required volume for the 100-year storm. Q_{p100} = 3.9 ac-ft × 1.15 = 4.5 ac-ft.

Step 6. Analyze Safe Passage of 100-Year Design Storm, (Q_{p100})

Safe passage for the 100-year event under ultimate buildout conditions requires passing Q_{ult} (248 cfs—see Table C.3 for 100-year) through the facility. The storage required will depend on the capacity of the spillway system (i.e., if the system is designed to pass 248 cfs, no additional storage would be required).

Table C.4 provides a summary of the general storage requirements for the Cole's Colony sizing example.

Table C.4. Summary of General Storage Requirements for Cole's Colony

Symbol	Category	Volume Required (ac-ft)	Notes
WQ_v	Water Quality Volume	0.98	Inclusive of Re_v
Re_v	Recharge Volume	0.08	Area-based approach requires 0.96 acres treated by nonstructural practices
Cp_v	Stream Protection	2.4	Average ED release rate is 1.2 cfs over 24 hours
Q_{p10}	Overbank Control	2.5	10-year control
Q_{p100}	Extreme Flood Control	4.5	100-year attenuation, safe passage of Q_{ult} = 248 cfs

Step 7. Compute preliminary runoff control volumes.

Assume that the site design employs both rooftop and non-rooftop disconnection credits (see Section 3 of Volume I) to reduce the water quality volume requirement. These credits also are used to meet the recharge criteria using the Percent Area Method (Re_a). In addition, because the water quality volume is inclusive of the recharge volume, the new required water quality volume can be further reduced by the recharge volume amount (Re_v). The new required water quality volume is computed by the following calculation:

- The connected imperviousness is reduced by 0.96 acres from approximately 12 acres to 11 acres.
- New site imperviousness is 11 ac ÷ 45.1 ac = 24.4%.
- New R_v = 0.05 + (24.4)(0.009) = 0.27
- New WQ_v = [(0.9")(0.27)(45.1 ac)/12] – [0.08 ac-ft] = 0.91 – 0.08 = 0.83 acre-feet, or a 0.15 acre-foot reduction

Step 8. Determine if the development site and conditions are appropriate for the use of a stormwater pond.

The drainage area to the pond is 65.1 acres. Existing ground at the proposed pond outlet is 619 MSL. Soil boring observations reveal that the seasonally high water table is at elevation 614. The underlying soils are SC (sandy clay) and are suitable for earthen embankments and to support a wet pond without a liner. The stream invert at the adjacent stream is at elevation 612.

Step 9. Confirm site-specific design criteria and applicability

Assume the site drains to a warm water fishery; therefore, no cold water limitations are in place. For the purposes of this design example, assume onsite control of the 10- and 100-year storms is also necessary.

Step 10. Determine pretreatment volume

Size wet forebay to treat 0.1"/impervious acre. (11.0 ac) (0.1") (1'/12") = **0.09 ac-ft** (forebay volume is included in the WQ_v as part of permanent pool volume)

Step 11. Determine permanent pool volume

Size the permanent pool volume to contain 100% of WQ_v (see Step 1 for derivation of new WQ_v, which accounts for credits and the fact that WQ_v is inclusive of the recharge volume): = **0.83 ac-ft** (includes 0.09 ac-ft of forebay volume)

Step 12. Determine pond location and preliminary geometry. Conduct pond grading and determine storage available for WQv permanent pool

This step involves initially grading the pond (establishing contours) and determining the elevation-storage relationship for the pond. Storage must be provided for the permanent pool (including sediment forebay), Cp_v, 10-year storm, 100-year storm, plus sufficient additional elevation and/or storage to pass the ultimate condition 100-year storm with required freeboard. An elevation-storage table and curve is prepared using the average area method for computing volumes. See Figure C.4 for pond location on site, Figure C.5 for grading and Figure C.6 for Elevation-Storage Data.

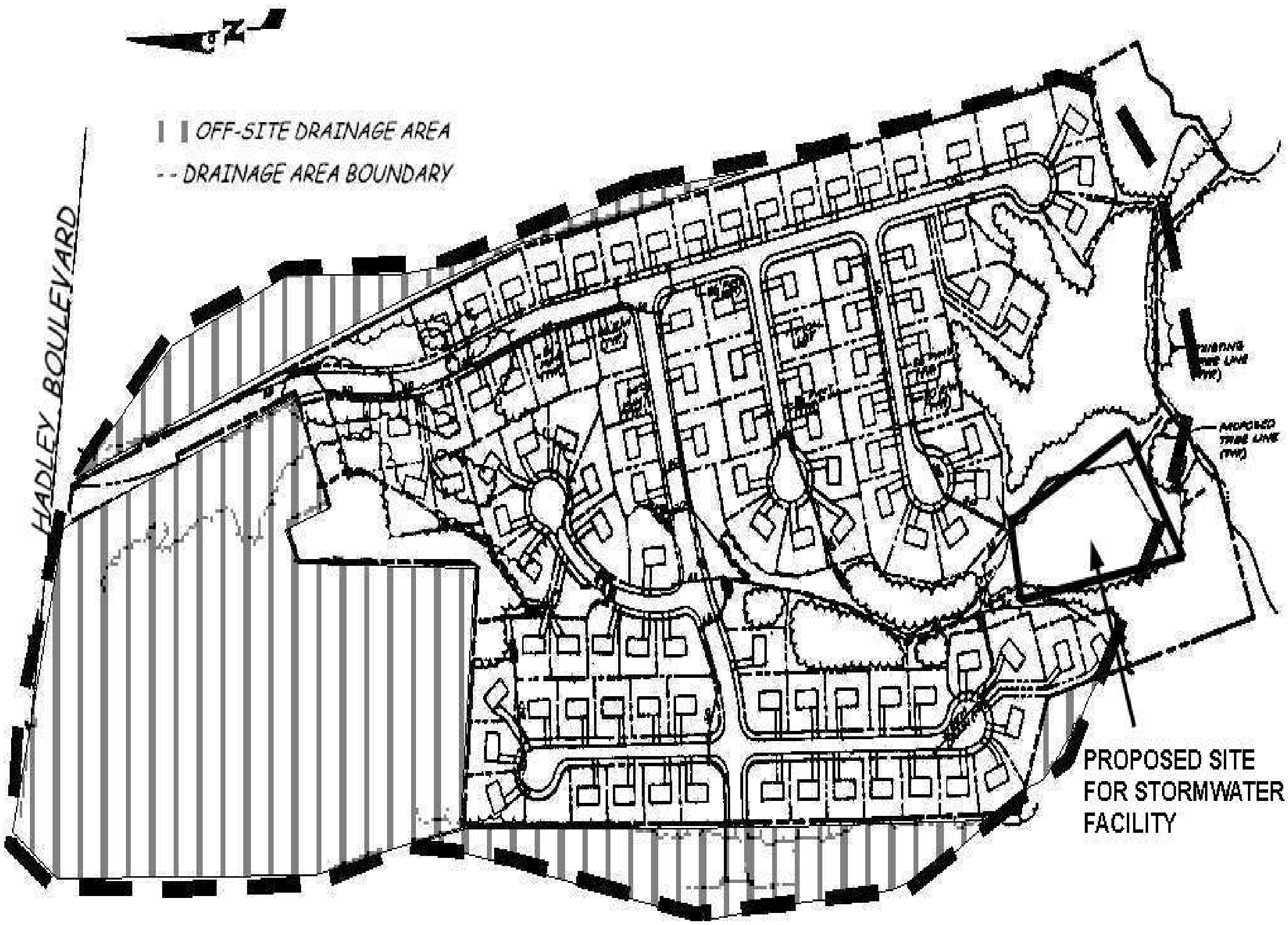


Figure C.4. Pond Location on Site

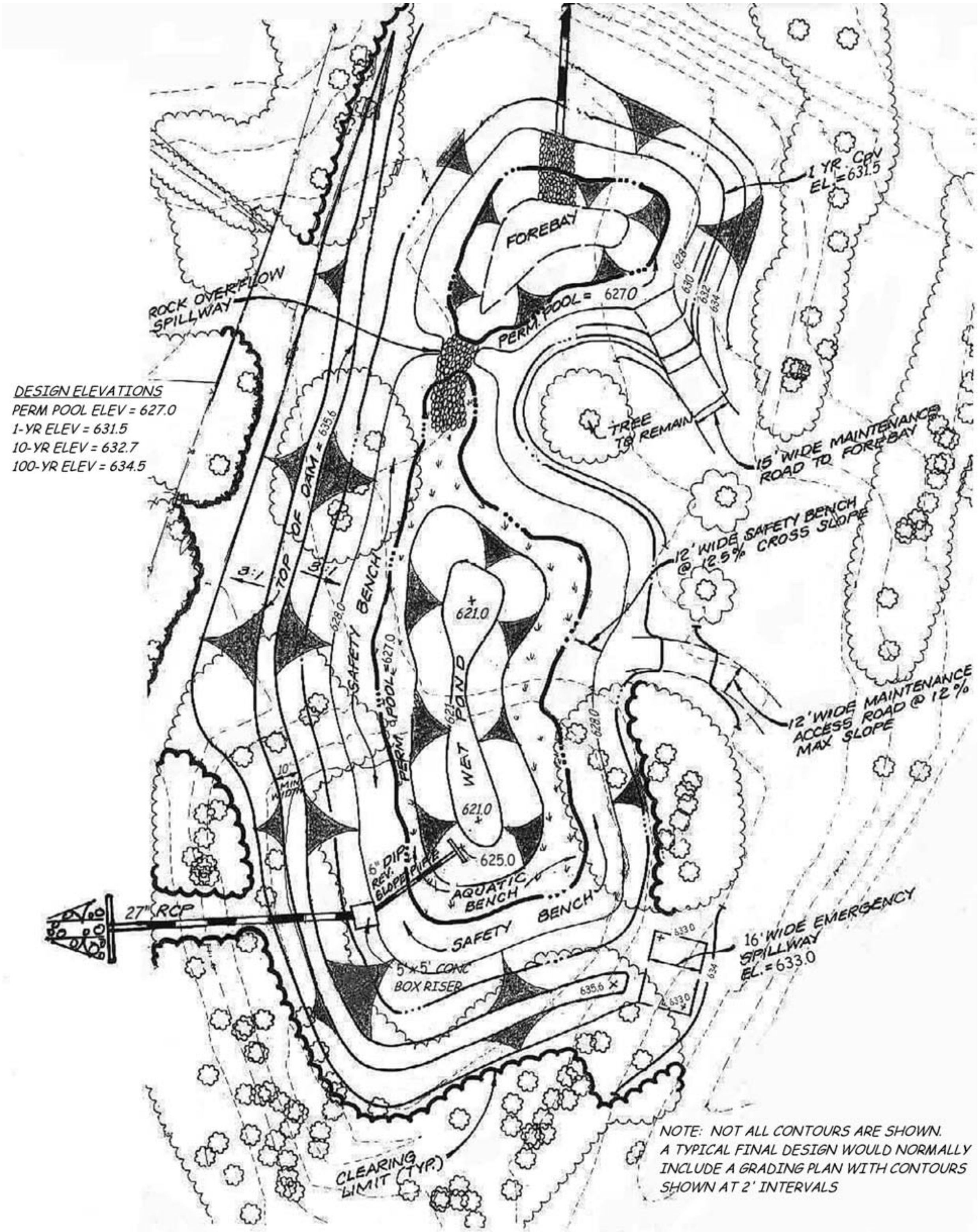


Figure C.5. Plan View of Pond Grading (Not to Scale)

Elevation MSL	Area ft^2	Average Area ft^2	Depth ft	Volume ft^3	Cumulative Volume ft^3	Cumulative Volume ac-ft	Volume Above Permanent Pool ac-ft
621.0	3150						
624.0	8325	5738	3	17213	17213	0.40	
625.0	10400	9363	1	9363	26575	0.61	
627.0	13850	12125	2	24250	50825	1.17	0.00
628.0	21850	17850	1	17850	68675	1.58	0.41
630.0	26350	24100	2	48200	116875	2.68	1.52
632.0	30475	28413	2	56825	173700	3.99	2.82
634.0	57685	44080	2	88160	261860	6.01	4.84
635.0	60125	58905	1	58905	320765	7.36	6.20

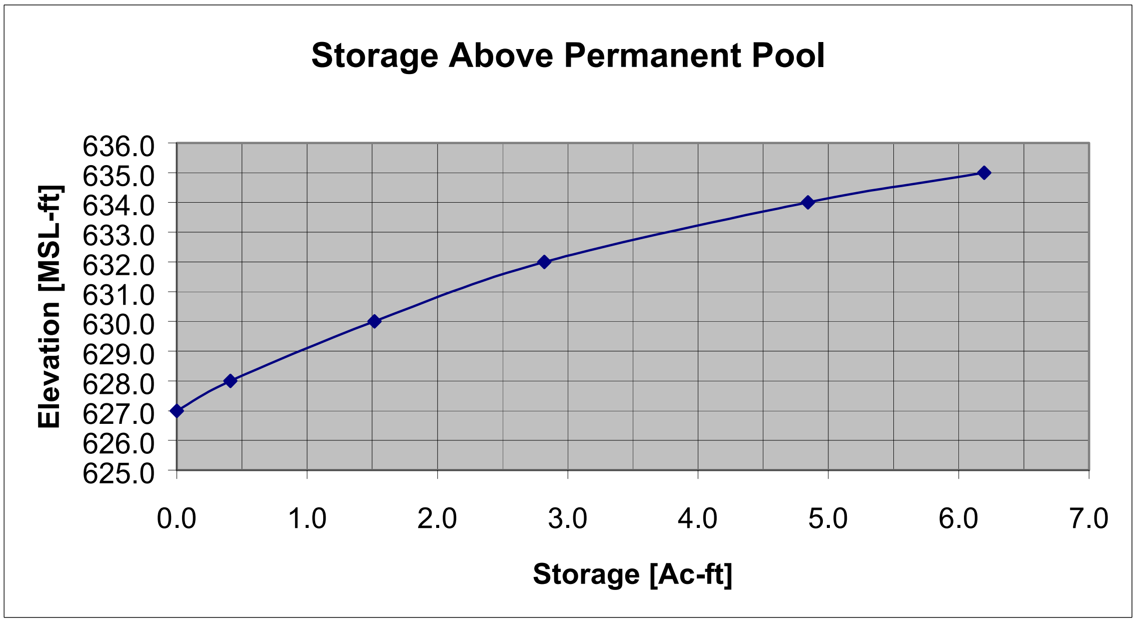


Figure C.6. Storage-Elevation Table/Curve

Set basic elevations for pond structures

- The pond bottom is set at elevation 621.0.
- Provide gravity flow to allow for pond drain, set riser invert at 620.5.
- Set barrel outlet elevation at 620.0.

Set permanent pool water surface elevation

- Required permanent pool volume = 100% of WQv = 0.83 ac-ft. From the elevation-storage table, read elevation 627.0 (1.17 ac-ft > 0.83 ac-ft) site can accommodate it and it allows for a safety factor for fine sediment accumulation – OK
- Set permanent pool wsel = 627.0

- Forebay volume provided in single pool with volume = 0.1 ac-ft (≥0.09 – OK)

Step 13. Compute release rate for Cp_v control and establish Cp_v elevation

This methodology assumes that required Cp_v is delivered instantaneously to the pond. Actual pond routing will likely yield a slightly lower water surface elevation (wsel) for Cp_v (see Table C.7).

Set the Cp_v pool elevation.

- Required Cp_v storage = 2.4 ac-ft (see Table C.2).
- From the elevation-storage table, read elevation 631.5.
- Set Cp_v wsel = 631.5

Size Cp_v orifice.

- Size to release average of 1.2 cfs (Cp_v rate)
  - Set invert of orifice at permanent pool wsel = 627.0
  - Approximate average head = (631.5 - 627.0)/2 = 2.25'

Use orifice equation to compute cross-sectional area and diameter.

- $Q = CA(2gh)^{0.5}$, for h = 2.25'
  - $A = 1.2 \text{ cfs} / [(0.6)((2)(32.2'/s^2)(2.25'))^{0.5}]$
  - $A = 0.17 \text{ ft}^2$, $A = \pi d^2 / 4$;
  - dia. = 0.46 ft = 5.5"
  - Use 6" pipe with 6" gate valve closed down approximately 8% to achieve equivalent diameter

Compute the stage-discharge equation for the 5.5" dia. Cp_v orifice.

- $Q_{Cpv} = CA(2gh)^{0.5} = (0.6)\ (0.17 \text{ ft}^2)\ [((2)\ (32.2'/s^2))^{0.5}]\ (h^{0.5})$,
- $Q_{Cpv} = (0.82)\ (h^{0.5})$, where: h = wsel – 627.23

(Note: account for one half of orifice diameter when calculating head – invert = 627.0 + d/2 = 627 + 5.5"/2 * 12 = 627.23)

Step 14. Calculate Q_{p10} (10-year storm) release rate and water surface elevation

In order to calculate the 10-year release rate and water surface elevation, the designer must set up a stage-storage-discharge relationship for the control structure for the Cp_v orifice plus the 10-year storm.

Develop basic data and information

- The 10-year pre-developed peak discharge = 63 cfs
- The post developed inflow = 101 cfs
- From previous estimate $Q_{p\text{-}10}$ = 2.15 ac-ft. Adding 15% to account for ED storage yields a preliminary volume of 2.47 ac-ft, say 2.5 ac-ft.

  Note: this is used as a starting point to compute the required head to size the release structure. The actual elevation is established by routing using the storage indication method (common option in many hydrologic computer models such as TR-20).

- From elevation-storage table (Figure C.6), read elevation 631.7, say 632.0.

Size 10-year slot to release 63 cfs at a water surface elevation of 632.0. At wsel 632.0:

- Cp_v orifice releases 1.8 cfs $[(0.82)(632 - 627.23)^{0.5}]$, therefore
- Allowable $Q_{p\text{-}10}$ = 63 cfs - 1.8 = 61.2 cfs.
- Set weir crest elevation at Cp_v wsel = 631.5 (this is max Cp_v elevation)
- Max head = (632.0 – 631.5) = 0.5’
- Use weir equation to compute slot length $\rightarrow Q = CLh^{3/2}$ (use C = 3.1)
- $L = 61.2 \text{ cfs} / (3.1)(0.5^{3/2}) = 55.8$ ft

This weir length is impractical, so adjust max head (and therefore slot height) to 1.5’ and recalculate weir length.

$L = 61.2 \text{ cfs} / (3.1)(1.5^{3/2}) = 10.75$ ft

<u>Use three 4ft x 1.5 ft slots for 10-year release</u> (opening should be slightly larger than needed so as to have the barrel control before slot goes from weir flow to orifice flow—this is done to avoid cavitation within the riser structure).

Maximum $Q = (3.1)(12)(1.5)^{3/2} = 68.3$ cfs

Check orifice equation using cross-sectional area of opening

- $Q = CA(2gh)^{0.5}$, for h = 0.75’ (For orifice equation, h is from midpoint of slot)
- $A = 3\,(4.0')\,(1.5') = 18.0\text{ft}^2$
- $Q = 0.6\,(18.0\text{ft}^2)\,[(64.4)(0.75)]^{0.5} = 75$ cfs > 68.3 cfs, so weir equation is the controlling equation at this elevation.
- $Q_{10} = (3.1)(12')\,h^{3/2}$, <u>$Q_{10} = (37.2)\,h^{3/2}$, where h = wsel – 631.5</u>

Size barrel to release approximately 63 cfs at elevation 633.0 (i.e., Cp_v elevation of 631.5 + 1.5, from above)

- Check inlet condition: (use FHWA culvert charts or similar hydraulic model)
- H_w = 633.0-620.5 = 12.5 ft
- Try 27" diameter RCP, Using FHWA Chart (“Headwater Depth for Concrete Pipe Culverts with Inlet Control”) with entrance condition 1 (see Figure C.7).
- $H_w / D = 12.5/2.25 = 5.56$, Discharge = 63 cfs

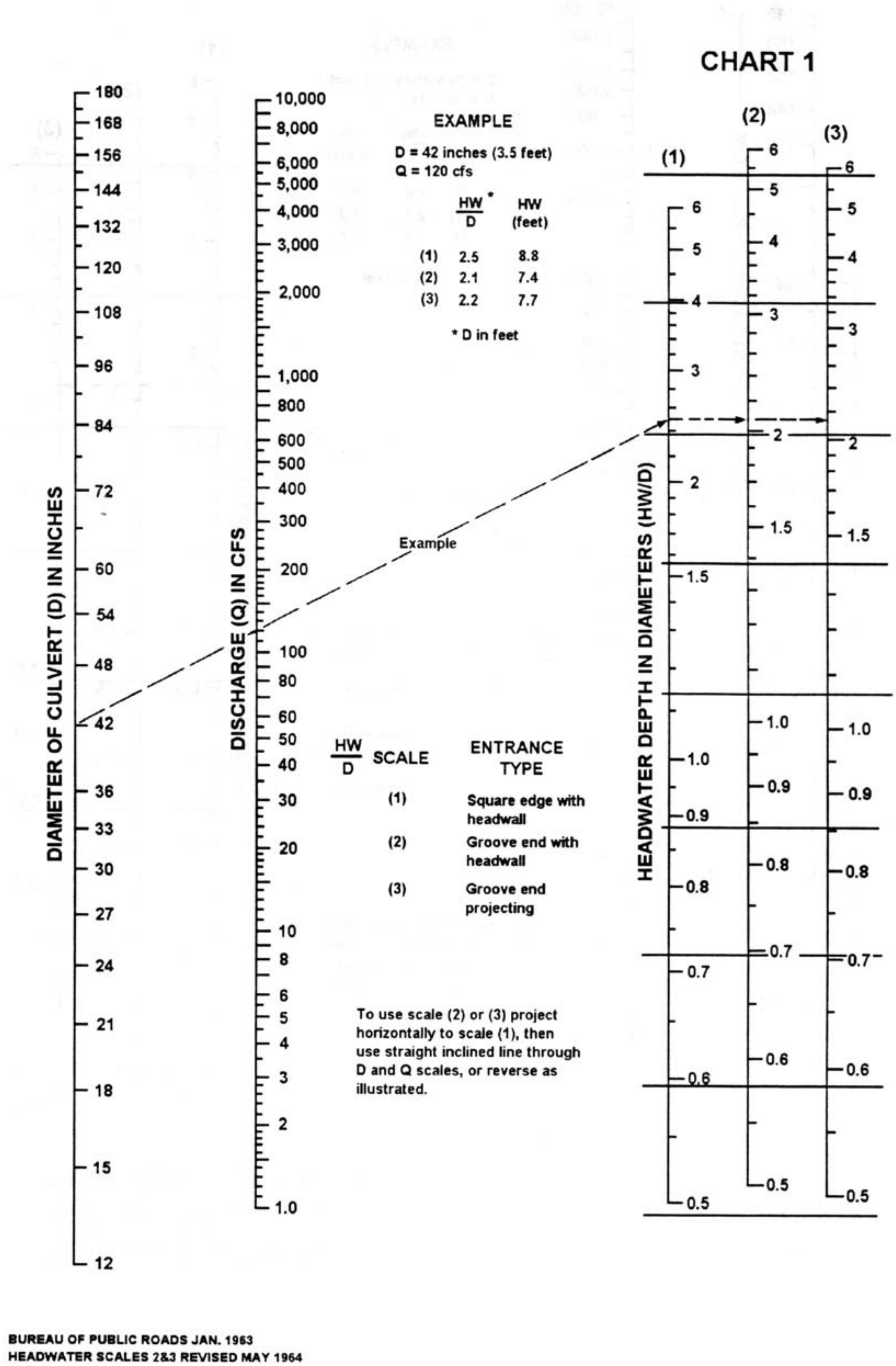


Figure C.7. Headwater Depth for Concrete Pipe Culverts with Inlet Control

- Check outlet condition (use NRCS pipe flow equation from NEH Section 5 ES-42):
- $Q = a\,[(2gh)/(1+k_m+k_pL)]^{0.5}$

where:
Q = discharge in cfs
a = pipe cross sectional area in ft^2
g = acceleration of gravity in ft/sec^2
h = head differential (wsel - downstream centerline of pipe or tailwater elevation, whichever is greater)
k_m = coefficient of minor losses (use 1.0)
k_p = pipe friction loss coef. (= $5087n^2/d^{4/3}$, d in inches, n is Manning's n)
L = pipe length in ft

h = 633.0 - (620.0 + 1.125) = 11.88'
for 27" RCP, approximately 70 feet long:
$Q = 4.0\,[(64.4)\,(11.88)\,/\,1+1+(0.0106)\,(70)]^{0.5}$ = 66.8 cfs
63.0 cfs < 66.8 cfs, so barrel is in inlet control

Note: barrel will control flow before high stage inlet reaches max head.

Complete stage-storage-discharge summary (Table C.5) up to preliminary 10-year wsel (633.0) and route 10-year post-developed condition inflow using computer software (e.g., TR-20).

- Pond routing computes 10-year wsel at 632.7 with discharge = 46.6 cfs < 63 cfs, OK (see Table C.6 and C.7 for TR-20 input and output files, respectively).

Table C.5. Stage-Storage-Discharge Summary

Elevation MSL	Storage ac-ft	Riser						27" Barrel				Emergency Spillway 16' earthen 3:1		Total Discharge
		Cpv-ED 5.5" eq. dia		High Stage Slot				Inlet		Pipe				
				Orifice		Weir								
		H ft	Q cfs	H ft	Q cfs	H ft	Q cfs	H ft	Q cfs	H ft	Q cfs	H ft	Q cfs	Q cfs
627.0	0.00	0.0	0.00											0.00
628.0	0.41	0.8	0.72											0.72
629.0	0.90	1.8	1.09											1.09
630.0	1.52	2.8	1.36											1.36
631.0	2.10	3.8	1.59											1.59
631.5	2.40	4.3	1.69	-	-	0.0	0.0							1.69
632.0	2.82	4.8	1.79	-	-	0.5	13.2							14.94
633.0	3.70	5.8	1.97	0.75	75	1.5	68.3	12.5	63.0	11.9	66.8	0.0	0.0	63.00
634.0	4.84	6.8	2.13	1.25	97	-	-	13.5	66.0	12.9	69.6	1.0	38.0	104.0
635.0	6.20	7.8	2.29	1.75	115	-	-	14.5	71.0	13.9	72.2	2.0	133.0	204.0

Note: Adequate outfall protection must be provided in the form of a riprap channel, plunge pool, or combination to ensure non-erosive velocities.

Table C.6. TR-20 Model Input

```
*****************80-80 LIST OF INPUT DATA FOR TR-20 HYDROLOGY*****************

JOB TR-20                    FULLPRINT                      NOPLOTS
TITLE      Vermont Manual Wet Pond Example 5/01          EWB
TITLE      Post Developed Conditions Routing for 1, 10, and 100
 3 STRUCT       1
 8                    627.0        0.0          0.0
 8                    628.0        0.72         0.41
 8                    629.0        1.09         0.90
 8                    630.0        1.36         1.52
 8                    631.0        1.59         2.10
 8                    631.5        1.69         2.40
 8                    632.0        14.94        2.82
 8                    633.0        63.00        3.70
 8                    634.0        104.0        4.84
 8                    635.0        204.0        6.20
 9 ENDTBL
 6 RUNOFF 1     1     2 0.102      78.0         0.35          1 1   0 0 1
 6 RESVOR 2     1 2   3 627.0                                 1 1       1
   ENDATA
 7 INCREM 6            0.1
 7 COMPUT 7     1     1 0.0        2.3          1.0           2 2   1  01
   ENDCMP 1
 7 COMPUT 7     1     1 0.0        3.7          1.0           2 2   1  10
   ENDCMP 1
 7 COMPUT 7     1     1 0.0        5.9          1.0           2 2   1  99
   ENDCMP 1
   ENDJOB 2

******************************END OF 80-80 LIST********************************
```

Table C.7. TR-20 Model Output

```
TR20 XEQ  4/16/**            Vermont Handbook Wet Pond Example 5/01        EWB                      JOB  1   SUMMARY
     REV 09/01/83            Post Developed Conditions Routing for 1, 10, and 100                            PAGE   8
```

SUMMARY TABLE 1 - SELECTED RESULTS OF STANDARD AND EXECUTIVE CONTROL INSTRUCTIONS IN THE ORDER PERFORMED
(A STAR(*) AFTER THE PEAK DISCHARGE TIME AND RATE (CFS) VALUES INDICATES A FLAT TOP HYDROGRAPH
A QUESTION MARK(?) INDICATES A HYDROGRAPH WITH PEAK AS LAST POINT.)

SECTION/ STRUCTURE ID	STANDARD CONTROL OPERATION	DRAINAGE AREA (SQ MI)	RAIN TABLE #	ANTEC MOIST COND	MAIN TIME INCREM (HR)	PRECIPITATION			RUNOFF AMOUNT (IN)	PEAK DISCHARGE			
						BEGIN (HR)	AMOUNT (IN)	DURATION (HR)		ELEVATION (FT)	TIME (HR)	RATE (CFS)	RATE (CSM)
ALTERNATE 1 STORM 1													
STRUCTURE 1	RUNOFF	.10	2	2	.10	.0	2.30	24.00	.66	---	12.13	40.62	398.2
STRUCTURE 1	RESVOR	.10	2	2	.10	.0	2.30	24.00	.41	631.18	18.00?	1.63?	15.9
ALTERNATE 1 STORM 10													
STRUCTURE 1	RUNOFF	.10	2	2	.10	.0	3.70	24.00	1.65	---	12.11	107.96	1058.4
STRUCTURE 1	RESVOR	.10	2	2	.10	.0	3.70	24.00	1.34	632.66	12.41	46.63	457.1
ALTERNATE 1 STORM 99													
STRUCTURE 1	RUNOFF	.10	2	2	.10	.0	5.90	24.00	3.49	---	12.11	229.32	2248.3
STRUCTURE 1	RESVOR	.10	2	2	.10	.0	5.90	24.00	3.16	634.53	12.28	156.56	1534.9

Step 15. Calculate Q_{p100} (100-year storm) release rate and water surface elevation, size emergency spillway, calculate 100-year water surface elevation

In order to calculate the 100-year release rate and water surface elevation, the designer must continue with the stage-storage-discharge relationship (Table C.5) for the control riser and emergency spillway.

Develop basic data and information

- The 100-year pre-developed peak discharge = 158 cfs,
- The post developed inflow = 222 cfs, from Table C.2,
- From previous estimate Q_{p-100} = 3.9 ac-ft. Adding 15% to account for ED storage yields a preliminary volume of 4.5 ac-ft (this is used as a starting point).
- From elevation-storage table (Figure C.6), read elevation 633.8, say 634.0.

The 10-year wsel is at 632.7 (see TR-20 output in Table C.7). Set the emergency spillway invert at elevation 633.0 (this allows for some additional storage above the 10-yr wsel) and use design information and criteria for earth spillways (not included in this manual, but can be found in the NRCS Engineering Field Manual, 1984).

- Size the pond outlet spillways to release 158 cfs (this is usually accomplished through a combination of the principle and emergency spillways).
- Using the preliminary wsel of 634.0, determine the discharge from the principle spillway (approximately 70 cfs from Table C.5). Emergency spillway is then sized based on the difference between the peak discharge and the principle spillway discharge (158 cfs – 70 cfs = 88 cfs).

Note: The process of sizing the emergency spillway and storage volume determination is usually iterative.

- For this example, the iterative approach results in a 16' wide vegetated emergency spillway with 3:1 side slopes.
- Finalize stage-storage-discharge relationships and perform pond routing

Pond routing (TR-20) computes 100-year wsel at 634.53 with discharge = 156.6 cfs < 158 cfs, OK (see Table C.7).

Step 16. Check for safe passage of Qp_{100} under ultimate buildout conditions and set top of embankment elevation

The safety design of the pond embankment requires that the 100-year discharge, based on ultimate buildout conditions be able to pass safely through the emergency spillway with sufficient freeboard (1 foot). This criteria <u>does not</u> mean that the ultimate buildout peak discharge be attenuated to pre-development rates, but must simply pass safely through the facility.

From previous hydrologic modeling:

- The 100 year ultimate buildout peak discharge = 248 cfs,
- The ultimate buildout composite curve number is 82.

Using TR-20 or equivalent routing model, determine peak wsel. Pond routing computes 100-year wsel at 634.6 with discharge = 162 cfs (Table C.8).

Therefore, with 1 foot of freeboard, the minimum embankment elevation is 635.6.

Table C.8. TR-20 Model Output for Ultimate Buildout Conditions

TR20 XEQ 4/16/** Vermont Handbook Wet Pond Example 5/01 EWB JOB 1 SUMMARY
REV 09/01/83 Post Developed Conditions Ultimate 100 PAGE 4

SUMMARY TABLE 1 - SELECTED RESULTS OF STANDARD AND EXECUTIVE CONTROL INSTRUCTIONS IN THE ORDER PERFORMED
(A STAR(*) AFTER THE PEAK DISCHARGE TIME AND RATE (CFS) VALUES INDICATES A FLAT TOP HYDROGRAPH
A QUESTION MARK(?) INDICATES A HYDROGRAPH WITH PEAK AS LAST POINT.)

SECTION/ STRUCTURE ID	STANDARD CONTROL OPERATION	DRAINAGE AREA (SQ MI)	RAIN TABLE #	ANTEC MOIST COND	MAIN TIME INCREM (HR)	PRECIPITATION BEGIN (HR)	AMOUNT (IN)	DURATION (HR)	RUNOFF AMOUNT (IN)	PEAK DISCHARGE ELEVATION (FT)	TIME (HR)	RATE (CFS)	RATE (CSM)
ALTERNATE 1 STORM 99													
STRUCTURE 1	RUNOFF	.10	2	2	.10	.0	5.50	24.00	3.53	---	12.10	230.71	2261.9
STRUCTURE 1	RESVOR	.10	2	2	.10	.0	5.50	24.00	3.20	634.58	12.26	162.13	1589.5

Other Computations Often Necessary for Pond Design (not included in this example)

This design example limits its focus on the basic steps necessary to size a stormwater pond facility. However, it is important to note that there are several other analyses that may be required for final design requirements. These include, but are not limited to:

- Outlet channel sizing and protection
- Water balance calculations
- Permanent pool drain pipe sizing (design to drain within a 24-hour period)
- Seepage control sizing (e.g., sand filter diaphragm)
- Dam breach analysis
- Geotechnical investigation and report
- Structural concrete design for riser, endwalls (if applicable), and cradle
- Slope stability analysis, underdrain or toe drain piping
- Anti-flotation computations for riser
- Inflow channel(s) to pond
- Detailed construction cost estimate

Table C.9 provides a summary of the storage, stage, and discharge relationships determined for this design example.

Table C.9. Summary of Controls Provided

Control	Type/Size	Storage Provided (Acre-feet)		Elevation	Discharge (cfs)		Remarks
		Req'd	Provided		Req'd	Provided	
Permanent Pool		0.83	1.17	627.0	0	0	part of WQ_v
Forebay	submerged berm	0.09	0.1	627.0	0	0	included in permanent pool vol.
Channel Protection (Cp_v)	6" pipe sized to 5.5" equivalent diameter	2.4	2.3	631.2	1.2	0.8 *	volume above perm. pool, discharge is average release rate over 24 hours
Overbank Protection (Q_{p-10})	Three 4' x 1.5' slots on a 5' x 5' riser, 27" barrel.	2.5	3.3	632.7	63.0	46.6	volume above perm. pool
Extreme Storm (Q_{p-100})	16' wide earth spillway	4.5	5.5	634.5	158.0	156.6	volume above perm. pool
Extreme Storm Ultimate Buildout	16' wide earth spillway	NA	5.6	634.6	NA	162.0	Set minimum embankment height at 635.6
* Estimated from TR-20 output as one half the peak discharge associated with the 1-yr storm (see Table C.7).							

Figure C.8 provides a schematic of the riser.

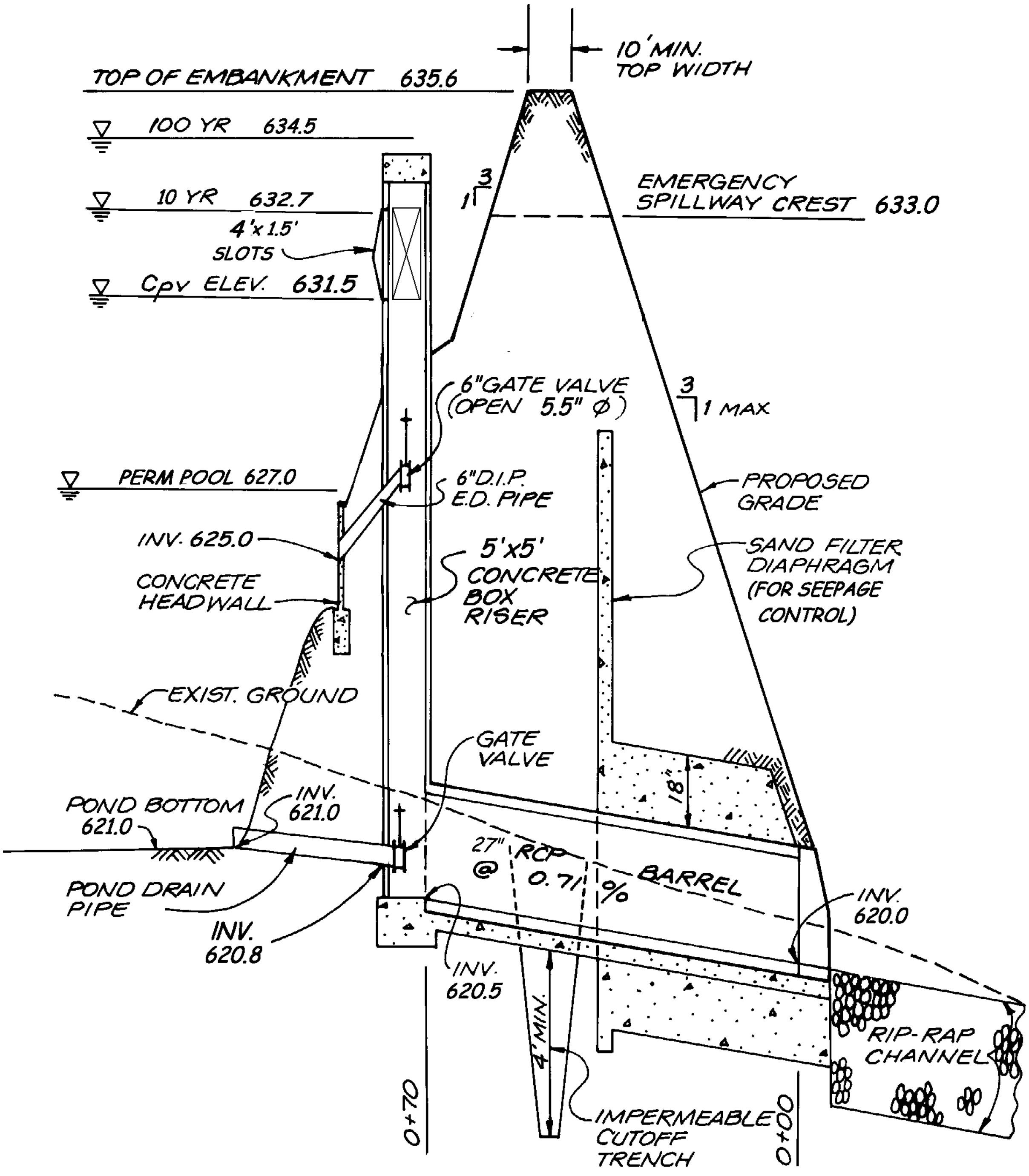
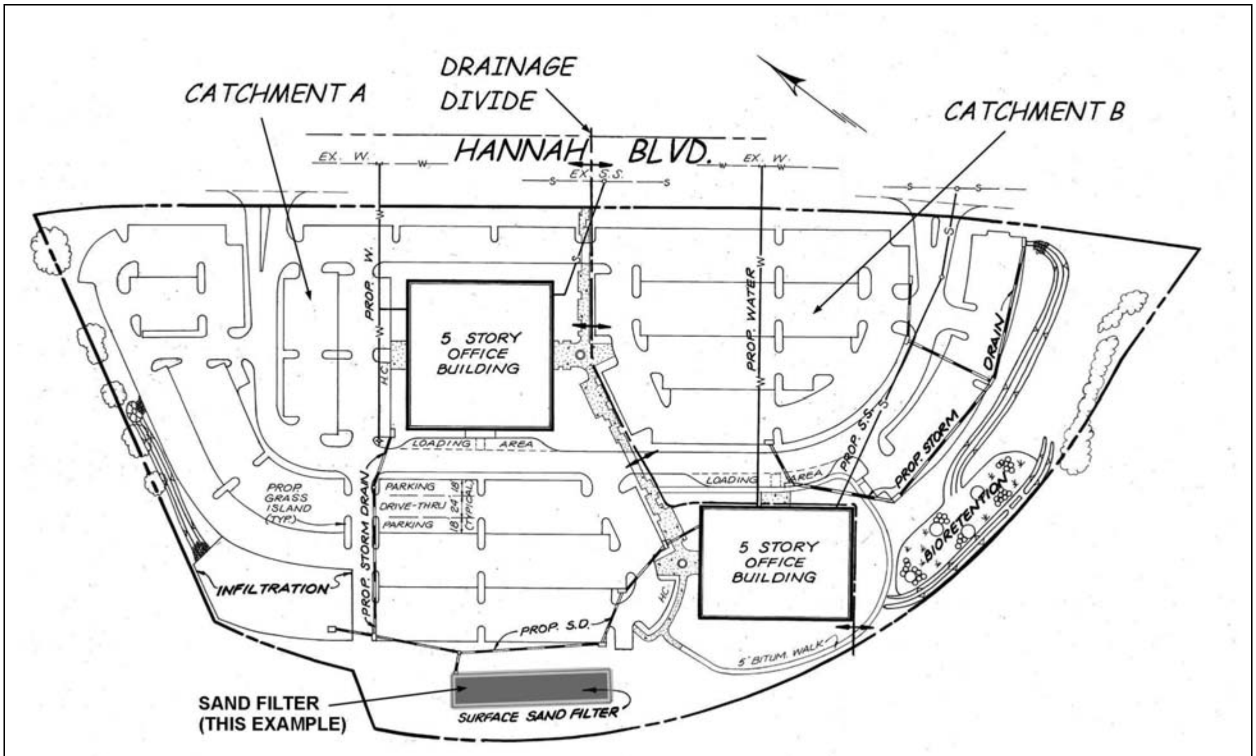


Figure C.8. Profile of Principle Spillway (Not to Scale)

Appendix C2 : Stormwater Sand Filter Design Example

This design example focuses on the design of a sand filter for Cameron Center, a hypothetical 12.8-acre commercial office park development located in Burlington, VT. Two five-story office buildings and associated parking are proposed for the site. The layout of the Cameron Center is shown in Figure C.9. Due to the site size and localized topographic features, the site area is divided into two catchments that drain to separate stormwater treatment practices. Catchment A is comprised of 8.1 acres and drains to the southwest to the proposed sand filter described in this example. Catchment B is on the remaining 4.7 acres and drains to the south to a proposed bioretention area (see Appendix C5 for the bioretention design). The recharge requirement for the site will be met using an infiltration trench (see Appendix C3 for the infiltration trench design), which receives runoff from about 2 acres of parking lot in Catchment A. The impervious cover (and therefore the runoff coefficient) is the same for each catchment, at 68% impervious. On-site soils are all HSG “B” soils.



Base Data	Hydrologic Data		
Location: Burlington, VT		Pre	Post
Site Area = Total Drainage Area (A) = 12.8 ac			
Impervious Area = 8.7 ac; or I =8.7/12.8 = 68%			
Soils Type “B”	CN	58	83
	t_c	.44	.10

Figure C.9. Cameron Center Site Plan

This step-by-step example will focus on meeting the water quality requirements for Catchment A only. Water quality treatment for Catchment B is described in Appendix C5. Channel protection control, overbank flood control, and extreme flood control are not addressed in this example. Therefore, a detailed hydrologic analysis is not presented. It is assumed that the designer can refer to the previous pond example in order to extrapolate the necessary information to determine and design the required storage and outlet structures to meet these criteria. In general, the primary function of sand filters is to provide water quality treatment and not large storm attenuation. As such, flows in excess of the water quality volume are typically routed to bypass the facility. For this example, the post-development 10-yr peak discharge is provided to appropriately size the necessary by-pass flow splitter. Where quantity control is required, bypassed flows can be routed to conventional detention basins (or some other facility such as underground storage vaults).

Step 1. Compute water quality volumes using the Unified Stormwater Sizing Criteria

Water Quality Volume, WQ_v

<u>Compute Runoff Coefficient, R_v</u>

R_v = 0.05 + (68) (0.009) = 0.66

<u>Compute WQ_v</u>

Catchment A:

WQ_v = (0.9”) (R_v) (A) / 12
= (0.9”) (0.66) (8.1ac) (43,560ft^2/ac) (1ft/12in)
= <u>17,465</u> ft^3 = <u>0.4</u> ac-ft

Recharge, Re_v (assume the Percent Volume Method will be used at the site)

Volume-based approach

Re_v = (0.25)(A)(I) / 12
= (0.25) (12.8) (0.68)/12
= <u>7,900</u> ft^3 = <u>0.18</u> ac-ft

- <u>Develop Site Hydrologic Input Parameters and Perform Preliminary Hydrologic Calculations (see Table C.10)</u>

  Note: For this design example, the 10-year peak discharge is given and will be used to size the bypass flow splitter. Any hydrologic models using SCS procedures, such as TR-20, HEC-HMS, or HEC-1, can be used to perform preliminary hydrologic calculations.

Table C.10. Site Hydrology

Condition	Area	CN	Q_1	Q_{10}	Q_{100}
	ac		*cfs*	*cfs*	*cfs*
Pre-developed	12.8	58	0.3	2	13
Post-Developed Catchment A	8.1	83	9.4	19	44
Post-Developed Catchment B	4.7	83	5.5	11	26
Post-Developed Total	12.8	83	15	30	70

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

Site Specific Data:

Existing ground elevation at practice location is 222.0 feet, mean sea level. Soil boring observations reveal that the seasonally high water table is at 213.0 feet. Adjacent creek invert is at 212.0.

Step 3. Confirm local design criteria and applicability.

The site drains to Lake Champlain, where phosphorus reduction has been identified as a major pollutant reduction goal.

Step 4. Compute WQ_v, available head, & peak discharge (Q_{wq})

- Compute Water Quality Volume:

  Initial WQ_v for Catchment A was previously determined to be 17,465 cubic feet (0.4 acre-feet)—see Step 1.

  In order to meet the recharge requirement at the site (see site layout discussion at the beginning of this example), an infiltration trench is proposed (see Appendix C3 for design example). Therefore, subtract the recharge volume (based on the proportional area of Catchment A) from the water quality volume.

  Net WQ_v to be treated by sand filter is: $17{,}465 ft^3 – (8.1ac/12.8ac)(7{,}900 ft^3) = 12{,}466\ ft^3$

- Determine available head (See Figure C.10):

  The low point at the parking lot is 223.5. Subtract 2' to pass the Q_{10} discharge (221.5) and a half foot for the inflow channel to the facility (221.0). The low point at the stream invert is 211.5. Set the outfall underdrain pipe 2.5' above the stream invert and add 0.5' to this value for the drain slope (214.5). Add to this value 8" for the gravel blanket over the underdrains, and 18" for the sand bed (216.67). The total available head is 221.0 - 216.67 or 4.33 feet. Therefore, the available average depth (h_f) = 4.33' / 2 = 2.17'.

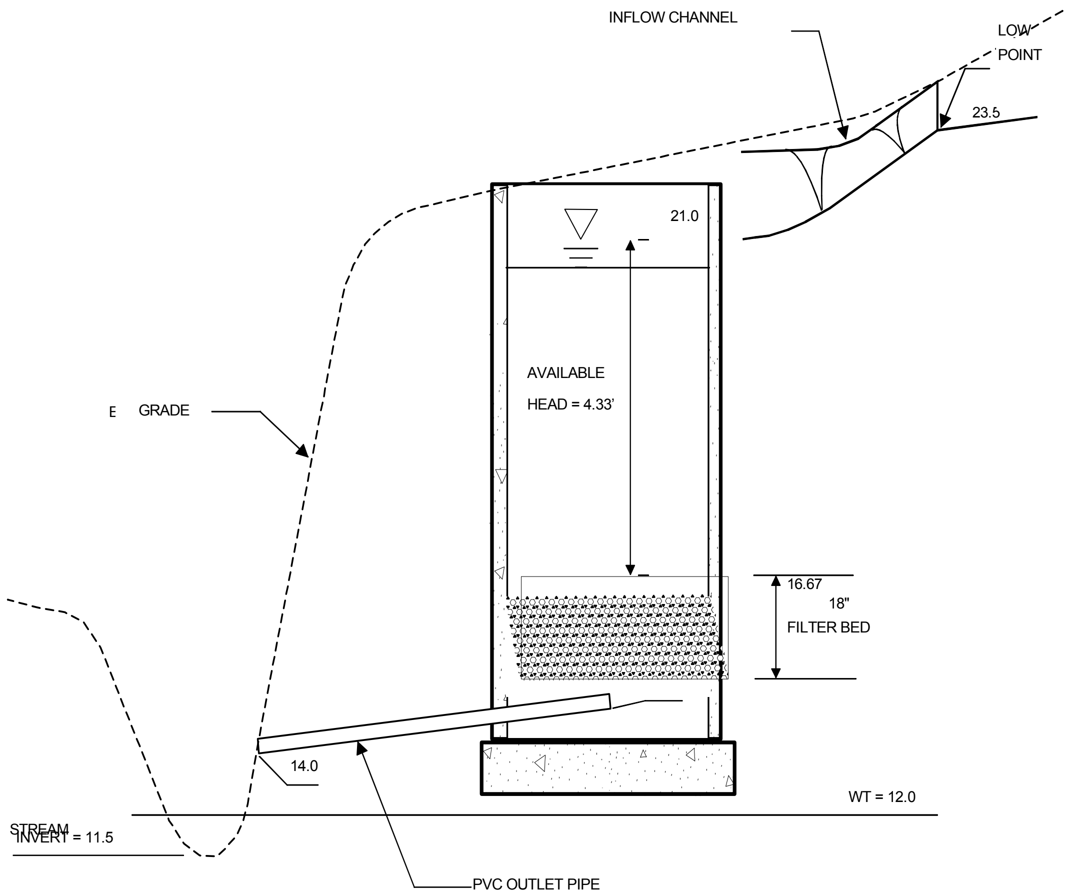


Figure C.10. Available Head Diagram

- Compute Peak Water Quality Discharge:

The peak rate of discharge for the water quality design storm is needed for the sizing of off-line diversion structures, such as sand filters and grass channels. The Small Storm Hydrology Method presented in Appendix D6 was followed to calculate a modified curve number and subsequent peak discharge associated with the 0.9-inch rainfall. Calculation steps are provided below.

Compute modified CN for 0.9" rainfall

P = 0.9"

Q_a = $WQ_v \div$ area = (12,466 $ft^3 \div$ 8.1ac $\div$ 43,560 ft^2/ac $\times$ 12 in/ft) = 0.42"

CN = $1000/[10+5P+10Q_a-10(Q_a^2+1.25 \times Q_a \times P)^{½}]$

= $1000/[10+5 \times 0.9+10 \times 0.42-10(0.42^2+1.25 \times 0.42 \times 0.9)^{½}]$

= 93.9

Use CN = 94

For CN = 94 and the t_c = 0.1 hours, compute the Q_{wq} for a 0.9" storm. With the CN = 94, a 0.9" storm will produce 0.4" of runoff. From TR-55 Chapter 2, Hydrology, I_a = 0.128, therefore:

$I_a/P = 0.128/0.9 = 0.142$.

From TR-55 Chapter 4 (or see Figure D.11 of this Manual), q_u = 975 csm/in, and

Q_{wq} = (975 csm/in) (8.1 ac/640ac/sq mi.) (0.42") = 5.2 cfs.

Step 5. Size flow diversion structure (see Figure C.11):

Size a low flow orifice to pass 5.2 cfs with approximately 1.5' of head using the Orifice equation.

$Q = CA(2gh)^{1/2}$; 5.2 cfs = (0.6) (A) [(2) (32.2 ft/s^2) (1.5')]$^{1/2}$

A = 0.88 sq ft = $\pi d^2/4$: d = 1.06' or 12.7"; use 13 inches

Size the 10-year overflow as follows:

The 10-year wsel is initially set at 23.0. Use a concrete weir to pass the 10-year flow (19.0 cfs) into a grassed overflow channel using the Weir equation. Assume 2' of head to pass this event. Overflow channel should be designed to provide sufficient energy dissipation (e.g., riprap, plunge pool, etc.) so that there will be non-erosive velocities.

$Q = CLH^{3/2}$

19 = 3.1 (L) (2')$^{1.5}$

L = 2.16'; use L = 2.2' which sets the width of the flow diversion overflow weir.

Weir wall elev. = 21.0. Set low flow invert at 21.0 - [1.5' + (0.5 x 13" x 1ft/12")] = 18.96.

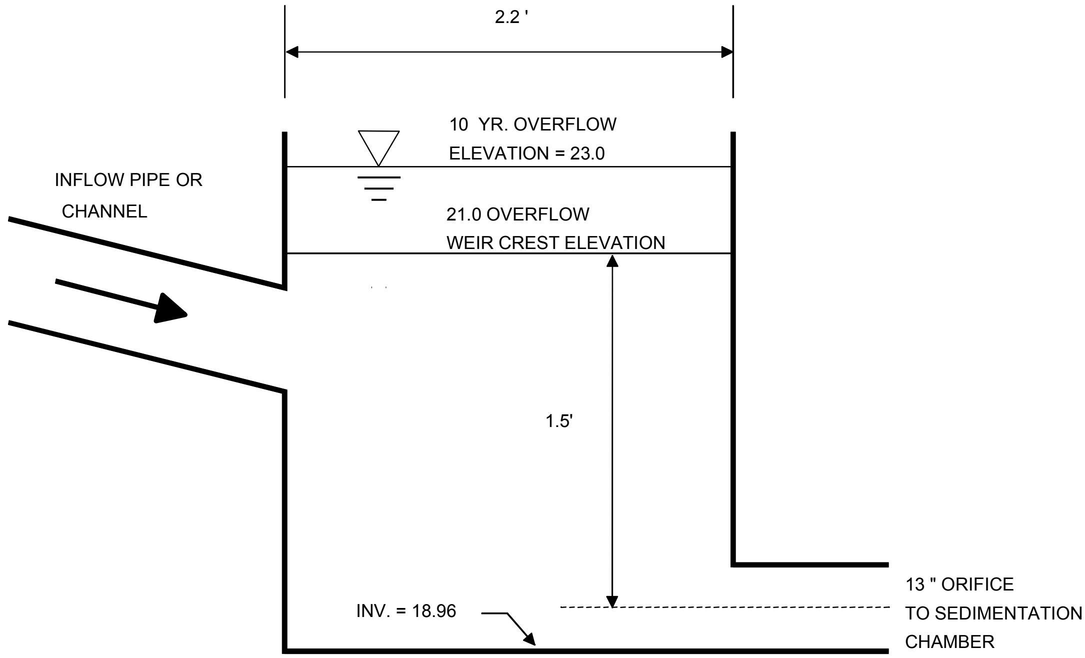


Figure C.11. Flow Diversion Structure

Step 6. Size filtration bed chamber (see Figure C.12).

From Darcy's Law: $A_f = WQ_v (d_f) / [k (h_f + d_f) (t_f)]$
where d_f = 18" or 1.5'
k = 3.5 ft/day
h_f = 2.17'
t_f = 40 hours
A_f = (12,466 cubic feet) (1.5') / [3.5 (2.17' + 1.5') (40hr/24hr/day)]

A_f = 873 sq ft; using a 2:1 ratio, say filter is 20' by 45' (= 900 sq ft)

Step 7. Size sedimentation chamber.

From Camp-Hazen equation: $A_s = 0.066 (WQ_v)$

A_s = 0.066 (12,466 cubic ft) or 823 sq ft

given a width of 20 feet, the length will be 823'/20' or 41.2 feet (use 20'x42')

Step 8. Compute V_{min}

V_{min} = ¾(WQ_v) or 0.75 (12,466 cubic feet) = 9,350 cubic feet

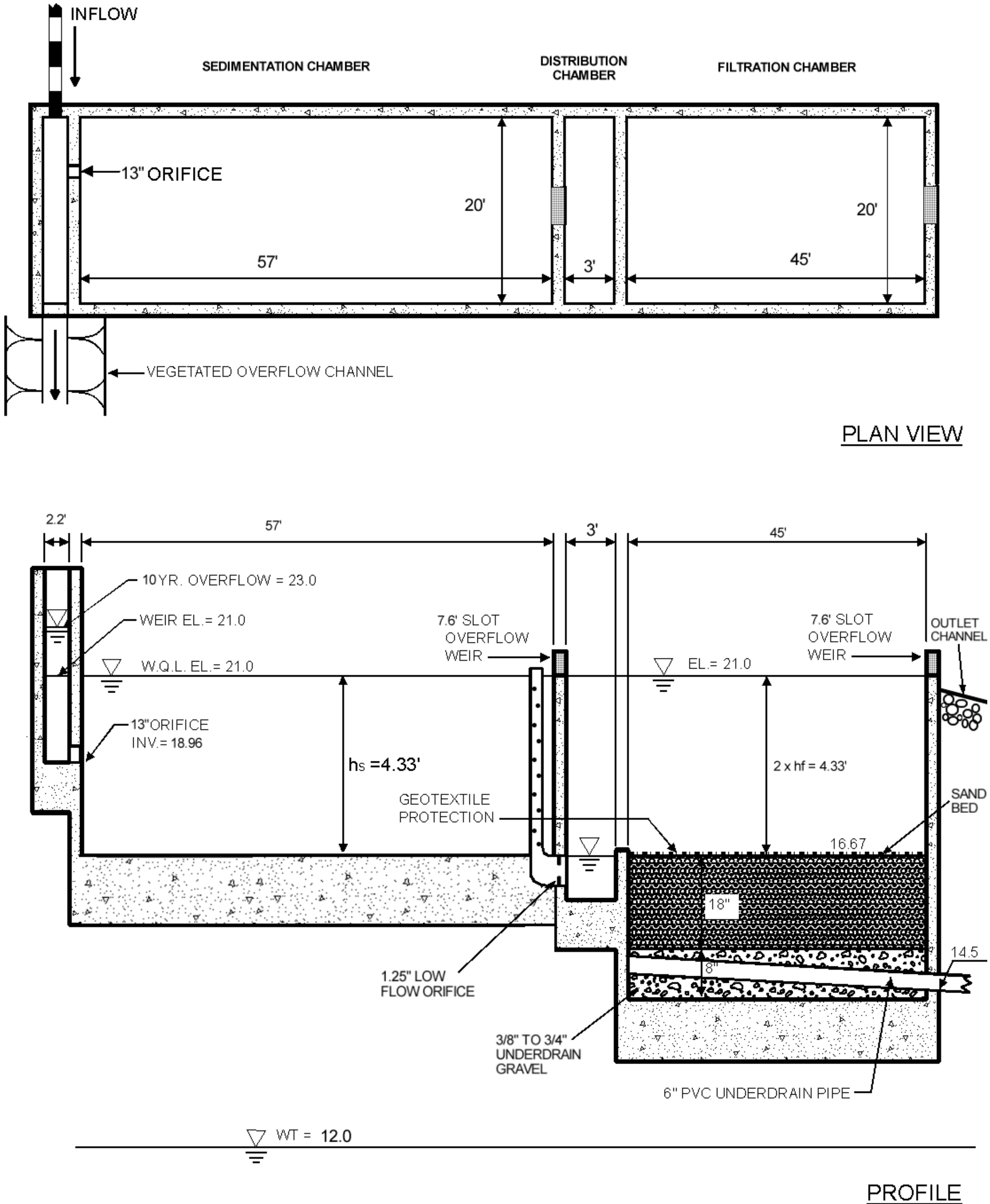


Figure C.12. Plan and Profile of Surface Sand Filter

Step 9. Compute volume within practice.

volume within filter bed (V_f): $V_f = A_f (d_f) (n)$; n = 0.4 for sand

V_f = (900 sq ft) (1.5') (0.4) = <u>540 cubic feet</u>

temporary storage above filter bed ($V_{f\text{-temp}}$): $V_{f\text{-temp}} = 2h_fA_f$

$V_{f\text{-temp}}$ = 2 (2.17') (900 sq ft) = <u>3,906 cubic feet</u>

Compute remaining volume for sedimentation chamber (V_s):

$V_s = V_{min} - [V_f + V_{f\text{-temp}}]$ or 9,350 - [540 + 3,906] = 4,904 cubic feet

compute height in sedimentation chamber (h_s): $h_s = V_s/A_s$

(4,904 cubic ft)/(20' x 42') = 5.84' which is larger than the head available (4.33'); increase the size of the settling chamber, using 4.33' as the design height;

(4,904 cubic ft)/4.33' = 1,133 sq ft; 1,133'/20' yields a length of 56.65 feet (say 57')

<u>new sedimentation chamber dimensions are 20' by 57'</u>

Provide a perforated standpipe with orifice sized to release volume (within sedimentation basin) over a 24 hr period (see Figure C.13). Average release rate equals 4,904 ft^3/24 hr = 0.06 cfs

Equivalent orifice size can be calculated using orifice equation:

$Q = CA(2gh)^{1/2}$, where h is average head, or 4.33'/2 = 2.17'.
0.06 cfs = 0.6 x A x $(2 \times 32.2\ ft/s^2 \times 2.17\ ft)^{1/2}$
A = 0.008 ft^2 = $\pi D^2/4$: therefore equivalent orifice diameter equals 1.25".

The recommended design is to cap the stand pipe with low flow orifice sized for 24 hr detention. Over-perforate pipe by a safety factor of 10 to account for clogging. Note that the size and number of perforations will depend on the release rate needed to achieve 24 hr detention. The stand pipe should discharge into a flow distribution chamber prior to filter bed. Distribution chamber should be between 2 and 4 feet in length and same width as filter bed (use 3' for this example). Flow distribution to the filter bed can be achieved either with a weir or multiple orifices at constant elevation. See Figure C.13 for stand pipe details.

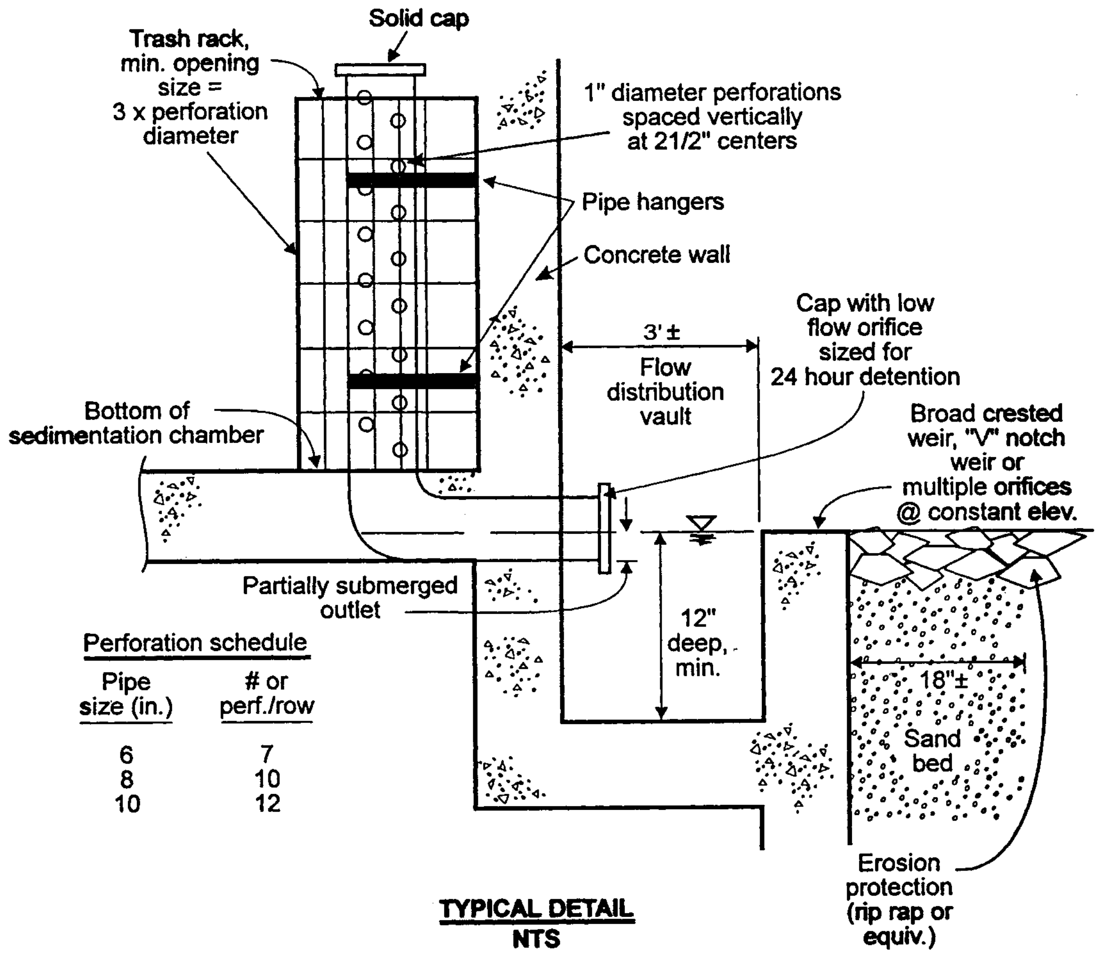


Figure C.13. Perforated Stand Pipe Detail

Step 10. Compute sedimentation chamber and filter bed overflow weir sizes

Assume overflow that needs to be handled is equivalent to the 13" orifice discharge under a head of 3.5 ft (i.e., the head in the diversion chamber associated with the 10-year peak discharge).

$Q = CA(2gh)^{1/2}$
$Q = 0.6(0.92\ ft^2)[(2)(32.2\ ft/s^2)(3.5\ ft)]^{1/2}$
$Q = 8.3$ cfs

Size the overflow weir from the sediment chamber and the filtration chamber to pass 8.3 cfs (this assumes no attenuation within the practice).

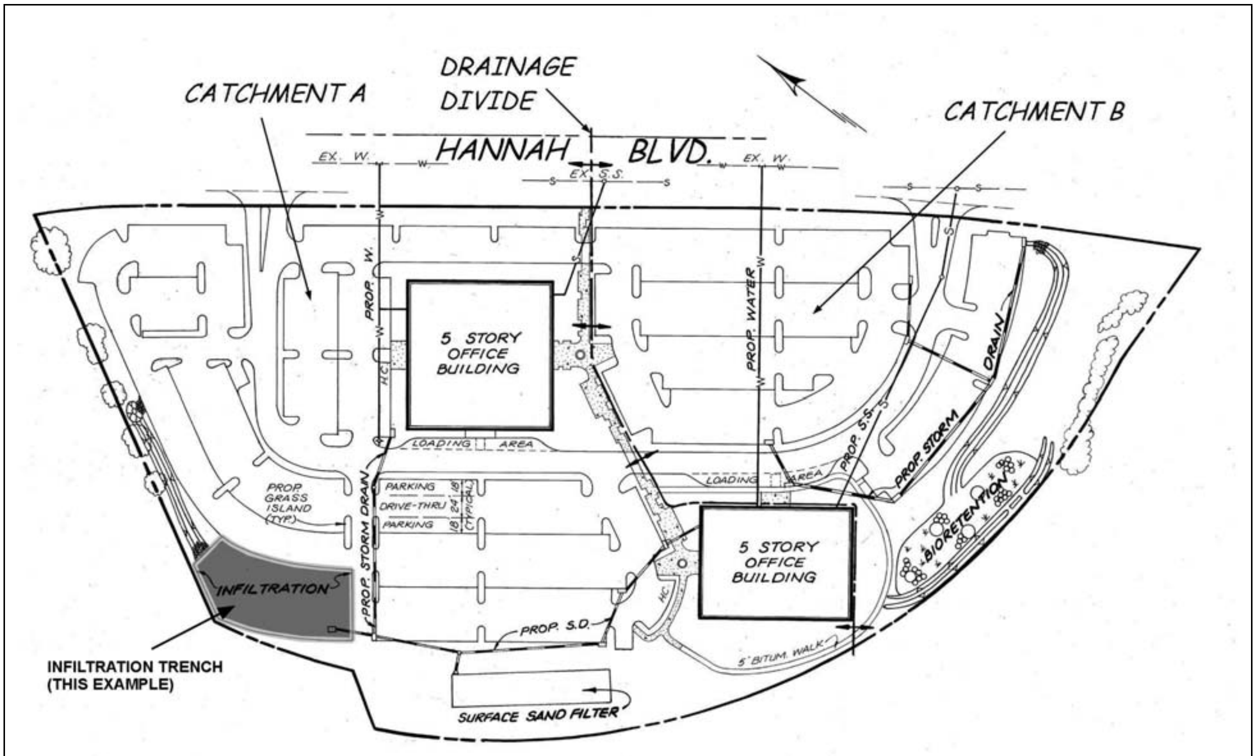
Weir equation: $Q = CLh^{3/2}$, assume a maximum allowable head of 0.5'
$8.3 = 3.1 \times L \times (0.5\ ft)^{3/2}$

L = 7.57 ft, Use L = 7.6 ft.

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

Appendix C3 : Stormwater Infiltration Trench Design Example

This design example focuses on the design of an infiltration trench for Cameron Center, a hypothetical 12.8 acre commercial office park development located in Burlington, VT. Two five-story office buildings and associated parking are proposed for the site. The layout of the Cameron Center is shown in Figure C.14. Due to the site size and localized topographic features, the site area is divided into two catchments that drain to separate stormwater treatment practices. Catchment A is comprised of 8.1 acres and drains to the southwest to the proposed sand filter (see Appendix C2 for the sand filter design). Catchment B is comprised on the remaining 4.7 acres and drains to the south to a proposed bioretention area (see Appendix C5 for the bioretention design). The recharge requirement for the site (the basis for this design example) will be met using an infiltration trench, which receives runoff from about 2 acres of parking lot in Catchment A. The impervious cover (and therefore the runoff coefficient) is the same for each catchment, at 68% impervious. On-site soils are predominantly HSG "B" soils.



Base Data	Hydrologic Data		
Location: Burlington, VT			
Site Area = Total Drainage Area (A) = 12.8 ac		**Pre**	**Post**
Impervious Area = 8.7 ac; or I =8.7/12.8 = 68%	CN	58	83
Soils Type "B"	t_c	.44	.10

Figure C.14. Cameron Center Site Plan

This step-by-step example will focus on meeting the groundwater recharge requirement for the entire site. The infiltration trench is located in a landscaped area adjacent to one of the office buildings. A drop inlet receives flows in excess of the infiltration trench capacity, which in turn delivers the water to the sand filter practice (see Appendix C2). Channel protection control, overbank flood control, and extreme flood control are not addressed in this example. Therefore, a detailed hydrologic analysis is not presented. It is assumed that the designer can refer to the previous pond example in order to extrapolate the necessary information to determine and design the required storage and outlet structures to meet these criteria. In general, the primary function of infiltration trenches is to provide water quality treatment and /or recharge and not large storm attenuation. As such, flows in excess of the water quality volume are typically routed to bypass the facility. For this example, the post-development 10-yr peak discharge is provided to appropriately size the necessary by-pass flow splitter. Where quantity control is required, bypassed flows can be routed to conventional detention basins (or some other facility such as underground storage vaults).

Step 1. Compute recharge volumes using the Unified Stormwater Sizing Criteria.

Recharge, Re_v

The volume-based approach will be used since an infiltration trench is proposed to meet the recharge requirement at the site.

Volume-based approach

$$
\begin{aligned}
Re_v &= (0.25)(A)(I) / 12 \\
&= (0.25)\ (12.8)\ (0.68)/12 \\
&= \underline{7,900}\ ft^3 = \underline{0.18}\ ac\text{-}ft
\end{aligned}
$$

Step 2. Determine if the development site and conditions are appropriate for the use of an infiltration trench.

Site Specific Data:

Table C.11 presents site-specific data, such as soil type, percolation rate, and slope, for consideration in the design of the infiltration trench. See Appendix D1 for infiltration testing requirements and Appendix B2 for infiltration practice construction specifications.

Table C.11. Site Specific Data

Criteria	Value
Soil	Silt Loam
Percolation Rate	0.5"/hour
Ground Elevation at BMP	220'
Seasonally High Water Table	212'
Soil slopes	<1%

Step 3. Confirm local design criteria and applicability.

Table C.12, below, summarizes the requirements that need to be met to successfully implement infiltration practices. On this site, infiltration is feasible, with restrictions on the depth and width of the trench.

Table C.12. Infiltration Feasibility

Criteria	Status
Infiltration rate (f_c) greater than or equal to 0.5 inches/hour.	Infiltration rate is 0.5 inches/hour. OK.
Soils have a clay content of less than 20% and a silt/clay content of less than 40%.	Silt Loam meets both criteria.
Infiltration cannot be located on slopes greater than 6% or in fill soils.	Slope is <1%; not fill soils. OK.
Hotspot runo	Not a hotspot land use. OK.
The bottom of the infiltration facility must be separated by at least two feet vertically from the seasonally high water table.	Elevation of seasonally high water table: 12' Elevation of BMP location: 20'. The difference is 8'. Thus, the trench can be up to 5' deep. OK.
Infiltration facilities must be located 100 feet horizontally from any water supply well.	No water supply wells nearby. OK.
Maximum contributing area generally less than 5 acres.	Area draining to facility is approximately 2 acres (see Figure C.14). OK.
Setback 25 feet down-gradient from structures.	Trench edge is > 25' from all structures. OK.

Step 4. Size the infiltration trench.

The area of the trench can be determined by the following equation:

$$A = \frac{Re_v}{(nd + kT/12)}$$

Where:

- A = Surface Area
- Re_v = Recharge volume (ft^3)
- n = Porosity
- d = Trench depth (feet)
- k = Percolation (inches/hour)
- T = Fill Time (time for the practice to fill with water), in hours

Assume that:

- n = 0.32
- d = 5 feet (see above; feasibility criteria)
- k = 0.5 inch/hour (see above; site data)
- T = 2 hours (this is recommended default value to be used unless site-specific data exists)

Therefore:

A = 7,900 ft^3 / (0.32 × 5 + 1 × 2/12)ft

A = 4,472 ft^2

The proposed location for the infiltration trench will accommodate a trench width of up to 45 feet (see Figure C.15 for a site plan view). Therefore, the minimum length required would be:

L = 4,472 ft^2 / 45 ft

L = 99 feet, say 100 feet

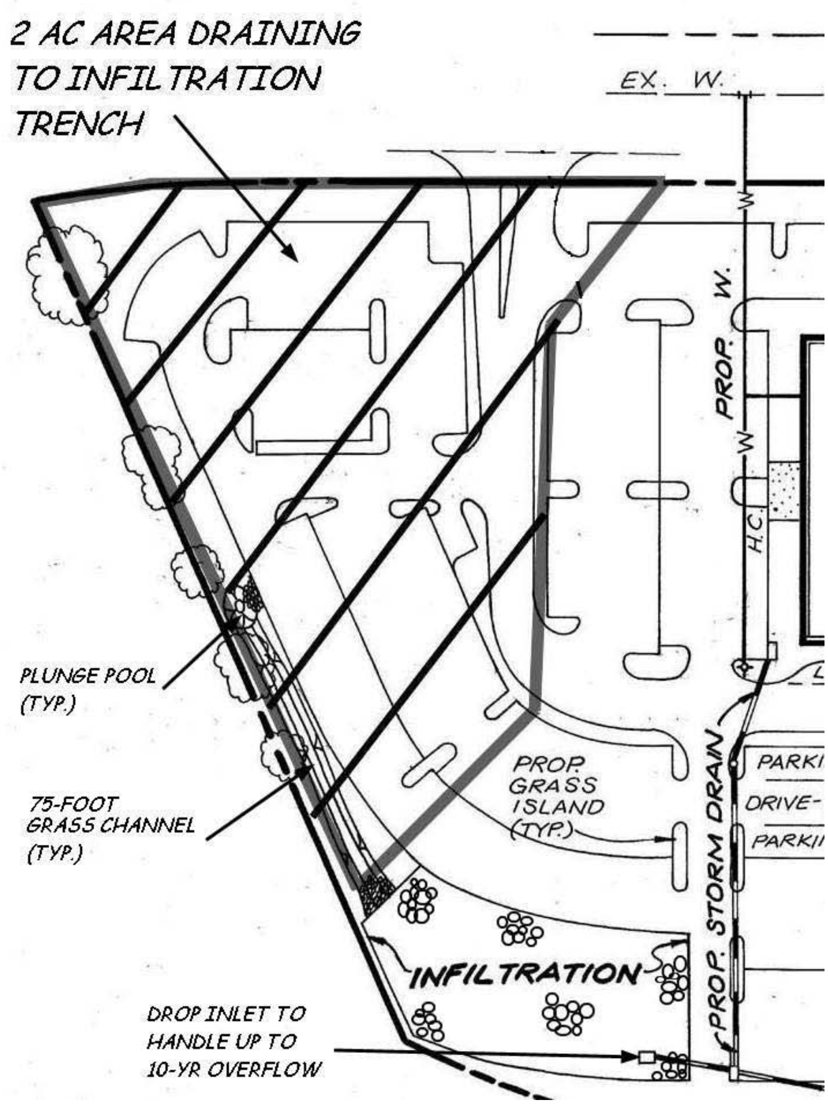


Figure C.15. Infiltration Trench Site Plan

Step 5. Size pretreatment.

As rule of thumb, size pretreatment to treat ¼ of the Re_v. Therefore, treat 7,900 × 0.25 = 1,975 ft^3.

For pretreatment, use a pea gravel filter layer with filter fabric, a plunge pool, and a grass channel.

Pea Gravel Filter

A pea gravel filter layer is used for this example. Alternatively, a 6" sand layer could also be used. The pea gravel filter layer covers the entire trench with 2" (see Figure C.16). Assuming a porosity of 0.32, the pretreatment volume (Pv) provided in the pea gravel filter layer is:

Pv_{filter} = (0.32)(2")(1 ft/12 inches)(4,472ft^2) = 239 ft^3

Plunge Pools

Use a 5'X10' plunge pool at inflow point of concentrated runoff to grass channel (see Figure C.15)

Pv_{pool} = (10 x 5 ft)(2 ft) = 100 ft^3

Grass Channel

Accounting for the pretreatment volumes provided by the pea gravel filter and plunge pool, the grass channel then needs to treat at least (1,975 - 239 - 100)ft^3 = 1,636 ft^3

Using guidance in Section 2.7.4.C of the *Vermont Stormwater Management Manual-Volume I,* size channel length based on imperviousness of 68%, channel slope of 1%, and drainage area of about 2 acres as follows:

- For a 2 acre site with 68% impervious cover (i.e., a runoff coefficient, R_v, of 0.66), the peak flow associated with the water quality rainfall of 0.9" is approximately 1.8 cfs (see Appendix D6 for guidance on computing water quality storm peak discharge).
- For a 4' wide (bottom width) channel with 3:1 sideslopes (horizontal:vertical) and a slope of 0.75%, the velocity is approximately 0.6 fps (this can be determined using nomographs, Manning's equation, or available computer software packages). (Note: the designer may need to balance the hydraulic parameters such as channel slope, bottom width, and sideslopes with the site constraints such local topography and available space to arrive at an acceptable design. In this example, the initial target slope of 1% was reduced to 0.75%.)
- Therefore, using a required residence time of 10 minutes (600 seconds), the required length of channel for 100% of the WQ_v would be 0.6 fps x 600 sec = 360ft.
- For pretreatment requirements, 25% of the WQ_v is needed, or 0.25 x 360 ft = 90 ft.
- Adjusting for pretreatment already provided by the pea gravel filter layer and plunge pool will generate the required grass channel length, or (1,636 ft^3 / 1,975 ft^3)(90 ft) = 75 ft.

Therefore, for this example, a grass channel length of at least <u>75 feet</u> is required.

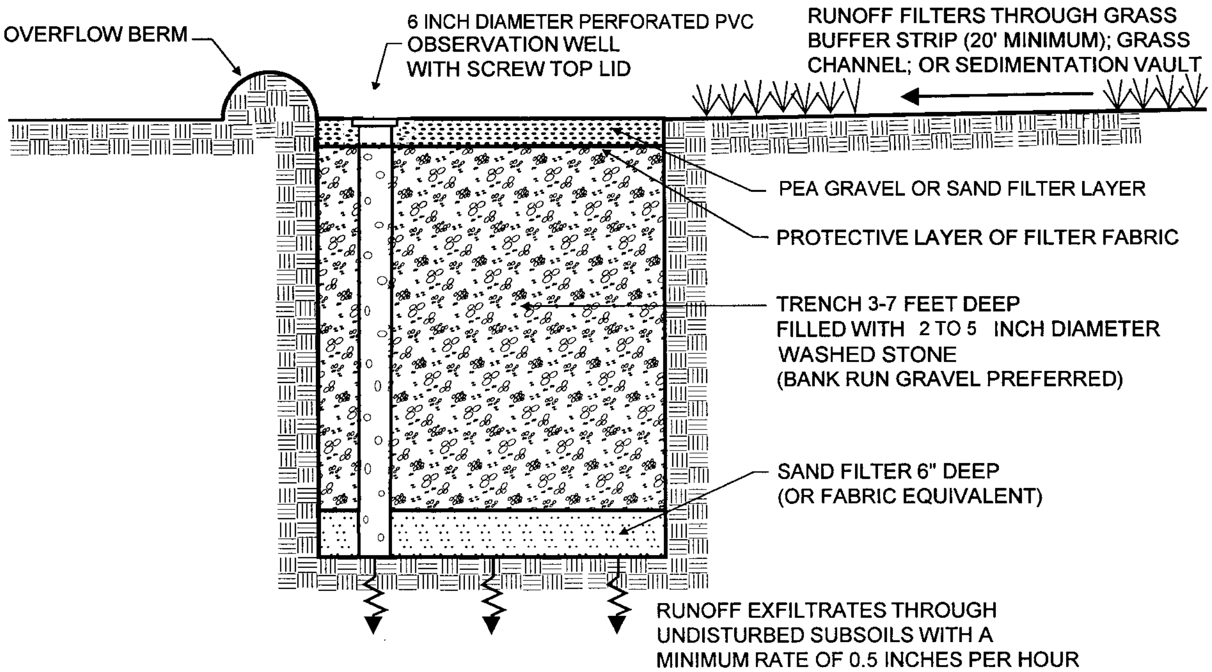


Figure C.16. Infiltration Trench Cross Section

Step 6. Design Spillway(s).

Locate drop inlet at downstream edge of infiltration facility and size to ensure passage of 10-yr peak discharge. Assume peak discharge is one fourth of that for Catchment A, or:

Q = 0.25 x 19 cfs = 4.75 cfs

Using the weir equation and assuming a maximum head of 0.5 feet, the minimum weir crest length (i.e., perimeter of the drop inlet) should be:

$Q = CLh^{3/2}$
$4.75 = (3.1)L(0.5)^{3/2}$
$L = 4.33$ feet

Appendix C4 : Grass Channel Design Example

This design example focuses on the design of a grass channel in association with a two lane arterial road known as Owens Parkway. Owens Parkway is a hypothetical road project located in Chittenden County. The applicant proposes to meet both the recharge and water quality requirements for the project using the grass channel practice (O3). The project consists of a 3-mile connector road and drains through 4 separate catchments. The site area for this example consists of a 2,200 foot section of the project and has an area 1.52 acres and drains to the "study point" illustrated in Figure C.17. The impervious cover is equal to 1.01 acres associated with one 12' lane and an 8' shoulder (I = 2,200' x 20' = 44,000 ft^2 = 1.01 acres). On-site soils are all HSG "B" soils.

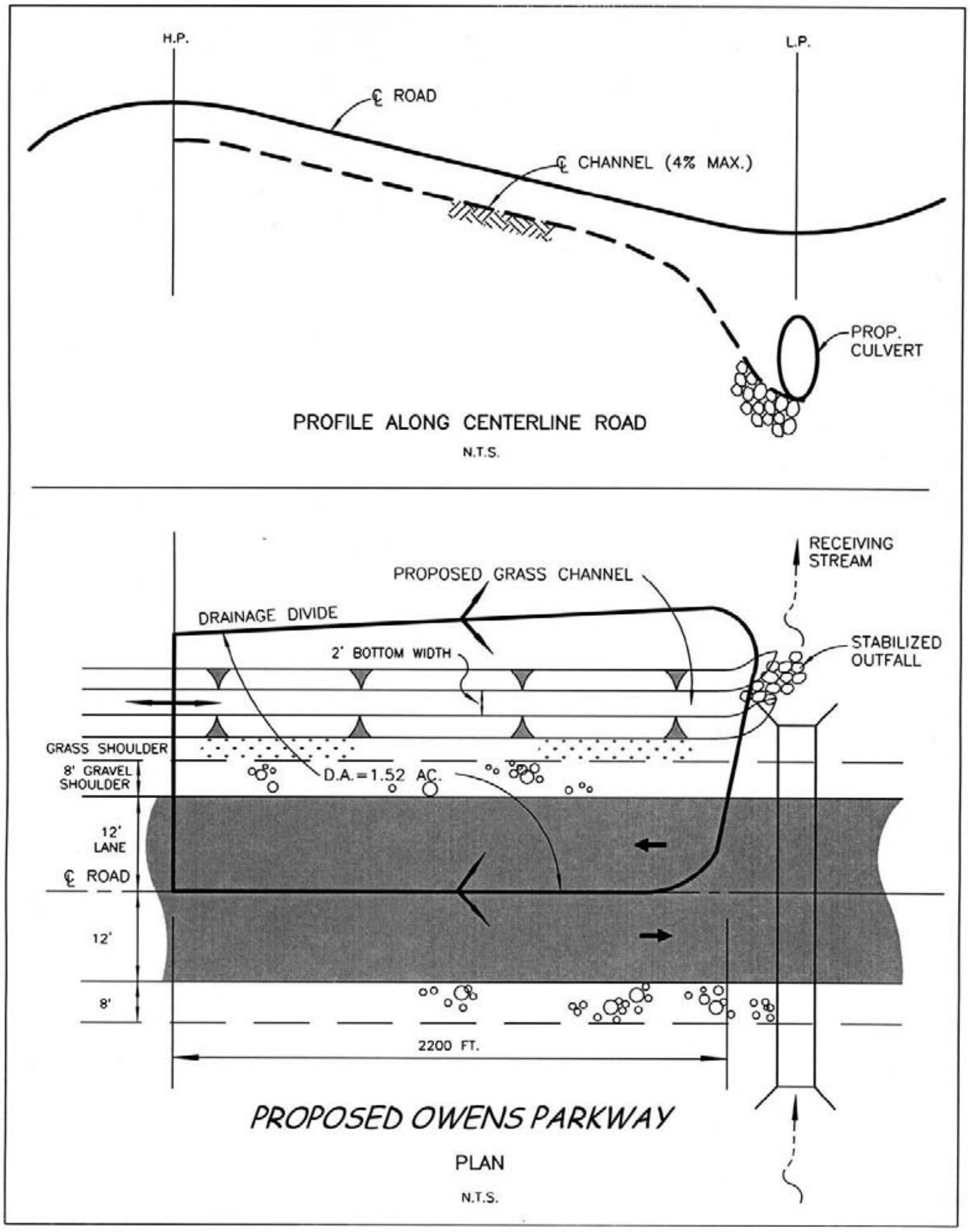
This step-by-step example will focus on meeting the water quality requirements for the site only. Management of the channel protection and overbank flood control are not presented in this example. Since the project will have open section roads, the recharge requirements (Re_v) are automatically met (see "Grass Channel Credit," Section 3.5 of the *Vermont Stormwater Management Manual-Volume I*). In general, the primary function of grass channels is to provide water quality treatment and convey larger storms in a non-erosive condition. For this example, the post-development 1-year discharge is computed to check for non-erosive flows and the post-development 10-year discharge is computed to ensure that the channel has capacity to convey the event. In cases where Cp_v and/or Q_p are required, flows can be routed to conventional downstream retention or detention basins.

Step 1. Determine if the development site and conditions are appropriate for the use of grass channel.

The proposed road grade in the vicinity of the grass channel is approximately 4.0%. Soils are modestly to well drained. There is sufficient area to accommodate the grass channel within the roadside drainage system.

Step 2. Confirm local design criteria and applicability.

The site drains to Lake Champlain, where phosphorus reduction has been identified as a major pollutant reduction goal.



Base Data	Hydrologic Data		
Location: Chittenden County, VT			
Site Area = Total Drainage Area (A) = 1.52 ac		**Pre**	**Post**
Impervious Area = 1.01 ac; or I = 1.01/1.52 = 66%			
R_v = 0.05 + (66 %)(0.009) = 0.64	CN	n/a	83
Soils Type “B”	t_c	n/a	.17

Figure C.17. Owens Parkway Site Plan

Step 3. Compute peak discharge associated with water quality volume storm

The water quality volume (WQ_v) is computed for illustrative purposes and to verify pretreatment requirements. Technically, since the grass channel is a "rate-based" design, only the peak discharge associated with the water quality rainfall of 0.9" is required for sizing the "treatment" portion of the practice. The small storm hydrology method (Appendix D6) is used for this.

$$WQ_v = (P)(R_v)(A) = (0.9")(0.64)(1.52 \text{ ac})(1 \text{ ft}/12 \text{ in})(43{,}560 \text{ ft}^2/\text{ac}) = 3{,}178 \text{ ft}^3$$

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

$$CN = 1000/[10 + 5P + 10Q_a - 10(Q_a^2 + 1.25\, Q_a P)^{½}]$$

Where, P = rainfall, in inches (0.9")
Q_a = runoff volume, in inches (equals $P \times R_v$) = (0.9")(0.64) = 0.58"

- $CN = 1000/[10 + 5(0.9) + 10(0.58) – 10((0.58)^2 + 1.25(0.58)(0.9))^{1/2}]$
- CN = 96.6, Use 97

Once a CN is computed, the time of concentration (t_c) is computed (based on the methods identified in TR-55, and Sections 2-3 of the Vermont Stormwater Management Manual-Volume I).

- Based on the site geometry and flow path, assume t_c = 10 min. = 0.17 hr.

Using a CN = 97, a t_c = 0.17 hrs. and a drainage area (A) = 1.52 ac; the peak discharge (Q_{wq}) for the water quality storm event is computed as follows:

- Read initial abstraction (Ia), from TR-55 page 4-1 (Table 4-1) = 0.062,
- compute Ia/P = 0.062/0.9 = 0.07
- from Appendix D6, Figure D.11, read the unit peak discharge (q_u) = 900 csm/in.

Using the water quality volume (WQ_V), compute the peak discharge (Q_{wq})

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

Where, Q_{wq} = the peak discharge, in cfs
q_u = the unit peak discharge, in cfs/mi²/inch
A = drainage area, in square miles
WQ_V = Water Quality Volume, in watershed inches

- Q_{wq} = (900 csm/in)(1.52 ac)(0.58 in)/640 ac/mi^2) = 1.2 cfs

Step 4. Size a channel and compute the required length to convey the peak discharge associated with the water quality volume storm

Check velocity and depth for the following parameters:

- Longitudinal slope = 4.0%
- Q_{wq} = 1.2 cfs
- Bottom width = 2.0 ft
- Side slopes = 3:1
- Manning's coefficient = 0.14 (see Section 2.7.5.B of the Vermont Stormwater Manual-Volume I and Appendix D7 of this manual for guidance on Manning's n determination)

Using Manning's equation: $Q = (v)(a) = (a)[1.49/n\ (R)^{2/3}\ (S)^{1/2}]$, where v = velocity, a = cross sectional area, n = Manning's coefficient, R = hydraulic radius, and S = channel longitudinal slope; solve for velocity and depth. (Note: this can be determined using nomographs, Manning's equation, or available computer software packages).

Results:
- v = 0.9 ft/s (v is less than 1.0 ft/s, so ok)
- Depth = 0.41 ft, check to make sure depth corresponds with Manning's n as illustrated in Appendix D7, Figure D.14. (depth = 0.41 ft = 4.9 in., which corresponds to a Manning's n of about 0.14, so ok).

For a 10 minute average residence time, the channel length (L) must equal or exceed: (v)(t), where v = water quality flow velocity and t = 10 min. residence time:

L = (v)(t) = (0.9 ft/s)(10 min.)(60 s/min.) = 540 feet.

(Channel length is 2,200 ft, so ok)

Step 5. Check the velocity of the 1-year storm and the hydraulic capacity of the 10-year storm

Develop Site Hydrologic Input Parameters and Perform Preliminary Hydrologic Calculations (see Table C.13)

Note: For this design example, the 1-year storm is used to check the grass channel geometry for non-erosive conditions, and the 10-year storm is used to check the conveyance capacity of the channel. Any hydrologic models using SCS procedures, such as TR-20, HEC-HMS, or HEC-1, can be used to perform preliminary hydrologic calculations. In this example, TR-55 was used to compute these values (see Table C.13).

Check to ensure non-erosive velocity for 1-year storm. Check velocity and depth for the following parameters:

- Longitudinal slope = 4.0%
- $Q_{1\text{-year}}$ = 1.8 cfs
- Bottom width = 2.0 ft
- Side slopes = 3:1
- Manning's coefficient = 0.13 (see Section 2.7.5.B of the Vermont Stormwater Manual-Volume I and Appendix D7 of this manual for guidance on Manning's n determination)

Using Manning's equation: $Q = (v)(a) = (a)[1.49/n\ (R)^{2/3}\ (S)^{1/2}]$, where v = velocity, a = cross sectional area, n = Manning's coefficient, R = hydraulic radius, and S = channel longitudinal slope; solve for velocity and depth.

Results:

- v = 1.03 ft/s (v is less than 2.5 ft/s, so ok for slope range 0-5%, see Appendix D7)
- Depth = 0.48 ft, check to make sure depth corresponds with Manning's n as illustrated in Appendix D7, Figure D.14. (depth = 0.48 ft = 5.8 in., which corresponds to a Manning's n of about 0.13, so ok).

Check to ensure adequate capacity for 10-year storm. Check depth, given the following:

- Longitudinal slope = 4.0%
- $Q_{10\text{-year}}$ = 4.0 cfs
- Bottom width = 2.0 ft
- Side slopes = 3:1
- Manning's coefficient = 0.11

Using Manning's equation: $Q = (v)(a) = (a)[1.49/n\ (R)^{2/3}\ (S)^{1/2}]$, where v = velocity (ft/s), n = Manning's coefficient, R = hydraulic radius, a = cross sectional area, and S = channel longitudinal slope; solve for depth.

Table C.13. Perkin's Parkway Post-Developed - TR-55 Output

DISCHARGE SUMMARY				
JOB:	Perkins Parkw			RAC
DRAINAGE AREA NAME:	Proposed Co			20-Dec-01
COVER DESCRIPTION	SOIL NAME	GROUP A,B,C,D?	CN from TABLE 2-2	AREA (In acres)
Impervious cover		B	98	1.01 Ac.
Pervious cover		B	61	0.51 Ac.
		AREA SUBTOTALS:		1.52 Ac.
Time of Concentration	Surface Cover	Manning 'n'	Flow Length	Slope
2-Yr 24 Hr Rainfall = 2.3 In	Cross Section	Wetted Per	Avg Velocity	Tt (Hrs)
Sheet Flow	impervious	'n'=0.01	18 Ft.	2.00%
(a)				0.01 Hrs
	short grass	'n'=0.150	10 Ft.	2.00%
(b)				0.03 Hrs
Shallow Flow	no match			
Channel Flow		'n'=0.030	2000 Ft.	4.00%
0.27	1.2 SqFt	4.4 Ft.	4.17	0.13 Hrs.
Total Area in Acres =	1.52 Ac.	Total Sheet	Total Shallow	Total Channel
Weighted CN =	86	Flow=	Flow=	Flow =
Time Of Concentration =	0.17 Hrs.	0.04 Hrs.	0.00 Hrs.	0.13 Hrs.
Pond Factor =	1	RAINFALL TYPE II		
STORM	*Precipitation* (P) inches	*Runoff* (Q)	Qp, PEAK DISCHARGE	TOTAL STORM Volumes
1 Year	2.1 In.	0.9 In.	1.8 CFS	5,106 Cu. Ft.
2 Year	2.3 In.	1.1 In.	2.1 CFS	5,971 Cu. Ft.
10 Year	3.2 In.	1.8 In.	4 CFS	10,125 Cu. Ft.
100 Year	5.2 In.	3.7 In.	7 CFS	20,162 Cu. Ft.

Results:

- Depth = 0.66 ft, check to make sure depth corresponds with Manning's n as illustrated in Appendix D7, Figure D.14. (depth = 0.66 ft = 7.9 in., which corresponds to a Manning's n of 0.09, so do another iteration with Manning's n set at 0.09. Resulting depth is 0.6 ft, ok).

Step 6. Set Design Elevations and Dimensions.

Using the information from the previous steps, set the design elevations for the water quality, 1-year, and 10-year discharges.

- WQ_v: v = 0.9 ft/s, depth = 0.41 ft
- 1-year: v = 1.03 ft/s, depth = 0.48 ft
- 10-year: depth = 0.6 ft

Set freeboard equal to 6 inches above 10-year depth = 0.6' + 0.5' = 1.1 ft.

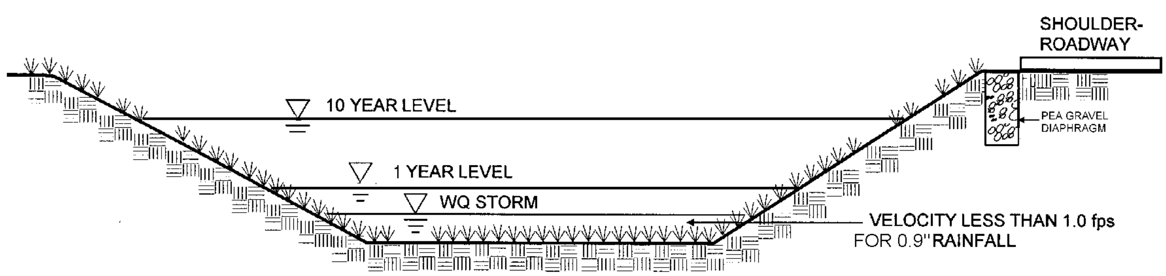


Figure C.18. Typical Section of Grass Channel Design Example

Step 7. Design Pretreatment.

Pretreat with side slopes and pea gravel diaphragm (curtain drain).

For pretreatment requirements, 10% of the WQ_v is needed.

- WQ_v = 3,178 ft^3, pretreatment volume = (0.1)(3,178 ft^3) = 318 ft^3

Since the channel is not discharging directly from a concentrated inflow point (i.e., a pipe), no formal pretreatment chamber is required. Instead, pretreatment is provided by a pea gravel diaphragm (running parallel to the roadway shoulder) and the slopes leading to the edge of the channel (see Figure C.18)

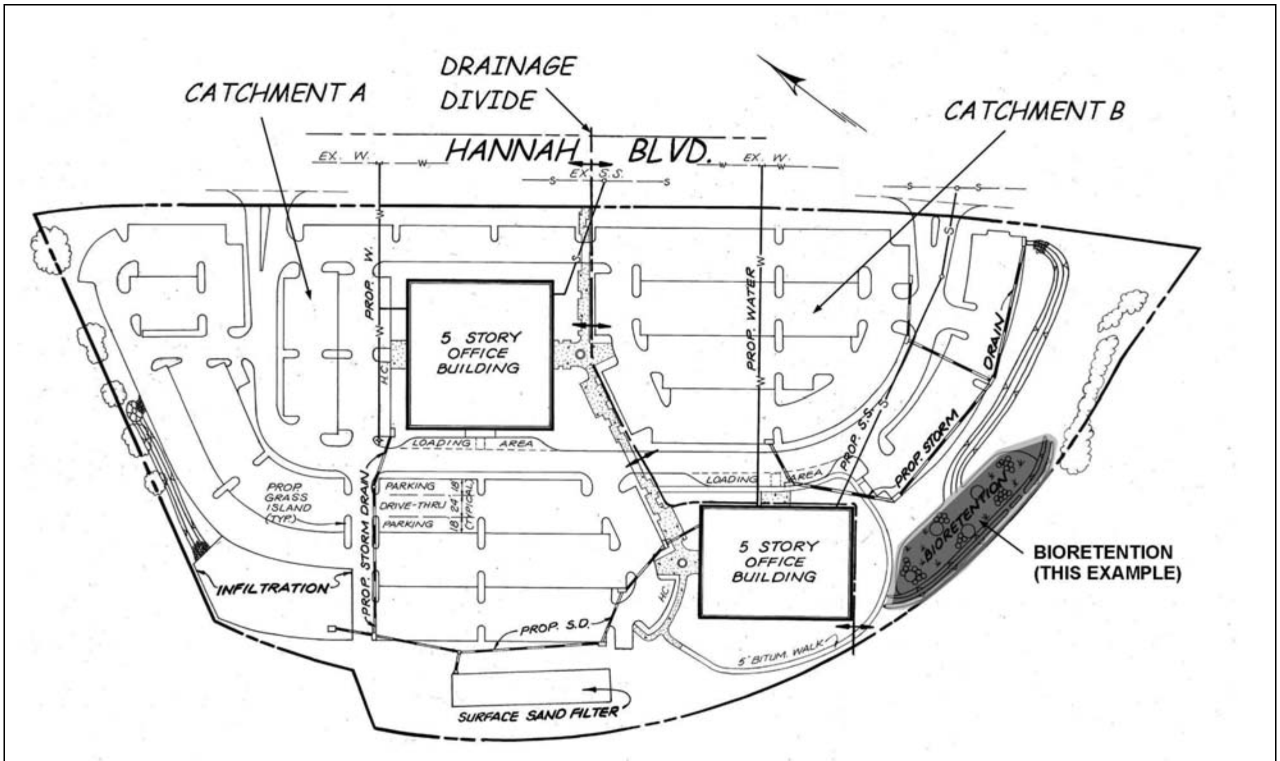
Step 8. Choose vegetation for channel.

Choose vegetation based on factors such as resistance to erosion, resistance to drought and inundation, cost, aesthetics, maintenance, etc (see Appendix D7).

Based on the project slope range (0-5%), and 1-year velocity equal to approximately 1.0 ft/s, choose Kentucky Bluegrass for grass channel (good cold temperature hardiness, moist to well drained soils, higher permissible velocities, and good establishment rate, but poor salt tolerance).

Appendix C5 : Stormwater Bioretention Design Example

This design example focuses on the design of a bioretention facility for Cameron Center, a hypothetical 12.8 acre commercial office park development located in Burlington, VT. Two five-story office buildings and associated parking are proposed for the site. The layout of the Cameron Center is shown in Figure C19. Due to the site size and localized topographic features, the site area is divided into two catchments that drain to separate stormwater treatment practices. Catchment A is comprised of 8.1 acres and drains to the southwest to the proposed sand filter described in Appendix C2. Catchment B is on the remaining 4.7 acres and drains to the south to a proposed bioretention area (described in this example).



<u>Base Data</u>
Location: Burlington, VT
Site Area = Total Drainage Area (A) = 12.8 ac
Impervious Area = 8.7 ac; or I =8.7/12.8 = 68%
Soils Type “B”

<u>Hydrologic Data</u>

	Pre	**Post**
CN	58	83
t_c	.44	.10

Figure C.19. Cameron Center Site Plan

The recharge requirement for the site will be met using an infiltration trench (see Appendix C3 for the infiltration trench design), which receives runoff from about 2 acres of parking lot in Catchment A. The impervious cover (and therefore the runoff coefficient) is the same for each catchment, at 68% impervious. On-site soils are predominantly HSG “B” soils.

This step-by-step example will focus on meeting the water quality requirements for Catchment B only. Water quality treatment for Catchment A is described in Appendix C2. Channel protection control, overbank flood control, and extreme flood control are not addressed in this example. Therefore, a detailed hydrologic analysis is not presented. It is assumed that the designer can refer to the previous pond example in order to extrapolate the necessary information to determine and design the required storage and outlet structures to meet these criteria. In general, the primary function of bioretention facilities is to provide water quality treatment and not large storm attenuation. As such, flows in excess of the water quality volume are typically routed to bypass the facility. For this example, the post-development 10-yr peak discharge is provided to appropriately size the necessary by-pass flow splitter. Where quantity control is required, bypassed flows can be routed to conventional detention basins (or some other facility such as underground storage vaults).

Step 1. Compute water quality volumes using the Unified Stormwater Sizing Criteria

Water Quality Volume, WQ_v

Compute Runoff Coefficient, R_v

R_v = 0.05 + (68) (0.009) = 0.66

Compute WQ_v

Use the 90% capture rule with 0.9” of rainfall.

Catchment B

$$\begin{aligned} WQ_v &= (0.9\text{''})\,(R_v)\,(A) / 12 \\ &= (0.9\text{''})\,(0.66)\,(4.7\text{ac})\,(43{,}560\text{ft}^2/\text{ac})\,(1\text{ft}/12\text{in}) \\ &= \underline{10{,}134}\ \text{ft}^3 = \underline{0.23}\ \text{ac-ft} \end{aligned}$$

Recharge, Re_v

Volume-based approach

$$\begin{aligned} Re_v &= (0.25)(A)(I) / 12 \\ &= (0.25)\,(12.8)\,(0.68)/12 \\ &= \underline{7{,}900}\ \text{ft}^3 = \underline{0.18}\ \text{ac-ft} \end{aligned}$$

As stated above, the recharge requirement at the site is being provided by an infiltration trench (see Appendix C3 for the design example). Therefore, subtract the recharge volume (based on the proportional area of Catchment B) from the Catchment B water quality volume (see above).

Net WQ_v to be treated by the bioretention facility is:
Net WQ_v = 10,134ft^3 – (4.7ac/12.8ac)(7,900ft^3) = **7,233 ft^3**

- Develop Site Hydrologic Input Parameters and Perform Preliminary Hydrologic Calculations (see Table C.14).

Note: For this design example, the 10-year peak discharge will be used to size the bypass flow splitter. Any hydrologic models using SCS procedures, such as TR-20, HEC-HMS, or HEC-1, can be used to perform preliminary hydrologic calculations

Table C.14. Site Hydrology

Condition	Area	CN	Q_1	Q_{10}	Q_{100}
	ac		*cfs*	*cfs*	*cfs*
Pre-developed	12.8	58	0.3	2	13
Post-Developed Catchment A	8.1	83	9.4	19	44
Post-Developed Catchment B	4.7	83	5.5	11	26
Post-Developed Total	12.8	83	15	30	70

Step 2. Determine if the development site and conditions are appropriate for the use of a bioretention area.

Site Specific Data:

Existing ground elevation at practice location is 222.0 feet, mean sea level. Soil boring observations reveal that the seasonally high water table is at 212.0 feet and underlying soil is silt loam (ML). Adjacent creek invert is at 211.0 feet.

Step 3. Confirm local design criteria and applicability.

The site drains to Lake Champlain, where phosphorus reduction has been identified as a major pollutant reduction goal. In addition, on-site snow storage and treatment of melt water has been raised as a local concern.

Step 4. Determine size of bioretention filter area.

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

Where:
A_f = surface area of filter bed (ft^2)
d_f = filter bed depth (ft)
k = coefficient of permeability of filter media (ft/day)
h_f = average height of water above filter bed (ft)
t_f = design filter bed drain time (days) (2 days is recommended)

A_f = (7,233 ft^3)(5') / [(0.5'/day) (0.25' + 5') (2 days)] (With k = 0.5'/day, h_f = 0.25', t_f = 2 days)

A_f = 6,889 sq ft

Step 5. Set design elevations and dimensions.

Assume a roughly 2 to 1 rectangular shape. Given a filter area requirement of 6,889 sq ft, say facility is roughly 60' by 115' (see Figure C.20). Set top of facility at 221.0 feet, with the berm at 222.0 feet. The facility is 5' deep, which will allow 4' of separation distance over the seasonally high water table. See Figure C.21 for a typical section of the facility.

Step 6. Design conveyance to facility.

Stormwater treatment practices can be either on or off-line. On-line facilities are generally sized to receive, but not necessarily treat, the 10-year event. Off-line facilities are designed to receive a more or less exact flow rate through a weir, channel, manhole "flow splitter", etc. The facility in this example is situated to receive direct runoff from the parking lot and associated landscaped areas; therefore, it is necessary to design the facility to pass the 10-year event (11.0 cfs), (i.e., no special splitter sizing is incorporated).

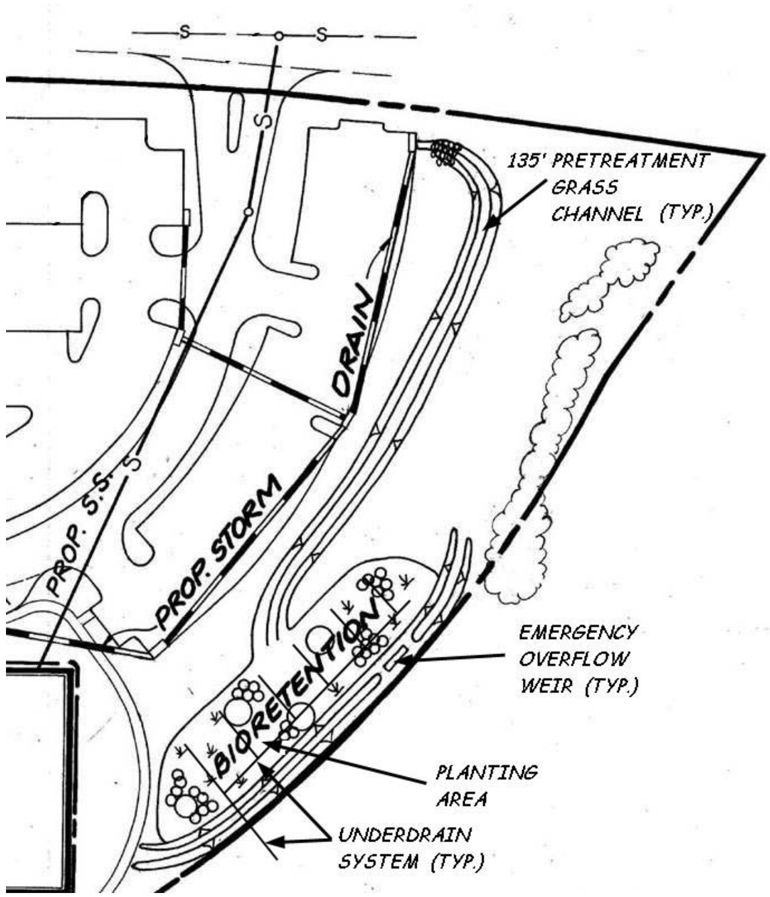


Figure C.20. Plan View of Bioretention Facility

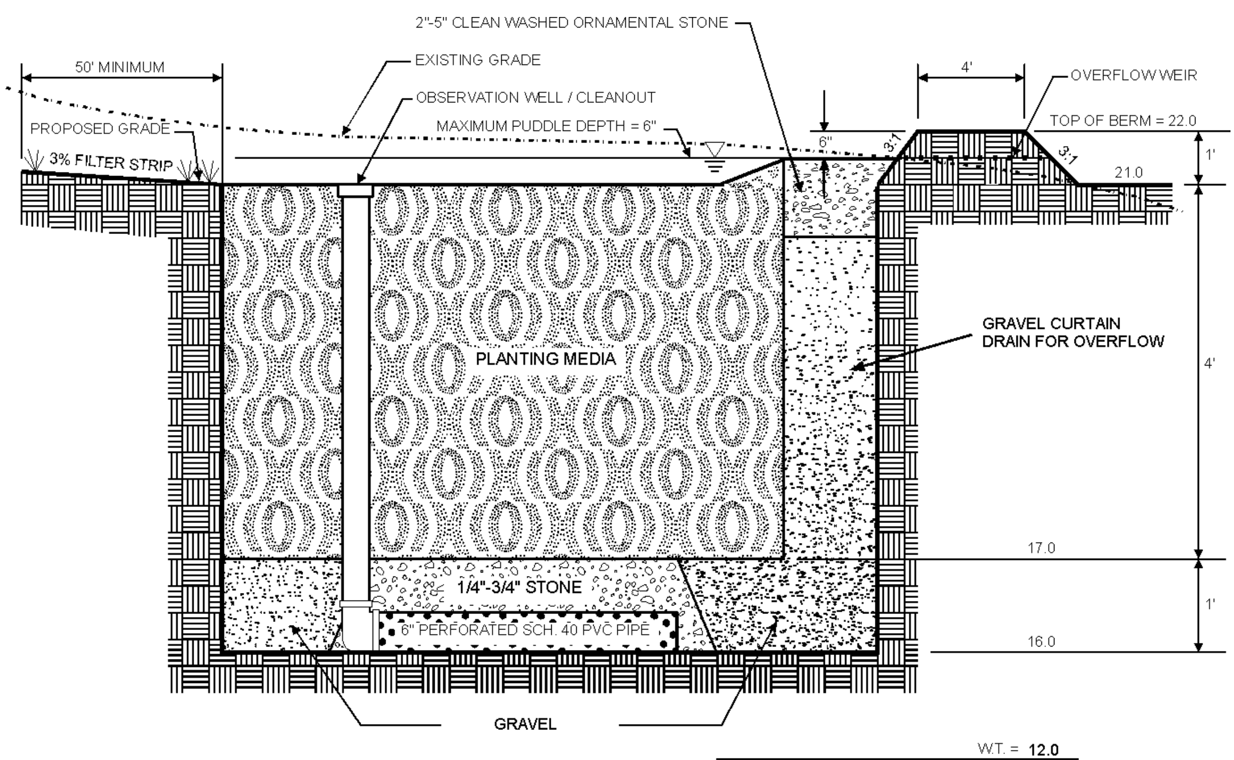


Figure C.21. Typical Section of Bioretention Facility

Step 7. Design pretreatment.

Pretreat with a grass channel. Using guidance in Section 2.7.5.D and 2.7.4.C of the *VT Stormwater Management Manual - Volume I,* size channel length based on imperviousness of 68%, channel slope less than 1%, and drainage area of about 4.7 acres as follows:

- For a 4.7-acre site with 68% impervious cover (i.e., a runoff coefficient, R_v, of 0.66), the peak flow associated with the water quality rainfall of 0.9" is approximately 3.9 cfs (see Appendix D6 for guidance on computing water quality storm peak discharge).
- For a 5' wide (bottom width) channel with 3:1 sideslopes (horizontal:vertical) and a slope of 0.75%, the velocity is approximately 0.9 fps (this can be determined using nomographs, Manning's equation, or available computer software packages). Note: the designer may need to balance the hydraulic parameters such as channel slope, bottom width, and sideslopes with the site constraints such local topography and available space to arrive at an acceptable design. In this example, the initial target slope of 1% was reduced to 0.75%.
- Therefore, using a required residence time of 10 minutes (600 seconds), the required length of channel for 100% of the WQ_v would be 0.9 fps x 600 sec = 540 ft.
- For pretreatment requirements, 25% of the WQ_v is needed, or 0.25 x 540 ft = 135 ft.

Therefore, for this example, a grass channel length of at least <u>135 feet</u> is required.

- In addition to sizing the grass channel for the pretreatment criteria, it is necessary to document that the channel design will convey the 10-yr peak discharge (11 cfs) to the bioretention facility at non-erosive velocities. For this example, given the channel geometry above, the velocity associated with the 10-yr flow was calculated to be 2.6 fps. Therefore, the grass specification for the channel must be able to withstand this velocity without eroding (see Appendix D7).

Step 8. Size underdrain area.

As a rule of thumb, the length of underdrain should be based on 10% of the A_f or 689 sq ft and a 3 ft wide zone of influence (see Figures C.20 and C.21). Using 6" perforated plastic pipes surrounded by a three-foot-wide gravel bed, 10' on center (o.c.), yields the following length of pipe:

(689 sq ft)/3' per foot of underdrain = <u>230' of perforated underdrain</u>

Step 9. Overdrain design.

To ensure against the planting media clogging, design a small ornamental stone window of 2" to 5" stone connected directly to the gravel curtain drain. This area is based on 5% of the A_f or 345 sq ft. Say 12' by 30' (see Figures C.20 and C.21).

Step 10. Emergency storm weir design.

The parking area curb and gutter is sized to convey the 10-year event to the facility. Should filtering rates become reduced due to facility age or poor maintenance, an overflow weir is provided to pass the 10-year event. Size this weir with 6" of head, using the weir equation.

$Q = CLH^{3/2}$

Where: C = 3.1
Q = 11.0 cfs
H = 6"

Solve for L: $L = Q / [(C)(H^{3/2})]$ or $(11.0 \text{ cfs}) / [(3.1)(.5)^{1.5}] = 10.03'$ (say 10')

Outlet protection in the form of riprap or a plunge pool/stilling basin should be provided to ensure non-erosive velocities.

Step 11. Choose plants for planting area.

Choose plants based on factors such as whether native or not, resistance to drought and inundation, cost, aesthetics, maintenance, etc. Select species locations (i.e., on center planting distances) so species will not “shade out” one another. Do not plant trees and shrubs with extensive root systems (e.g., willows) near pipe work. A potential plant list for this site is presented in Appendix A2.

Appendix D: Assorted Design Tools

Appendix D1 : Infiltration and Bioretention Testing Requirements

General Notes Pertinent to All Testing

1. For infiltration practices, a minimum field infiltration rate (f_c) of 0.5 inches per hour is required; areas yielding a lower rate preclude these practices. If the minimum f_c exceeds two inches per hour, half of the WQ_v must be treated by an upstream STP that does not allow infiltration. For filters, no minimum infiltration rate is required if these facilities are designed with a “day-lighting” underdrain system; otherwise these facilities require a 0.5 inch per hour rate.

2. Number of required borings is based on the size of the proposed facility. Testing is done in two phases, (1) Initial Feasibility, and (2) Concept Design Testing.

3. Testing is to be conducted by a qualified professional. This professional must either be a registered professional engineer in the State of Vermont, a soil scientist or geologist also licensed in the State of Vermont.

Initial Feasibility Testing

Feasibility testing is conducted to determine whether full-scale testing is necessary, and is meant to screen unsuitable sites, and reduce testing costs. A soil boring is not required at this stage. However, a designer or landowner may opt to engage Concept Design Borings per Table D.1 at his or her discretion, without feasibility testing.

Initial testing involves either one field test per facility, regardless of type or size, or previous testing data, such as the following:

* septic percolation testing on-site, within 200 feet of the proposed STP location, and on the same contour [can establish initial rate, water table and/or depth to bedrock]
* previous written geotechnical reporting on the site location as prepared by a qualified geotechnical consultant
* NRCS County Soil Mapping *showing an unsuitable soil group* such as a hydrologic group “D” soil in a low-lying area, or a Marlboro Clay

If the results of initial feasibility testing as determined by a qualified professional show that an infiltration rate of greater than 0.5 inches per hour is probable, then the number of concept design test pits should be per the following table. An encased soil boring may be substituted for a test pit, if desired.

Table D.1. Infiltration Testing Summary

Type of Facility	Initial Feasibility Testing	Concept Design Testing (initial testing yields a rate greater than 0.5"/hr)	Concept Design Testing (initial testing yields a rate lower than 0.5"/hr)
I-1 (trench)	1 field percolation test, test pit not required	1 infiltration test and 1 test pit per 50' of trench	practice not acceptable
I-2 (basin)	1 field percolation test, test pit not required	1 infiltration test* and 1 test pit per 200 sf of	practice not acceptable
F-1 (sand filter	1 field percolation test, test pit not required	test pit per 200 sf of filter area (no underdrains required**)	required
F-5 (bioretention)	1 field percolation test, test pit not required	1 infiltration test and 1 test pit per 200 sf of filter area (no underdrains required**)	underdrains required

*feasibility test information already counts for one test location
** underdrain installation still strongly suggested

Documentation

Infiltration testing data should be documented, which should also include a description of the infiltration testing method, if completed. This is to ensure that the tester understands the procedure.

Test Pit/Boring Requirements

a. Excavate a test pit or dig a standard soil boring to a depth of 4 feet below the proposed facility bottom.

b. Determine depth to groundwater table (if within 4 feet of proposed bottom) upon initial digging or drilling, and again 24 hours later.

c. Determine USDA or Unified Soil Classification System textures at the proposed bottom and 4 feet below the bottom of the STP.

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

e. The soil description should include all soil horizons.

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

Infiltration Testing Requirements

a. Install casing (solid 6 inch diameter) to 24" below proposed STP bottom.

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

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

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

e. The location of the test should correspond to the STP location.

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

Laboratory Testing

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

Bioretention Testing

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

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

a) USDA minimum textural analysis requirements: A textural analysis is required from the site stockpiled topsoil. If topsoil is imported, then a texture analysis should be performed for each location where the topsoil was excavated.

   Minimum requirements:
   sand 35 - 60%
   silt 30 - 55%
   clay 10 - 25%

b) The soil should be a uniform mix, free of stones, stumps, roots or other similar objects larger than one inch.

c) Consult the bioretention construction specifications (Appendix B3) for further guidance on preparing the soil for a bioretention area.

Appendix D2 : Short-cut Method for a Wetland Drawdown Assessment

(Use Pond Design Example [see Appendix C1] as a basis for this analysis).

This section presents a simple method for calculating whether a stormwater pond or wetland has an appropriate water balance over a 30-day period without rainfall.

Table D.2. Data from Pond Design Example for Sample Water Balance Analysis

Drainage Area = 65.1 ac
Dev. CN = 78
2-yr. Vol of Runoff = 0.8"
2-yr. Rainfall Event = 2.5"
Pond Surface Area = 0.32 ac
Pond Permanent Pool Storage = 1.17 ac-ft.

1. Check maximum drawdown during periods of high evaporation and during an extended period of no appreciable rainfall.
2. The change in storage within a pond = Inflows - Outflows
3. Potential inflows: Runoff, baseflow and rainfall
4. Potential outflows: Infiltration, surface overflow and evaporation (and evapotranspiration)
5. Assume no inflow from baseflow, no losses for infiltration and because only the permanent pool volume is being evaluated, no losses for surface overflows:
6. Therefore, storage = runoff - evaporation

For permanent pool volume = 1.17 acre-ft = 0.22 watershed inches, a rainfall event yielding 0.22" or more of runoff will fill pond.

Evaporation Rates for Vermont Ponds (based on Burlington, VT data): (from Ferguson and Debo, "On-Site Stormwater Management," 1990).

Month	April	May	June	July	Aug.	Sept.
Precip.(ft)	0.25	0.29	0.31	0.34	0.28	0.31
Pond Evap.(ft)	0.23	0.36	0.44	0.49	0.39	0.25

Look at worst case (based on assumption of maximum evaporation and no rain): July

Runoff volume = P * E
Where:
P = precipitation
E = Efficiency of runoff (assumed to be ratio of SCS runoff depth to rainfall depth for 2-year storm)
For CN = 78, Vr (2) = 0.8"
E = 0.8"/2.5" = .32

Under average conditions:

Inflow: (.34 ft) (.32) = 0.11 ft over entire site area: (0.11 ft) (65.1 ac/ 12"/ft) = 0.6 ac-ft
Outflow: (surface area) (evap losses) = (0.32 ac) (0.49 ft) = 0.16 ac-ft

Therefore, drainage area is adequate to support wet pond during normal conditions.

For extended period with no rainfall: (assume 45 days during June/July period)
Avg. evaporation: (0.44 ft + 0.49 ft) / 2 = 0.47 ft / 30.5 days = 0.015 ft/day
for 45 days, loss = 45 * .015 ft/day = 0.68 ft

Assume permanent pool will drop between .7 ft and 1.0 ft for this period. Specify vegetation for the aquatic shelves (to 12"), which can tolerate periods of drawdowns.

REFERENCES

Ferguson, B. And T.N. Debo. 1990. On-Site Stormwater Management - Applications for Landscape and Engineering. Van Nodstrandt, Reinhold, New York.

App nd 40% TP R irement

Basis of Recommendation for Proposed Practices

Proposed practices were selected primarily on their ability to remove 80% of total suspended solids (TSS) and 40% of total phosphorus (TP) from stormwater runoff. These practices also tend to have the highest removal capabilities for other common pollutants such as trace metals. The primary data source for removal efficiencies is the Center for Watershed Protection's *National Pollutant Removal Performance Database* (Winer, 2000; Table D.3)[1]. In some cases, practices with a reported TSS removal of less than 80% are included. This is particularly true when the reported removal is impacted by some poorly designed practices. In other cases, while there are no monitoring data available, there is a presumption of performance based on similarity in design to other practices with performance data. The "notes" column in Table D.3 documents these considerations and assumptions.

Removal of other pollutants may be an important consideration for many applications as well. For most pollutants, insufficient data are available to make conclusions about *individual* practices. Therefore, this Appendix presents data or presumed removals for the practice *groups* as guidance on appropriate STP selection. Similar to TSS and TP, these data are based on pollutant removals reported in Winer (2000) (Table D.4).

[1] In 1997, the Center completed the first version of a national pollutant removal performance database for stormwater management treatment practices. The database contained entries from 123 performance monitoring studies for ponds, stormwater wetlands, infiltration, filters and open channel practices. Recently, the Center has completed an update of the database with data from additional studies and a somewhat stricter threshold on accepting a performance study. The database now includes data from studies where at least five storm events were sampled. The updated database includes 139 entries. In addition, data fields with pertinent information such as drainage area, impervious cover, total treatment storage volume, pollutant effluent concentration, and other factors helpful for statistical analysis were updated and/or added.

Table D.3. Total Suspended Sediment and Total Phosphorus Removal of Acceptable Stormwater Treatment Practices for Water Quality

Group	Practice	N	TSS	TP	Notes
Pond	Detention Pond		ND	ND	This practice is presumed to have removal rates similar to the wet extended detention pond. While this practice has not been monitored the pollutant removal mechanisms are similar.
	Wet Pond	29	79%	49%	Wet pond performance is highly variable, with some practices in the database with poor design features. Practices that follow the recommended criteria will exceed 80% TSS removal consistently (See Chapter 3 of Manual).
	Wet Extended Detention Pond	14	80%	55%	
	Multiple Pond System	1	91%	76%	Although only based on one study, it is presumed that this practice will consistently exceed the 80% removal. The design should result in slightly higher removals than the wet pond.
	Pocket Pond	5	87%	ND	Pocket ponds are a subgroup of other pond designs, including all ponds with drainage areas less than 10 acres.
Wetland	Shallow Marsh	23	83%	43%	
	Extended Detention Wetland	4	69%	39%	The database is dominated by highly in the database treats more than 0.15 watershed inches. Even among these, one practice achieves 80% removal. It is commonly accepted that practices that follow required performance criteria will achieve 80% TSS removal consistently.
	Pond/ Wetland System	10	71%	56%	The current database is biased by poorly designed facilities. Removals similar to the Wet Pond and Shallow Marsh designs are anticipated. Also, removals were highly variable. Four of the 10 practices actually had higher than 90% removals. It is commonly accepted that practices that follow required performance criteria will achieve 80% TSS removal consistently.

Group	Practice	N	TSS Removal	TP Removal	Notes
Wetland cont.	Gravel Wetland	2	83%	64%	
Infiltration	Infiltration Trench	3	ND	100%	Infiltration practices are difficult to monitor, but are presumed to have high removal rates based on filtration processes of the soil and pollutant land application studies.
	Infiltration Basin	0	ND	ND	
Filtering Practices	Surface Sand Filter	8	87%	59%	
	Underground Sand Filter	0	ND	ND	Presumed similar removal to other filtering practices.
	Perimeter Sand Filter	3	79%	41%	Result impacted by one study with very low inflow concentrations. Presumed similar removal to other filtering practices.
	Organic Filter	7	88%	61%	
	Bioretention	1	ND	65%	Presumed similar removal to other filtering practices.
Open Channels	Dry Swale	4	93%	83%	
	Wet Swale	2	74%	28%	The two wet swale designs in the database actually achieve relatively low outflow concentrations. Results are biased by relatively low inflow concentrations.
	Grass Channel	3	68%	29%	The current database is slightly biased by poorly designed facilities. Removals similar to the Dry Swale are anticipated with appropriate design.

Notes: Removals represent median values from Winer (2000)
N = number of studies
TSS = total suspended solids; TP = total phosphorus
ND = No Data

Table D.4. Percent Removal of Key Pollutants by Practice Group

Practice	Total Nitrogen [%]	[%]	[%]	Hydrocarbons [%]
Detention Ponds		26	78[2]	ND
	33	62		81[2]
Stormwater Wetlands	30	42		85[2]
Filtering Practices	38	69		84[2]
Infiltration P	5	99[2]	ND	ND
Water Quality Swales and Grass Channels[4]	84[2]	61		62[2]

1. Average of zinc and copper. Only zinc for infiltra
2. Based on fewer than five data points (i.e., indep
3 Includ e hich t on th f ap ermont.
4. Higher removal rates for dry swales.

ND: No Data

R movals ed valu m W 200

Appendix D4: Industrial Categories required to obtain Multi-Sector General Permit for Stormwater Discharges

The Multi-Sector General Permit (MSGP) is a Federally mandated National Pollutant Discharge Elimination System (NPDES) five-year permit that covers new and existing discharges of stormwater associated with certain types of industrial activity. The permit is required by industries identified by EPA in CFR 122.26(b)(14)(i through xi) that have a stormwater discharge to either a municipal separate storm sewer system (MS4) or to a receiving water.

Identified by EPA in CFR 122.26(b)(14)(i through xi):

(i) Facilities subject to stormwater effluent limitations...

(ii)
- 24 (except 2434) Lumber and Wood Products, Except Furniture
  - 2434 - Wood Kitchen Cabinets
- 26 (except 265 and 267) Paper and Allied Products
  - 265 - Paperboard Containers and Boxes
  - 267 - Converted Paper and Paperboard Products, Except Containers and Boxes
- 28 (except 283) Chemicals and Allied Products
  - 283 - Drugs
- 29 Petroleum Refining and Related Industries
- 31 Leather and Leather Products
- 32 (except 323) Stone, Clay, Glass, and Concrete Products
  - 323 - Glass Products made of Purchased Glass
- 33 Primary Metal Industries
- 344 Fabricated Structural Metal Products
- 373 Ship and Boat Building and Repairing

(iii)
- 10 Metal Mining
- 12 Coal Mining
- 13 Oil and Gas Extraction (including facilities where stormwater comes into contact with overburden or raw materials)
- 14 Mining and Quarrying of Nonmetallic Minerals, Except Fuels

(iv) Hazardous waste treatment, storage or disposal facilities

(v) Landfills, land application sites, and open dumps

(vi) Facilities involved in the recycling of materials...
including, but not limited to;
5015 Motor Vehicle Parts, Used
5093 Scrap and Waste Materials

(vii) Steam electric power general facilities...
No specific SIC codes cited.

(viii) Transportation facilities classified as:
40 Railroad Transportation
41 Local and Suburban Transit and Inter-urban Highway Passenger Transportation
42 (except 4221-4225) Motor Freight Transportation and Warehousing
4221 - Farm Product Warehousing and Storage
4222 - Refrigerated Warehousing and Storage
4225 - General Warehousing and Storage
43 United States Postal Service
44 Water Transportation
45 Transportation by Air
5171 Petroleum Bulk Stations and Terminals

(ix) Treatment works treating domestic sewage...

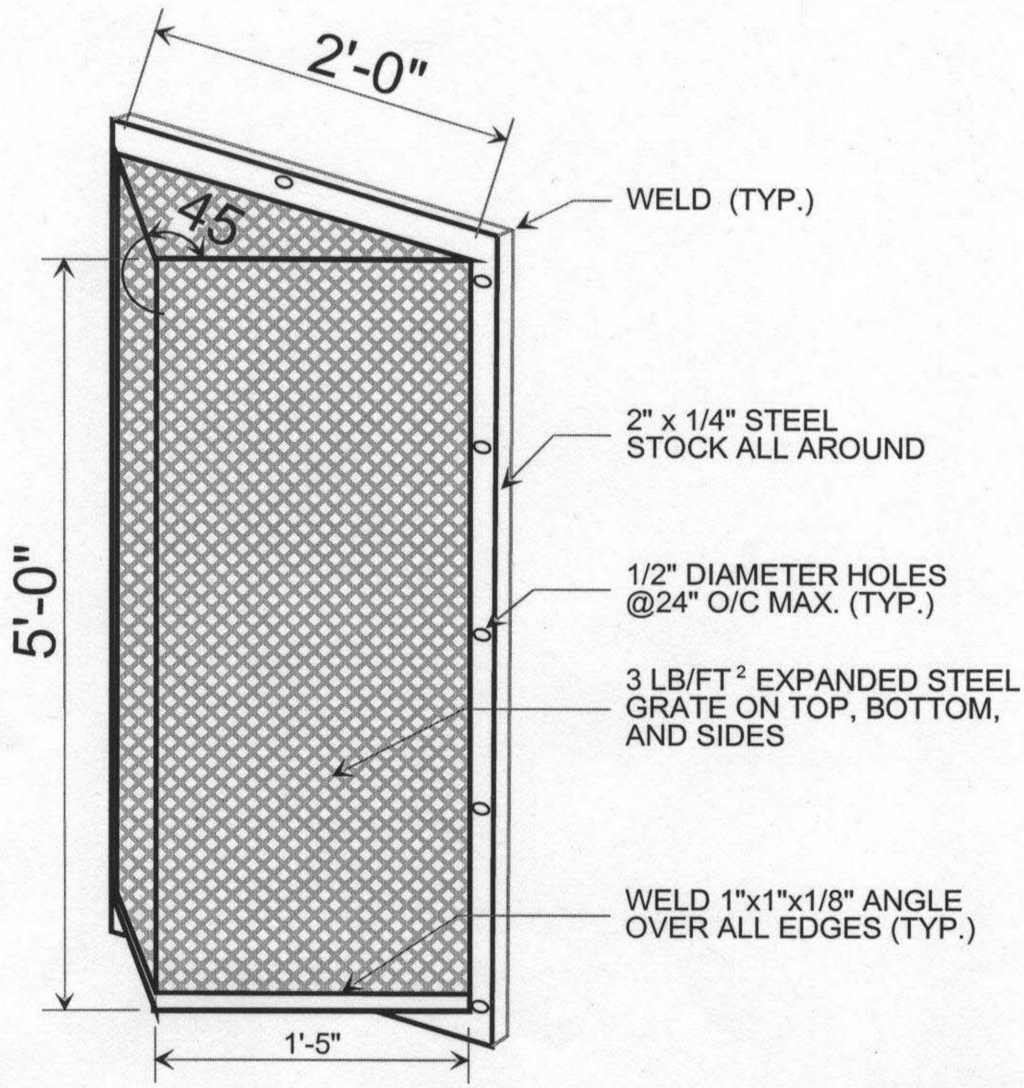
(x) Construction Activity

(xi) 20 Food and Kindred Products
21 Tobacco Products
22 Textile Mill Products
23 Apparel and other Finished Products made from Fabrics and Similar Materials
2434 Wood Kitchen Cabinets
25 Furniture and Fixtures
265 Paperboard Containers and Boxes
267 Converted Paper and Paperboard Products, Except Containers and Boxes
27 Printing, Publishing, and Allied Industries
283 Drugs
285 Paints, Varnishes, Lacquers, Enamels, and Allied Products
30 Rubber and Miscellaneous Plastics Products
31 (except 311) Leather and Leather Products
311 - Leather Tanning and Finishing
323 Glass Products, made of Purchased Glass
34 (except 3441) Fabricated Metal Products, Except Machinery and Transportation Equipment
3441 - Fabricated Structural Metal
35 Industrial and Commercial Machinery and Computer Equipment
36 Electronic and other Electrical Equipment and Components, Except Computer Equipment
37 (except 373) Transportation Equipment
373 - Ship and Boat Building and Repairing

38 Measuring, Analyzing, and Controlling Instruments; Photographic, Medical and Optical Goods; Watches and Clocks
39 Miscellaneous Manufacturing Industries
4221 Farm Product Warehousing and Storage
4222 Refrigerated Warehousing and Storage
4225 General Warehousing and Storage

Appendix D5: Miscellaneous Details

Miscellaneous Design Schematics for Compliance with Performance Criteria



NOTES FOR TRASH RACK

1. TRASH RACK TO BE CENTERED OVER OPENING.
2. STEEL TO CONFORM TO ASTM A-36.
3. ALL SURFACES TO BE COATED WITH ZRC COLD GALVANIZING COMPOUND AFTER WELDING.
4. TRASH RACK TO BE FASTENED TO THE WALL WITH 1/2" MASONRY ANCHORS. TRASH RACK TO BE REMOVABLE.

Figure D.1. Trash Rack Protection for Low Flow Orifice

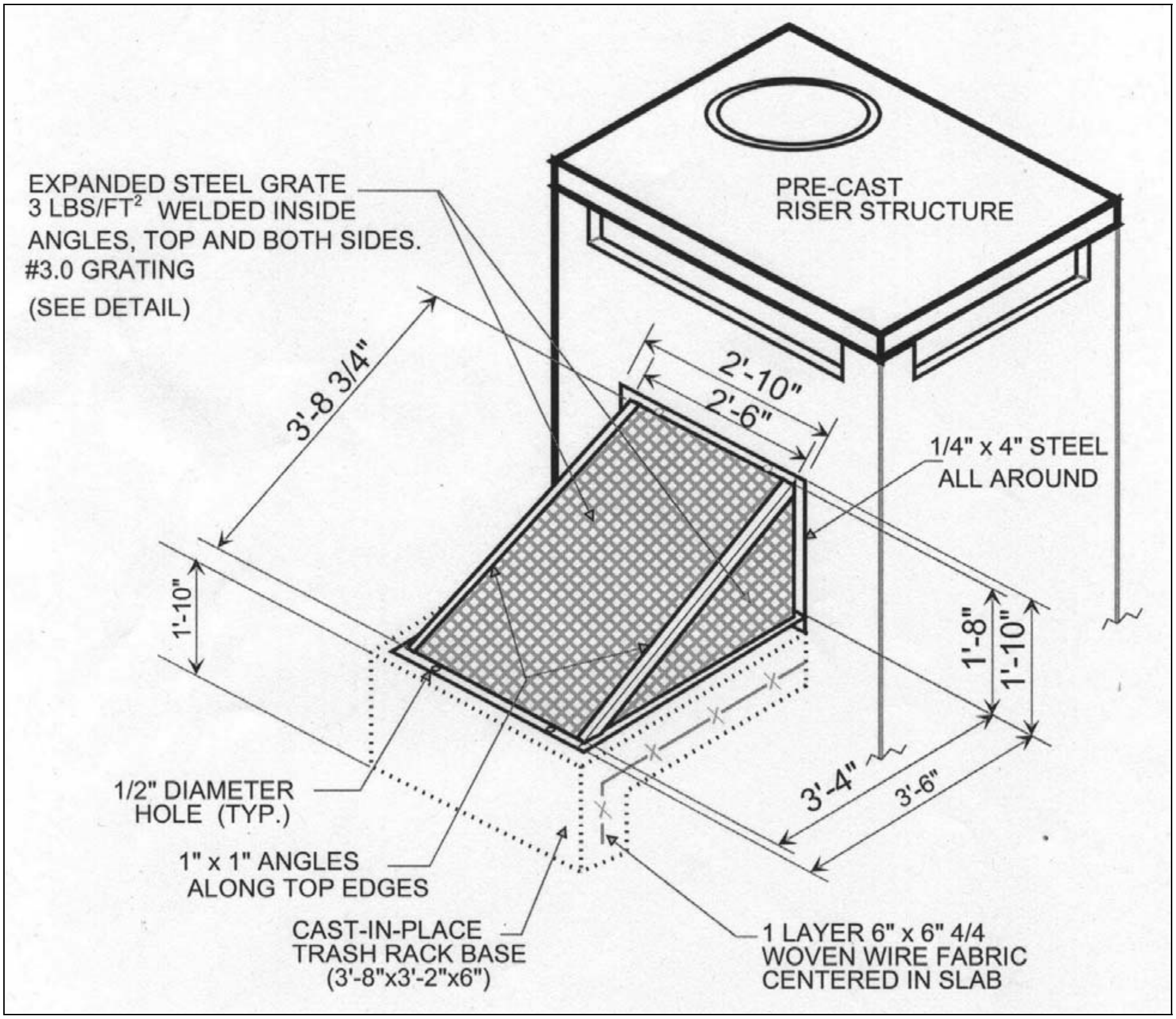


Figure D.2. Expanded Trash Rack Protection for Low Flow Orifice

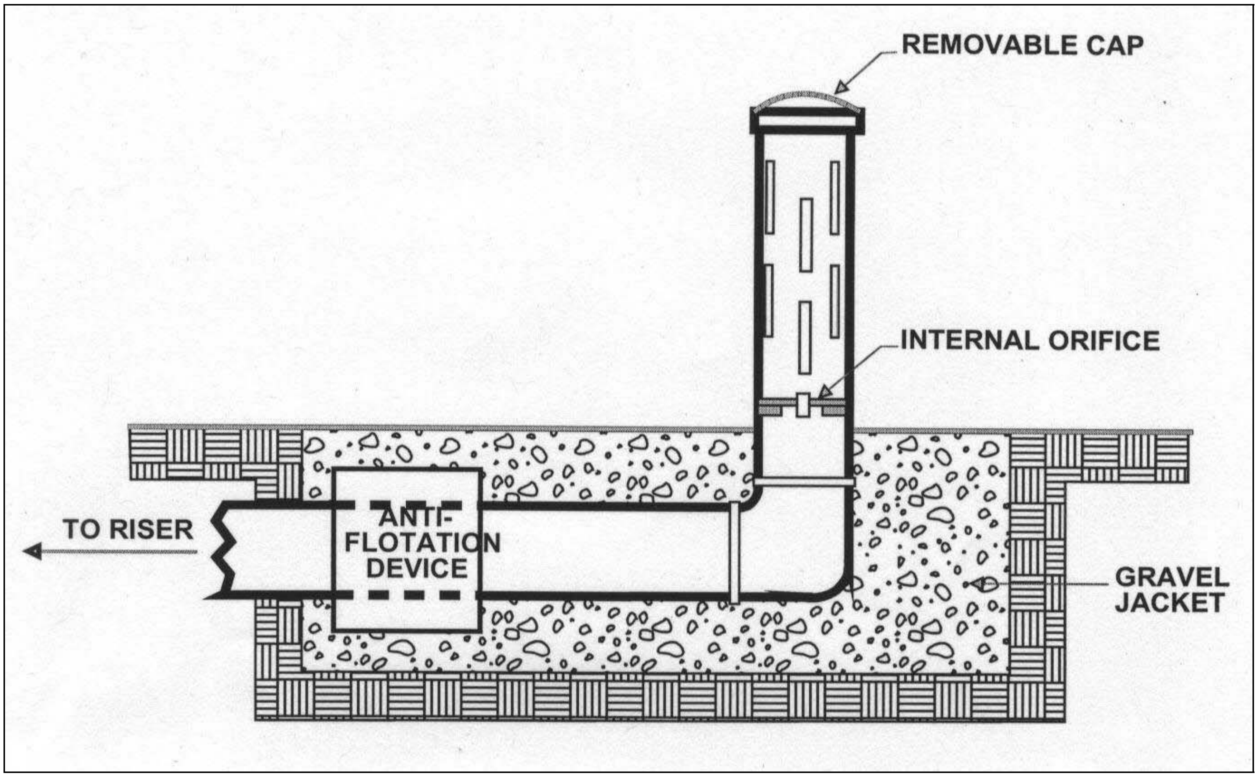
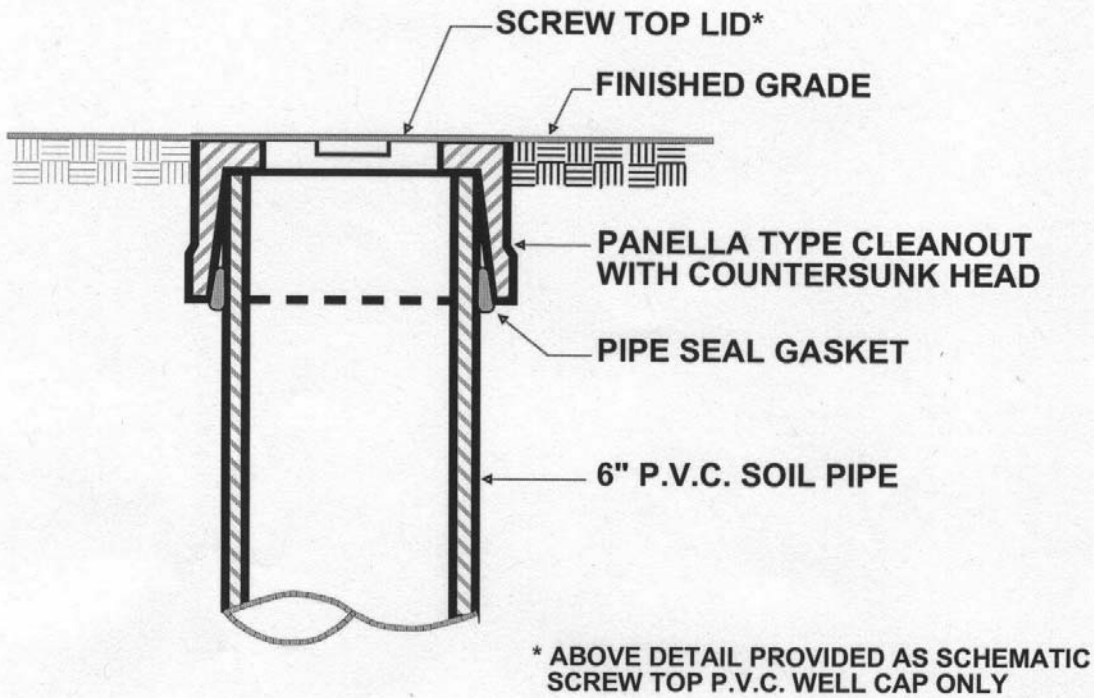


Figure D.3. Internal Control for Orifice Protection



EACH OBSERVATION WELL / CLEANOUT SHALL INCLUDE THE FOLLOWING:

1. FOR AN UNDERGROUND FLUSH MOUNTED OBSERVATION WELL / CLEANOUT, PROVIDE A TUBE MADE OF NON-CORROSIVE MATERIAL, SCHEDULE 40 OR EQUAL, AT LEAST THREE FEET LONG WITH AN INSIDE DIAMETER OF AT LEAST 6 INCHES.
2. THE TUBE SHALL HAVE A FACTORY ATTACHED CAST IRON OR HIGH IMPACT PLASTIC COLLAR WITH RIBS TO PREVENT ROTATION WHEN REMOVING SCREW TOP LID. THE SCREW TOP LID SHALL BE CAST IRON OR HIGH OMPACT PLASTIC THAT WILL WITHSTAND ULTRA-VIOLET RAYS.

Figure D.4. Observation Well for Infiltration Practices

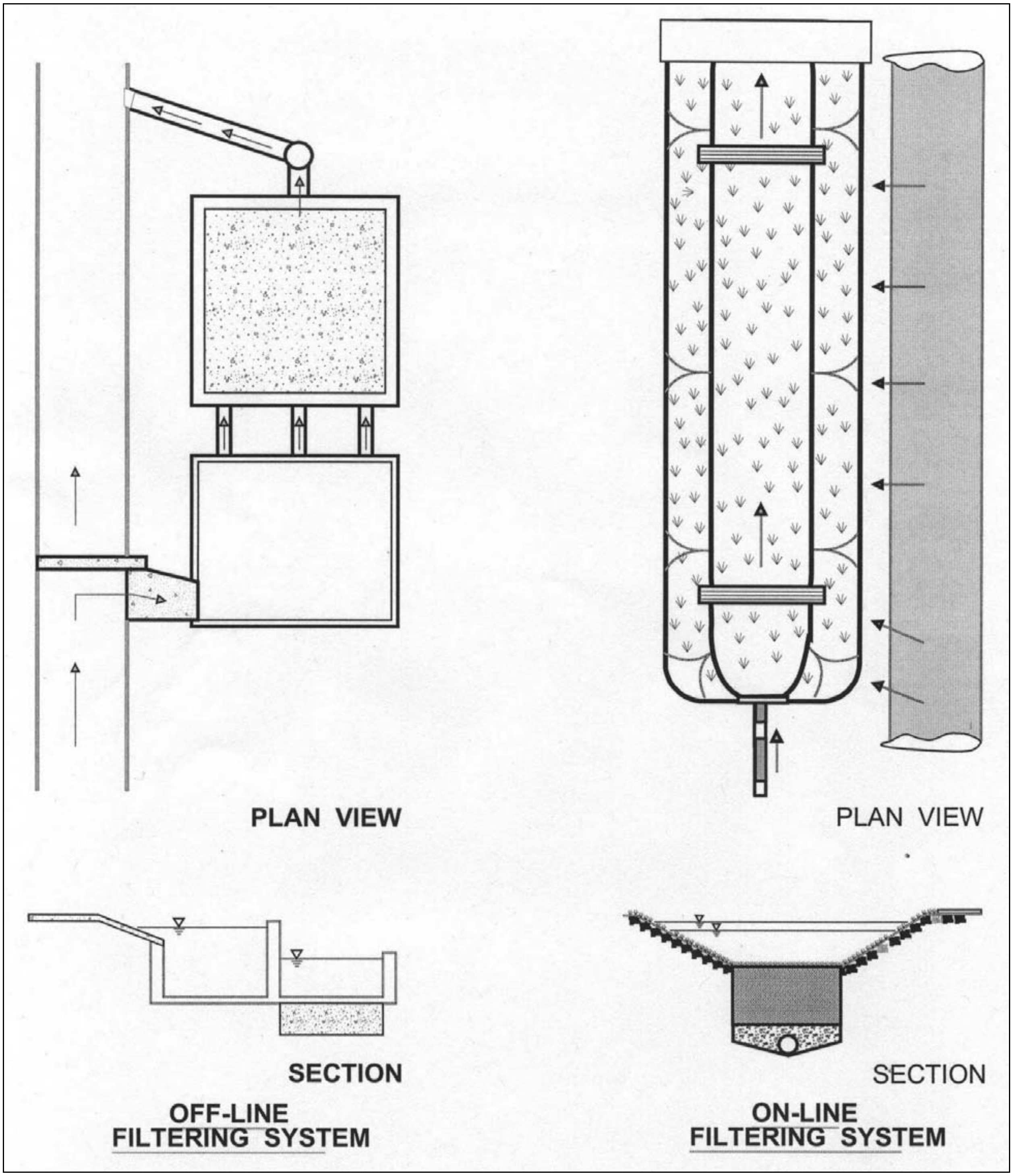


Figure D.5. On-Line Versus Off-Line Schematic

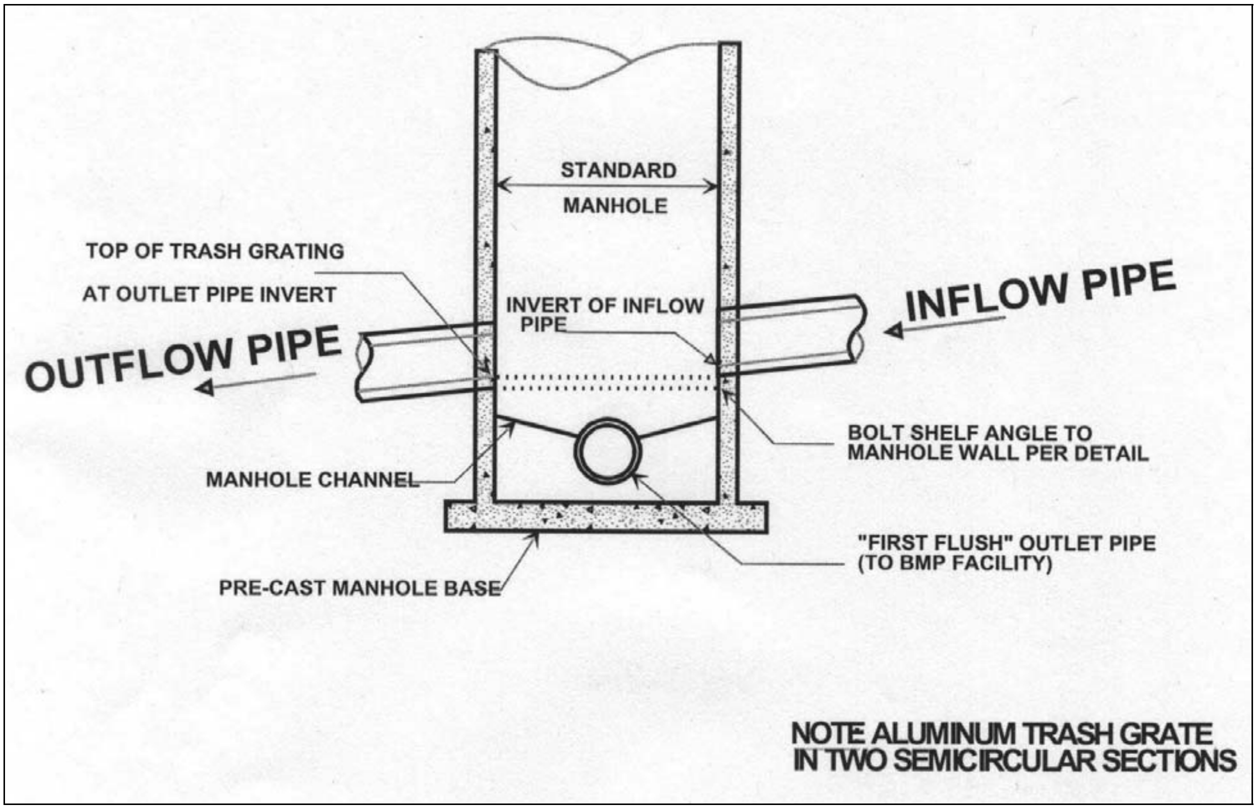


Figure D.6. Isolation Diversion Structure

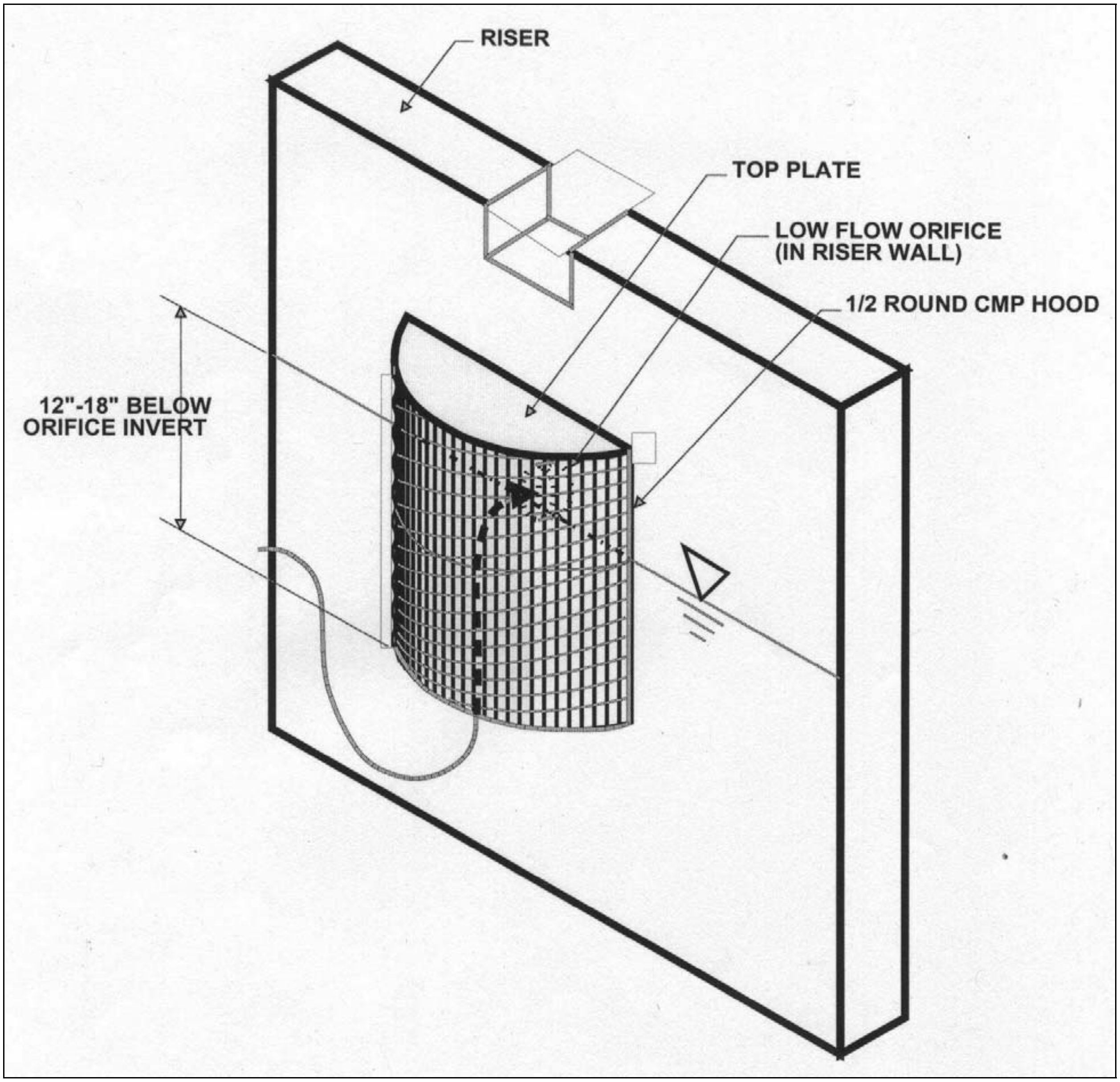


Figure D.7. Half Round CMP Hood

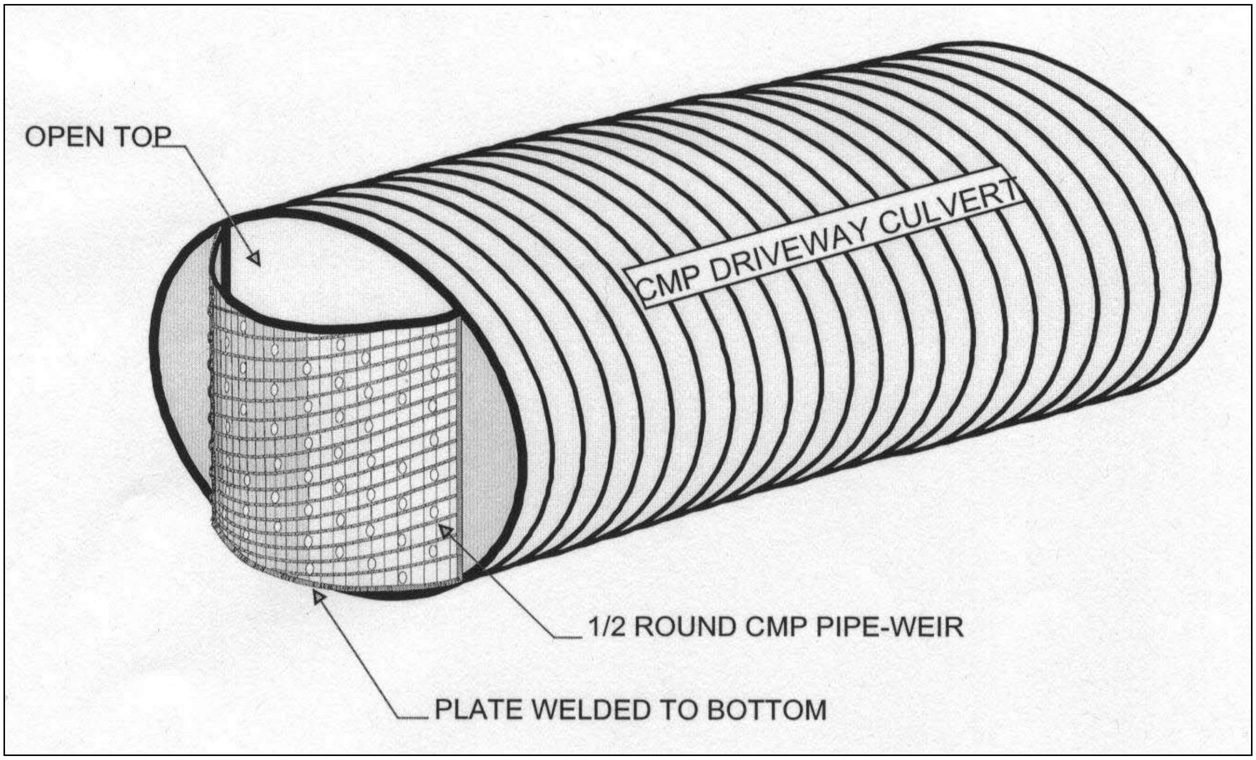


Figure D.8. Half Round CMP Weir

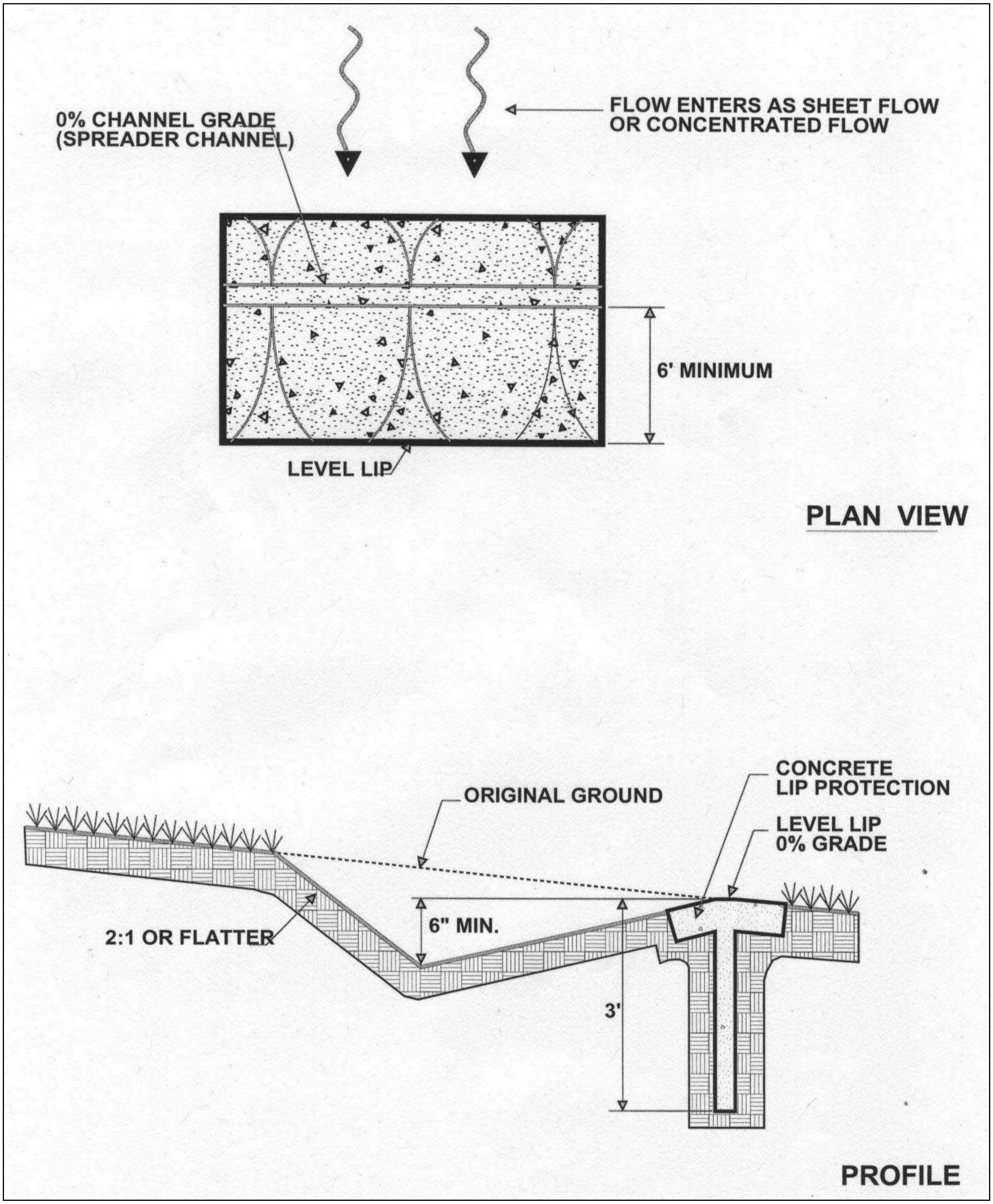


Figure D.9. Concrete Level Spreader

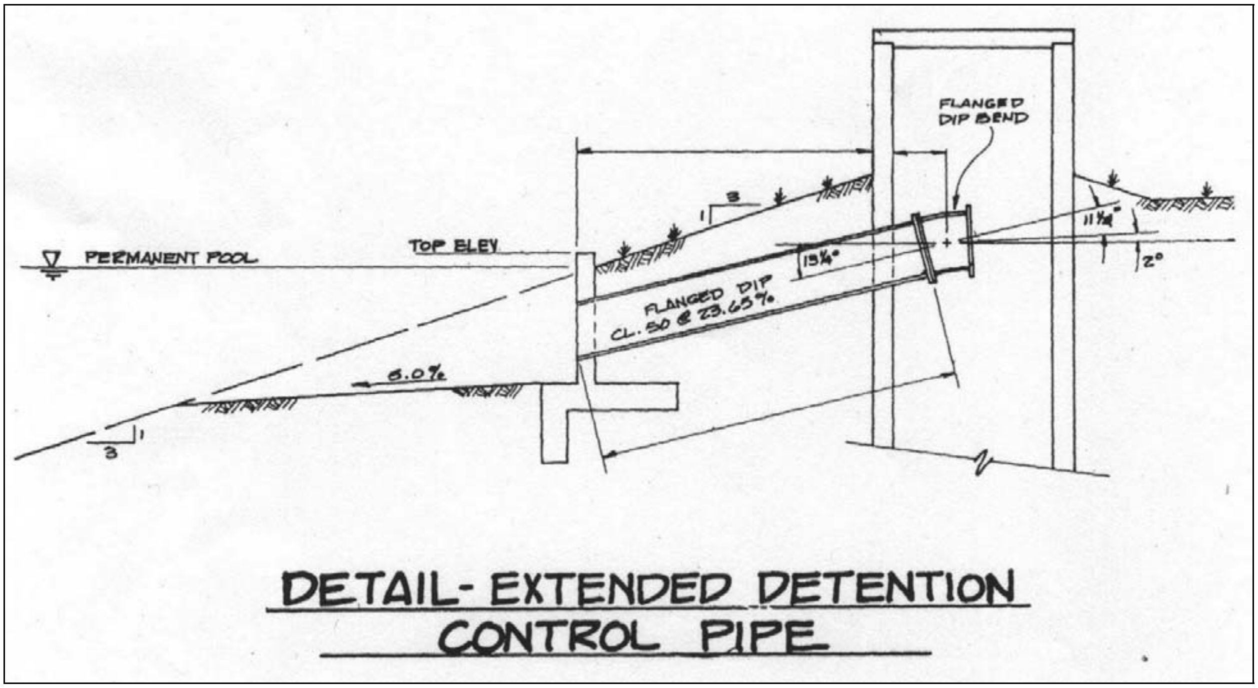


Figure D.10. Example of Reverse Slope Pipe

Appendix D6: Hydrologic Analysis Tools

This Appendix presents two hydrologic and hydraulic analysis tools that can be used to size stormwater treatment practices (STPs). The first is the TR-55 "short-cut" sizing technique, used to size practices designed for extended detention, slightly modified to incorporate the flows necessary to provide channel protection. The second is a method used to determine the peak flow from water quality storm events. (This is often important when the water quality storm is diverted to a water quality practice, with other larger events bypassed).

Storage Volume Estimation

This section presents a modified version of the TR-55 (NRCS, 1986) short cut sizing approach. The method was modified by Harrington (1987), for applications where the peak discharge is very small compared with the uncontrolled discharge. This often occurs in the 1-year, 24-hour detention sizing.

Using TR-55 guidance, the unit peak discharge (q_u) can be determined based on the Curve Number and Time of Concentration (Figure D.11). Knowing qu and T (extended detention time), q_o/q_i (peak outflow discharge/peak inflow discharge) can be estimated from Figure D.12.

Then using q_o/q_i, Figure D.13 can be used to estimate V_s/V_r. For a Type II or Type III rainfall distribution, V_s/V_r can also be calculated using the following equation:

$$V_S/V_r = 0.682 - 1.43\,(q_O/q_I) + 1.64\,(q_O/q_I)^2 - 0.804\,(q_O/q_I)^3$$

Where: V_S = required storage volume (acre-feet)
V_r = runoff volume (acre-feet)
q_O = peak outflow discharge (cfs)
q_i = peak inflow discharge (cfs)

The required storage volume can then be calculated by:

$$V_S = \frac{(V_S/V_r)(Q_d)(A)}{12}$$

Where: V_S and V_r are defined above
Q_d = the developed runoff for the design storm (inches)
A = total drainage area (acres)

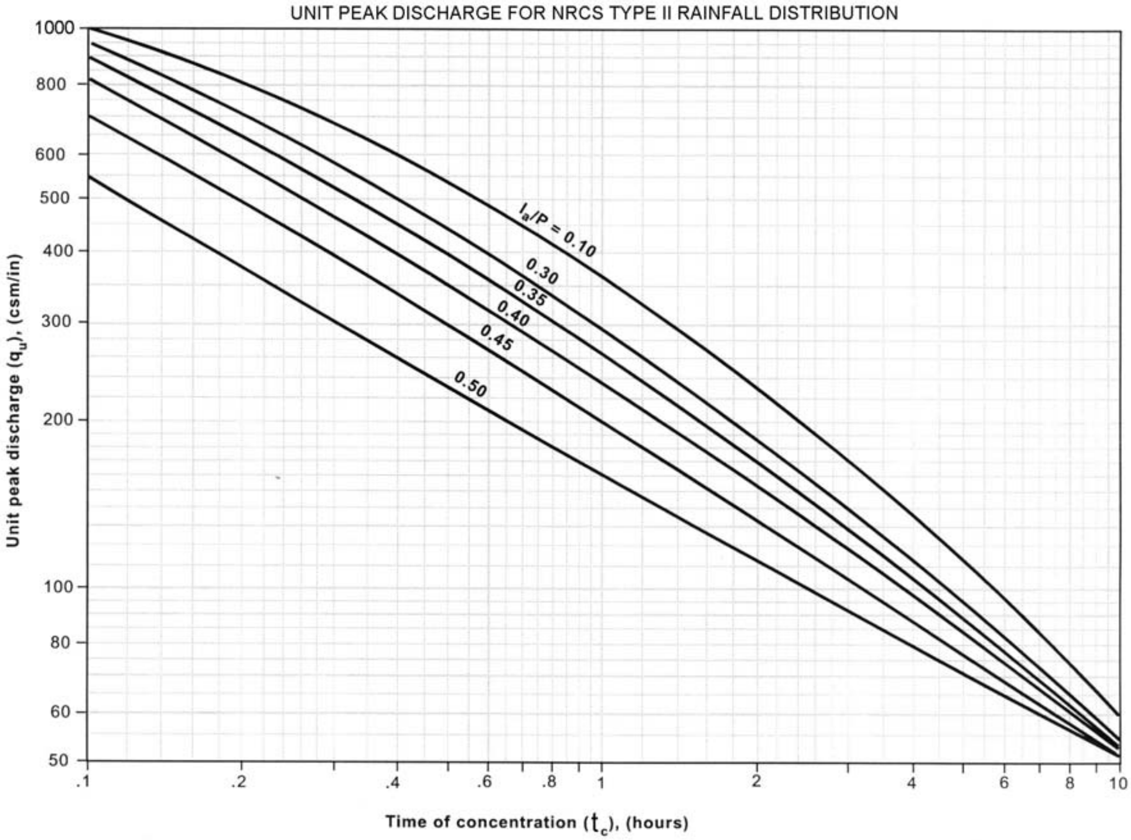


Figure D.11. Unit Peak Discharge for Type II Rainfall Distribution (Source: NRCS, 1986)

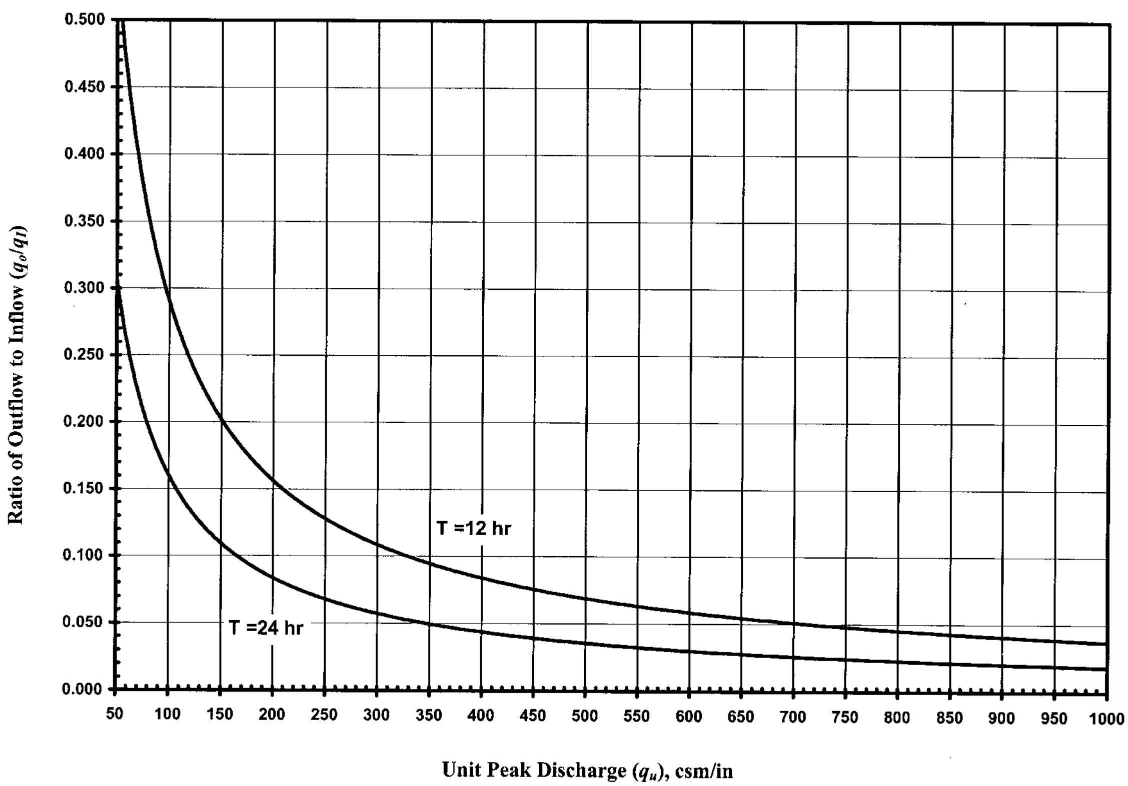


Figure D.12. Detention Time vs. Discharge Ratios (Source: Harrington, 1987)

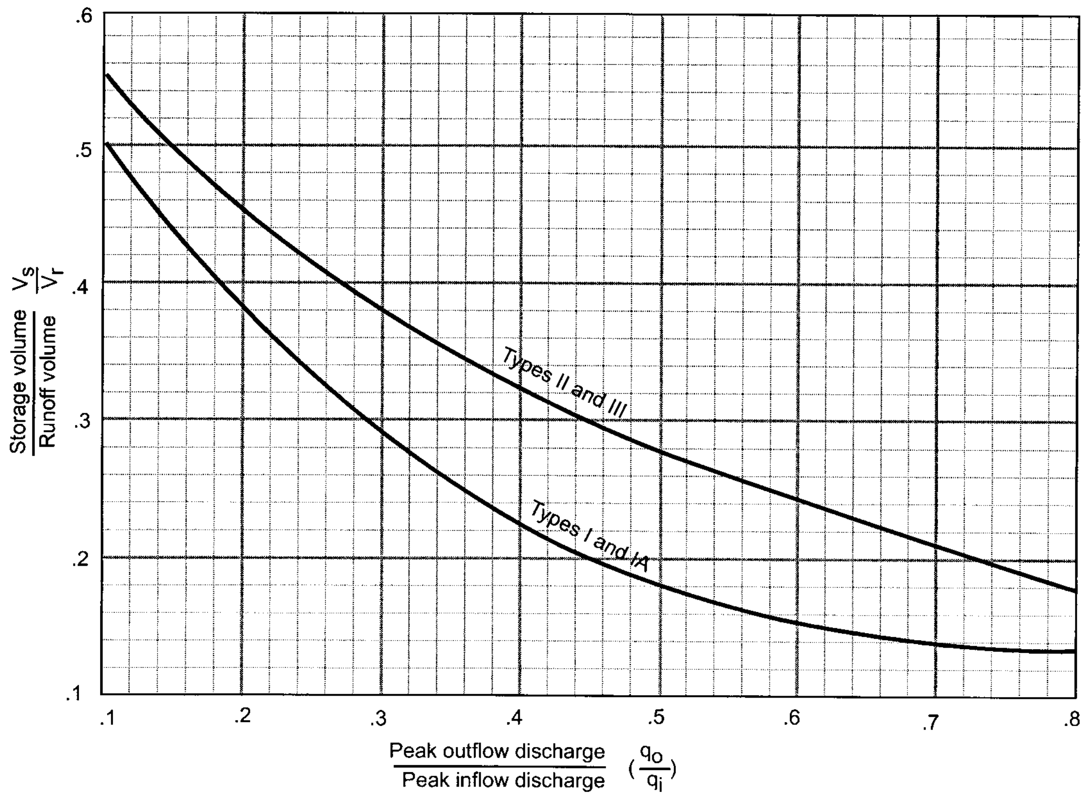


Figure D.13. Approximate Detention Basin Routing For Rainfall Types I, IA, II, and III. (Source: NRCS, 1986)

Water Quality Peak Flow Calculation

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

The following procedure can be used to estimate peak discharges for small storm events. It relies on the water quality volume and a modified approach to the NRCS peak flow estimating method. A brief description of the calculation procedure is presented below.

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

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

where P = rainfall, in inches (use the Water Quality Storm depth)
Q_a = runoff volume, in inches (equal to $WQ_V \div$ area)

Once a CN is computed, the time of concentration (t_c) is computed (based on the methods identified in TR-55 and Section 1 and 2 of the *Vermont Stormwater Management Manual - Volume I*).

Using the computed CN, t_c and drainage area (A), in acres; the peak discharge (Q_{wq}) for the water quality storm event is computed as follows.

Read initial abstraction (I_a), compute I_a/P

Read the unit peak discharge (q_u) for appropriate t_c

Using the water quality volume (WQ_v), compute the peak discharge (Q_{wq})

$$Q_{wq} = q_u * A * WQ_V$$

where Q_{wq} = the peak discharge, in cfs
q_u = the unit peak discharge, in cfs/mi²/inch
A = drainage area, in square miles
WQ_V = Water Quality Volume, in watershed inches

Appendix D7: Critical Erosive Velocities for Grass and Soil

Velocity

Maximum permissible velocities of flow in vegetated channels absent of permanent turf reinforcement matting must not exceed the values shown in the following table:

Table D.5. Permissible Velocities for Channels Lined with Vegetation

Channel Slope	Lining	Permissible Velocity[1] (ft/sec)
0-5%	Tall fescue Kentucky bluegrass	5
	Grass-legume mixture	4
	Red fescue Redtop Serices lespedeza Annual lespedeza Small grains	2.5
5-10%	Tall fescue Kentucky bluegrass	4
	Grass-legume mixture	3
Greater than 10%	Tall fescue Kentucky bluegrass	3

Source: Schwab, G. O., D.D. Fangmeier, W. J. Elliot, and R. K. Frevert, 1992. *Soil and Water Conservation Engineering*. John Wiley & Sons. 528 pp.

For vegetated earth channels having permanent turf reinforcement matting, the permissible flow velocity must not exceed 8 ft/sec. Turf reinforcement matting must be a machine produced mat of non-degradable fibers or elements having a uniform thickness and distribution of weave throughout. Matting must be installed per manufacturer's recommendations with appropriate fasteners as required. Examples of acceptable products include but are not limited to:

- North American Green "C350" or "P300"
- Greenstreak "PEC-MAT"
- Tensar "Erosion Mat"

[1] For highly erodible soils, permissible velocities should be decreased 25%. An erodibility factor (K) greater than 0.35 would indicate a highly erodible soil. Erodibility factors (K-factors) can be obtained from local NRCS offices.

Manning's n value

The roughness coefficient, n, varies with the type of vegetative cover and flow depth. At very shallow depths, where the vegetation height is equal to or greater than the flow depth, the n value should be approximately 0.15. This value is appropriate for flow depths up to 4 inches typically. For higher flow rates and flow depths, the n value decreases to a minimum of 0.03 for grass channels at a depth of approximately 12 inches. The n value must be adjusted for varying flow depths between 4" and 12" (see Figure D.14).

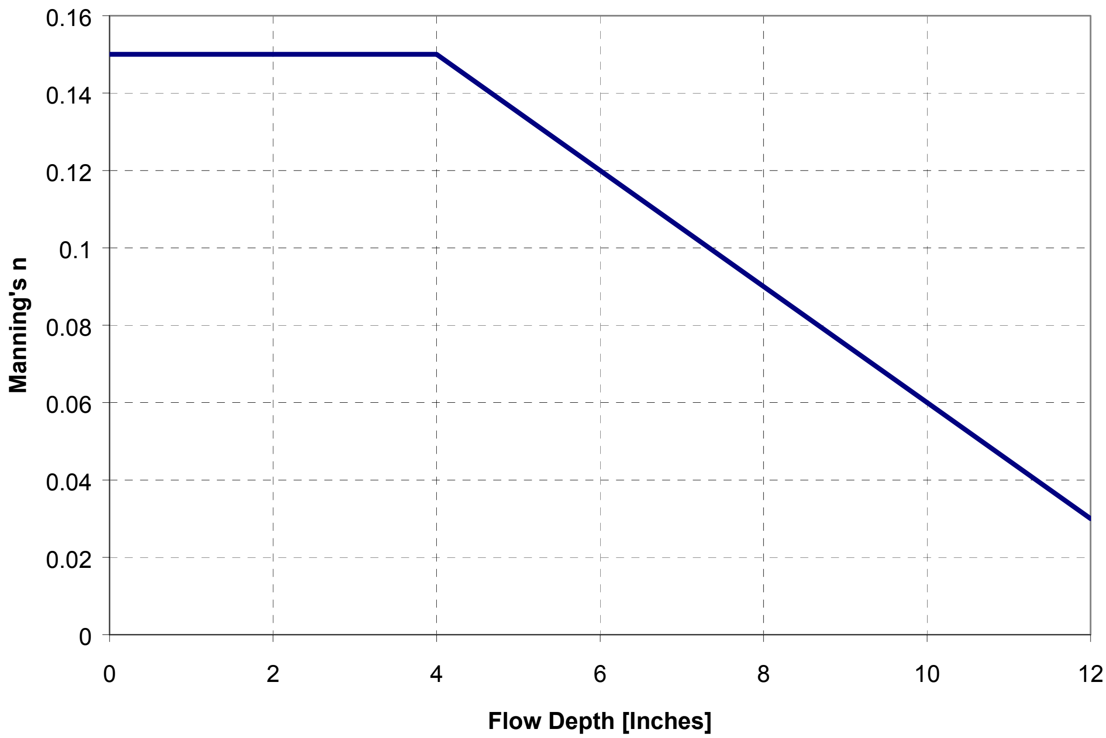


Figure D.14. Manning's n Value with Varying Flow Depth (Source: Claytor and Schueler, 1986)

Appendix D8: Maintenance and Inspection Checklists

Checklist for Preliminary/Concept Stormwater Management Plan Preparation and Review

- ☐ Applicant information
- ☐ Name, legal address, and telephone number
- ☐ Common address and legal description of site
- ☐ Vicinity map
- ☐ Existing and proposed mapping and plans (recommended scale of 1" = 50') which illustrate at a minimum:
    - Existing and proposed topography (minimum of 2-foot contours recommended)
    - Perennial and intermittent streams
    - Mapping of predominant soils from USDA soil surveys
    - Boundaries of existing predominant vegetation and proposed limits of clearing
    - Location and boundaries of resource protection areas such as wetlands, lakes, ponds, and other setbacks (e.g., stream buffers, drinking water well setbacks, septic setbacks)
    - Location of existing and proposed roads, buildings, and other structures
    - Existing and proposed utilities (e.g., water, sewer, gas, electric) and easements
    - Location of existing and proposed conveyance systems such as grass channels, swales, and storm drains
    - Flow paths
    - Location of floodplain/floodway limits and relationship of site to upstream and downstream properties and drainages
    - Preliminary location and dimensions of proposed channel modifications, such as bridge or culvert crossings
    - Preliminary location, size, and limits of disturbance of proposed stormwater treatment practices
- ☐ Hydrologic and hydraulic analysis including:
    - Existing condition analysis for runoff rates, volumes, and velocities presented showing methodologies used and supporting calculations
    - Proposed condition analysis for runoff rates, volumes, and velocities showing the methodologies used and supporting calculations
    - Preliminary analysis of potential downstream impact/effects of project, where necessary
    - Preliminary selection and rationale for structural stormwater management practices
    - Preliminary sizing calculations for stormwater treatment practices including contributing drainage area, storage, and outlet configuration
- ☐ Preliminary landscaping plans for stormwater treatment practices and any site reforestation or revegetation
- ☐ Preliminary erosion and sediment control plan that at a minimum meets the requirements outlined in local Erosion and Sediment Control guidelines
- ☐ Identification of preliminary waiver requests

Checklist for Final Stormwater Management Plan Preparation and Review

☐ Applicant information
☐ Name, legal address, and telephone number
☐ Common address and legal description of site
☐ Signature and stamp of registered engineer/surveyor and design/owner certification
☐ Vicinity map
☐ Existing and proposed mapping and plans (recommended scale of 1" = 50' or greater detail) which illustrate at a minimum:
- Existing and proposed topography (minimum of 2-foot contours recommended)
- Perennial and intermittent streams
- Mapping of predominant soils from USDA soil surveys as well as location of any site-specific borehole investigations that may have been performed.
- Boundaries of existing predominant vegetation and proposed limits of clearing
- Location and boundaries of resource protection areas such as wetlands, lakes, ponds, and other setbacks (e.g., stream buffers, drinking water well setbacks, septic setbacks)
- Location of existing and proposed roads, buildings, and other structures
- Location of existing and proposed utilities (e.g., water, sewer, gas, electric) and easements
- Location of existing and proposed conveyance systems such as grass channels, swales, and storm drains
- Flow paths
- Location of floodplain/floodway limits and relationship of site to upstream and downstream properties and drainages
- Location and dimensions of proposed channel modifications, such as bridge or culvert crossings
- Location, size, maintenance access, and limits of disturbance of proposed structural stormwater Management practices

☐ Representative cross-section and profile drawings and details of structural stormwater Management practices and conveyances (i.e., storm drains, open channels, swales, etc.) which include:
- Existing and proposed structural elevations (e.g., invert of pipes, manholes, etc.)
- Design water surface elevations
- Structural details of outlet structures, embankments, spillways, stilling basins, grade control structures, conveyance channels, etc.
- Logs of borehole investigations that may have been performed along with supporting geotechnical report.

- ☐ Hydrologic and hydraulic analysis for all structural components of stormwater system (e.g., storm drains, open channels, swales, Management practices, etc.) for applicable design storms including:
  - Existing condition analysis for time of concentrations, runoff rates, volumes, velocities, and water surface elevations showing methodologies used and supporting calculations
  - Proposed condition analysis for time of concentrations, runoff rates, volumes, velocities, water surface elevations, and routing showing the methodologies used and supporting calculations
  - Final sizing calculations for structural stormwater Management practices including, contributing drainage area, storage, and outlet configuration
  - Stage-discharge or outlet rating curves and inflow and outflow hydrographs for storage facilities (e.g., stormwater ponds and wetlands)
  - Final analysis of potential downstream impact/effects of project, where necessary
  - Dam breach analysis, where necessary
- ☐ Final landscaping plans for structural stormwater Management practices and any site reforestation or revegetation
- ☐ Structural calculations, where necessary
- ☐ Applicable construction specifications
- ☐ Erosion and sediment control plan that at a minimum meets the requirements of the local Erosion and Sediment Control Guidelines
- ☐ Sequence of construction
- ☐ Maintenance plan which will include:
  - Name, address, and phone number of responsible parties for maintenance.
  - Description of annual maintenance tasks
  - Description of applicable easements
  - Description of funding source
  - Minimum vegetative cover requirements
  - Access and safety issues
  - Testing and disposal of sediments that will likely be necessary
- ☐ Evidence of acquisition of all applicable local and non-local permits
- ☐ Evidence of acquisition of all necessary legal agreements (e.g., easements, covenants, land trusts)
- ☐ Waiver requests
- ☐ Review agency should have inspector's checklist identifying potential features to be inspected on site visits

Stormwater Pond / Wetland Construction Inspection Checklist

Project:
Location:
Site Status:
Date:
Time:
Inspector:

CONSTRUCTION SEQUENCE	SATISFACTORY / UNSATISFACTORY	COMMENTS
1. Pre-Construction / Materials and Equipment		
Pre-construction meeting		
Pipe and appurtenances on-site prior to construction and dimensions checked		
1. Material (including protective coating, if specified)		
2. Diameter		
3. Dimensions of metal riser or pre-cast concrete outlet structure		
4. Required dimensions between water control structures (orifices, weirs, etc.) are in accordance with approved plans		
5. Barrel stub for prefabricated pipe structures at proper angle for design barrel slope		
6. Number and dimensions of prefabricated anti-seep collars		
7. Watertight connectors and gaskets		
8. Outlet drain valve		
Project benchmark near pond site		
Equipment for temporary de-watering		

CONSTRUCTION SEQUENCE	SATISFACTORY / UNSATISFACTORY	COMMENTS
2. Subgrade Preparation		
Area beneath embankment stripped of all vegetation, topsoil, and organic matter		
3. Pipe Installation		
Method of installation detailed on plans		
A. Bed preparation		
Installation trench excavated with specified side slopes		
Stable, uniform, dry subgrade of relatively impervious material (If subgrade is wet, contractor must have defined steps before proceeding with installation)		
Invert at proper elevation and grade		
B. Pipe placement		
Metal / plastic pipe		
1. Watertight connectors and gaskets properly installed		
2. Anti-seep collars properly spaced and having watertight connections to pipe		
3. Backfill placed and tamped by hand under Ahaunches@ of pipe		
4. Remaining backfill placed in max. 8 inch lifts using small power tamping equipment until 2 feet cover over pipe is reached		

CONSTRUCTION SEQUENCE	SATISFACTORY / UNSATISFACTORY	COMMENTS
Concrete pipe		
1. Pipe set on blocks or concrete slab for pouring of low cradle		
2. Pipe installed with rubber gasket joints with no spalling in gasket interface area		
3. Excavation for lower half of anti-seep collar(s) with reinforcing steel set		
4. Entire area where anti-seep collar(s) will come in contact with pipe coated with mastic or other approved waterproof sealant		
5. Low cradle and bottom half of anti-seep collar installed as monolithic pour and of an approved mix		
6. Upper half of anti-seep collar(s) formed with reinforcing steel set		
7. Concrete for collar of an approved mix and vibrated into place (protected from freezing while curing, if necessary)		
8. Forms stripped and collar inspected for honeycomb prior to backfilling. Parge if necessary.		
C. Backfilling		
Fill placed in maximum 8 inch lifts		
Backfill taken minimum 2 feet above top of anti-seep collar elevation before traversing with heavy equipment		

CONSTRUCTION SEQUENCE	SATISFACTORY / UNSATISFACTORY	COMMENTS
4. Riser / Outlet Structure Installation		
Riser located within embankment		
A. Metal riser		
Riser base excavated or formed on stable subgrade to design dimensions		
Set on blocks to design elevations and plumbed		
Reinforcing bars placed at right angles and projecting into sides of riser		
Concrete poured so as to fill inside of riser to invert of barrel		
B. Pre-cast concrete structure		
Dry and stable subgrade		
Riser base set to design elevation		
If more than one section, no spalling in gasket interface area; gasket or approved caulking material placed securely		
Watertight and structurally sound collar or gasket joint where structure connects to pipe spillway		
C. Poured concrete structure		
Footing excavated or formed on stable subgrade, to design dimensions with reinforcing steel set		
Structure formed to design dimensions, with reinforcing steel set as per plan		
Concrete of an approved mix and vibrated into place (protected from freezing while curing, if necessary)		

CONSTRUCTION SEQUENCE	SATISFACTORY / UNSATISFACTORY	COMMENTS
Forms stripped & inspected for "honeycomb" prior to backfilling; parge if necessary		
5. Embankment Construction		
Fill material		
Compaction		
Embankment		
1. Fill placed in specified lifts and compacted with appropriate equipment		
2. Constructed to design cross-section, side slopes and top width		
3. Constructed to design elevation plus allowance for settlement		
6. Impounded Area Construction		
Excavated / graded to design contours and side slopes		
Inlet pipes have adequate outfall protection		
Forebay(s)		
Pond benches		
7. Earth Emergency Spillway Construction		
Spillway located in cut or structurally stabilized with riprap, gabions, concrete, etc.		
Excavated to proper cross-section, side slopes and bottom width		
Entrance channel, crest, and exit channel constructed to design grades and elevations		

CONSTRUCTION SEQUENCE	SATISFACTORY / UNSATISFACTORY	COMMENTS
8. Outlet Protection		
A. End section		
Securely in place and properly backfilled		
B. Endwall		
Footing excavated or formed on stable subgrade, to design dimensions and reinforcing steel set, if specified		
Endwall formed to design dimensions with reinforcing steel set as per plan		
Concrete of an approved mix and vibrated into place (protected from freezing, if necessary)		
Forms stripped and structure inspected for Ahoneycomb@ prior to backfilling; parge if necessary		
C. Riprap apron / channel		
Apron / channel excavated to design cross-section with proper transition to existing ground		
Filter fabric in place		
Stone sized as per plan and uniformly place at the thickness specified		
9. Vegetative Stabilization		
Approved seed mixture or sod		
Proper surface preparation and required soil amendments		
Excelsior mat or other stabilization, as per plan		

CONSTRUCTION SEQUENCE	SATISFACTORY / UNSATISFACTORY	COMMENTS
10. Miscellaneous		
Drain for ponds having a permanent pool		
Trash rack / anti-vortex device secured to outlet structure		
Trash protection for low flow pipes, orifices, etc.		
Fencing (when required)		
Access road		
Set aside for clean-out maintenance		
11. Stormwater Wetlands		
Adequate water balance		
Variety of depth zones present		
Approved pondscaping plan in place and budget for additional plantings		
Plants and materials ordered 6 months prior to construction		
Construction planned to allow for adequate planting and establishment of plant community (April-June planting window)		
Wetland buffer area preserved to maximum extent possible		

Comments:

Actions to be taken:

Infiltration Trench Construction Inspection Checklist

Project:
Location:
Site Status:
Date:
Time:
Inspector:

CONSTRUCTION SEQUENCE	SATISFACTORY / UNSATISFACTORY	COMMENTS
1. Pre-Construction		
Pre-construction meeting		
Runoff diverted		
Soil permeability tested		
Groundwater / bedrock depth		
2. Excavation		
Size and location		
Side slopes stable		
Excavation does not compact subsoils		
3. Filter Fabric Placement		
Fabric specifications		
Placed on bottom, sides, and top (Note: sand filter it acceptable alternative for bottom of trench)		

CONSTRUCTION SEQUENCE	SATISFACTORY / UNSATISFACTORY	COMMENTS
4. Aggregate Material		
Size as specified		
Clean / washed material		
Placed properly		
5. Observation Well		
Pipe size		
Removable cap / footplate		
Initial depth =_______feet		
6. Final Inspection		
Pretreatment facility in place		
Contributing watershed stabilized prior to flow diversion		
Outlet		

Comments:

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Actions to be Taken:

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Infiltration Basin Construction Inspection Checklist

Project:
Location:
Site Status:
Date:
Time:
Inspector:

CONSTRUCTION SEQUENCE	SATISFACTORY / UNSATISFACTORY	COMMENTS
1. Pre-Construction		
Runoff diverted		
Soil permeability tested		
Groundwater / bedrock depth		
2. Excavation		
Size and location		
Side slopes stable		
Excavation does not compact subsoils		
3. Embankment		
Barrel		
Anti-seep collar or Filter diaphragm		
Fill material		

CONSTRUCTION SEQUENCE	SATISFACTORY / UNSATISFACTORY	COMMENTS
4. Final Excavation		
Drainage area stabilized		
Sediment removed from facility		
Basin floor tilled		
Facility stabilized		
5. Final Inspection		
Pretreatment facility in place		
Inlets / outlets		
Contributing watershed stabilized before flow is routed to the factility		

Comments:

Actions to be Taken:

Sand/Organic Filter System Construction Inspection Checklist

Project:
Location:
Site Status:
Date:
Time:
Inspector:

CONSTRUCTION SEQUENCE	SATISFACTORY / UNSATISFACTORY	COMMENTS
1. Pre-construction		
Pre-construction meeting		
Runoff diverted		
Facility area cleared		
Facility location staked out		
2. Excavation		
Size and location		
Side slopes stable		
Foundation cleared of debris		
If designed as exfilter, excavation does not compact subsoils		
Foundation area compacted		
3. Structural Components		
Dimensions and materials		
Forms adequately sized		
Concrete meets standards		
Prefabricated joints sealed		
Underdrains (size, materials)		
4. Completed Facility Components		
24 hour water filled test		

CONSTRUCTION SEQUENCE	SATISFACTORY / UNSATISFACTORY	COMMENTS
Contributing area stabilized		
Filter material per specification		
Underdrains installed to grade		
Flow diversion structure properly installed		
Pretreatment devices properly installed		
Level overflow weirs, multiple orifices, distribution slots		
5. Final Inspection		
Dimensions		
Surface completely level		
Structural components		
Proper outlet		
Ensure that site is properly stabilized before flow is directed to the structure.		

Comments:

Actions to be Taken:

Bioretention Construction Inspection Checklist

Project:
Location:
Site Status:
Date:
Time:
Inspector:

CONSTRUCTION SEQUENCE	SATISFACTORY / UNSATISFACTORY	COMMENTS
1. Pre-Construction		
Pre-construction meeting		
Runoff diverted		
Facility area cleared		
If designed as exfilter, soil testing for permeability		
Facility location staked out		
2. Excavation		
Size and location		
Lateral slopes completely level		
If designed as exfilter, ensure that excavation does not compact susoils.		
Longitudinal slopes within design range		
3. Structural Components		
Stone diaphragm installed correctly		
Outlets installed correctly		
Underdrain		
Pretreatment devices installed		
Soil bed composition and texture		

4. Vegetation		
Complies with planting specs		
Topsoil adequate in composition and placement		
Adequate erosion control measures in place		
5. Final Inspection		
Dimensions		
Proper stone diaphragm		
Proper outlet		
Soil/ filter bed permeability testing		
Effective stand of vegetation and stabilization		
Construction generated sediments removed		
Contributing watershed stabilized before flow is diverted to the practice		

Comments:

Actions to be Taken:

Open Channel System Construction Inspection Checklist

Project:
Location:
Site Status:
Date:
Time:
Inspector:

CONSTRUCTION SEQUENCE	SATISFACTORY / UNSATISFACTORY	COMMENTS
1. Pre-Construction		
Pre-construction meeting		
Runoff diverted		
Facility location staked out		
2. Excavation		
Size and location		
Side slope stable		
Soil permeability		
Groundwater / bedrock		
Lateral slopes completely level		
Longitudinal slopes within design range		
Excavation does not compact subsoils		
3. Check dams		
Dimensions		
Spacing		
Materials		

Construction Sequence	Satisfactory / Unsatisfactory	Comments
4. Structural Components		
Underdrain installed correctly		
Inflow installed correctly		
Pretreatment devices installed		
5. Vegetation		
Complies with planting specifications		
Topsoil adequate in composition and placement		
Adequate erosion control measures in place		
6. Final inspection		
Dimensions		
Check dams		
Proper outlet		
Effective stand of vegetation and stabilization		
Contributing watershed stabilized before flow is routed to the factility		

Comments:

Actions to be Taken:

Stormwater Pond/Wetland Operation, Maintenance and Management Inspection Checklist

Project
Location:
Site Status:
Date:
Time:
Inspector:

Maintenance Item	Satisfactory/ Unsatisfactory	Comments
1. Embankment and emergency spillway (Annual, After Major Storms)		
Vegetation and ground cover adequate		
Embankment erosion		
Animal burrows		
Unauthorized planting		
Cracking, bulging, or sliding of dam		
a. Upstream face		
b. Downstream face		
c. At or beyond toe		
downstream		
upstream		
d. Emergency spillway		
Pond, toe & chimney drains clear and functioning		
Seeps/leaks on downstream face		
Slope protection or riprap failure		
Vertical/horizontal alignment of top of dam “As-Built”		
Emergency spillway clear of obstructions and debris		
Other (specify)		

Maintenance Item	**Satisfactory / Unsatisfactory**	**Comments**
2. Riser and principal spillway (Annual) Type: Reinforced concrete ________ Corrugated pipe ________ Masonry ________		
Low flow orifice obstructed		
Low flow trash rack a. Debris removal necessary		
b. Corrosion control		
Weir trash rack maintenance a. Debris removal necessary		
b. corrosion control		
Excessive sediment accumulation insider riser		
Concrete/masonry condition riser and barrels a. cracks or displacement		
b. Minor spalling (<1")		
c. Major spalling (rebars exposed)		
d. Joint failures		
e. Water tightness		
Metal pipe condition		
Control valve a. Operational/exercised		
b. Chained and locked		
Pond drain valve a. Operational/exercised		
b. Chained and locked		
Outfall channels functioning		
Other (specify)		

Maintenance Item	**Satisfactory / Unsatisfactory**	**Comments**
3. Permanent Pool (Wet Ponds) (Monthly)		
Undesirable vegetative growth		
Floating or floatable debris removal required		
Visible pollution		
Shoreline problem		
Other (specify)		
4. Sediment Forebays		
Sedimentation noted		
Sediment cleanout when depth < 50% design depth		
5. Dry Pond Areas		
Vegetation adequate		
Undesirable vegetative growth		
Undesirable woody vegetation		
Low flow channels clear of obstructions		
Standing water or wet spots		
Sediment and / or trash accumulation		
Other (specify)		
6. Condition of Outfalls (Annual , After Major Storms)		
Riprap failures		
Slope erosion		
Storm drain pipes		
Endwalls / Headwalls		
Other (specify)		

7. Other (Monthly)		
Encroachment on pond, wetland or easement area		
Complaints from residents		
Aesthetics a. Grass growing required		
b. Graffiti removal needed		
c. Other (specify)		
Conditions of maintenance access routes.		
Signs of hydrocarbon build-up		
Any public hazards (specify)		
8. Wetland Vegetation (Annual)		
Vegetation healthy and growing Wetland maintaining 50% surface area coverage of wetland plants after the second growing season. (If unsatisfactory, reinforcement plantings needed)		
Dominant wetland plants: Survival of desired wetland plant species Distribution according to landscaping plan?		
Evidence of invasive species		
Maintenance of adequate water depths for desired wetland plant species		
Harvesting of emergent plantings needed		
Have sediment accumulations reduced pool volume significantly or are plants “choked” with sediment		
Eutrophication level of the wetland.		
Other (specify)		

Comments:

Actions to be Taken:

Infiltration Trench Operation, Maintenance, and Management Inspection Checklist

Project:
Location:
Site Status:
Date:
Time:
Inspector:

MAINTENANCE ITEM	SATISFACTORY / UNSATISFACTORY	COMMENTS
1. Debris Cleanout (Monthly)		
Trench surface clear of debris		
Inflow pipes clear of debris		
Overflow spillway clear of debris		
Inlet area clear of debris		
2. Sediment Traps or Forebays (Annual)		
Obviously trapping sediment		
Greater than 50% of storage volume remaining		
3. Dewatering (Monthly)		
Trench dewaters between storms		
4. Sediment Cleanout of Trench (Annual)		
No evidence of sedimentation in trench		
Sediment accumulation doesn=t yet require cleanout		
5. Inlets (Annual)		
Good condition		
No evidence of erosion		
6. Outlet/Overflow Spillway (Annual)		

MAINTENANCE ITEM	SATISFACTORY / UNSATISFACTORY	COMMENTS
Good condition, no need for repair		
No evidence of erosion		
7. Aggregate Repairs (Annual)		
Surface of aggregate clean		
Top layer of stone does not need replacement		
Trench does not need rehabilitation		

Comments:

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Actions to be Taken:

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Sand / Organic Filter Operation, Maintenance and Management Inspection Checklist

Project:
Location:
Site Status:
Date:
Time:
Inspector:

MAINTENANCE ITEM	**SATISFACTORY / UNSATISFACTORY**	**COMMENTS**
1. Debris Cleanout (Monthly)		
Contributing areas clean of debris		
Filtration facility clean of debris		
Inlet and outlets clear of debris		
2. Oil and Grease (Monthly)		
No evidence of filter surface clogging		
Activities in drainage area minimize oil and grease entry		
3. Vegetation (Monthly)		
Contributing drainage area stabilized		
No evidence of erosion		
Area mowed and clipping removed		
4. Water Retention Where Required (Monthly)		
Water holding chambers at normal pool		
No evidence of leakage		

MAINTENANCE ITEM	SATISFACTORY / UNSATISFACTORY	COMMENTS
5. Sediment Deposition (Annual)		
Filter chamber free of sediments		
Sedimentation chamber not more than half full of sediments		
6. Structural Components (Annual)		
No evidence of structural deterioration		
Any grates are in good condition		
No evidence of spalling or cracking of structural parts		
7. Outlet/Overflow Spillway (Annual)		
Good condition, no need for repairs		
No evidence of erosion (if draining into a natural channel)		
8. Overall Function of Facility (Annual)		
Evidence of flow bypassing facility		
No noticeable odors outside of facility		

Comments:

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Actions to be Taken:

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Bioretention Operation, Maintenance and Management Inspection Checklist

Project:
Location:
Site Status:
Date:
Time:
Inspector:

MAINTENANCE ITEM	SATISFACTORY / UNSATISFACTORY	COMMENTS
1. Debris Cleanout (Monthly)		
Bioretention and contributing areas clean of debris		
No dumping of yard wastes into practice		
Litter (branches, etc.) have been removed		
2. Vegetation (Monthly)		
Plant height not less than design water depth		
Fertilized per specifications		
Plant composition according to approved plans		
No placement of inappropriate plants		
Grass height not greater than 6 inches		
No evidence of erosion		
3. Check Dams/Energy Dissipaters/Sumps (Annual, After Major Storms)		
No evidence of sediment buildup		
Sumps should not be more than 50% full of sediment		

MAINTENANCE ITEM	SATISFACTORY / UNSATISFACTORY	COMMENTS
No evidence of erosion at downstream toe of drop structure		
4. Dewatering (Monthly)		
Dewaters between storms		
No evidence of standing water		
5. Sediment Deposition (Annual)		
Swale clean of sediments		
Sediments should not be > 20% of swale design depth		
6. Outlet/Overflow Spillway (Annual, After Major Storms)		
Good condition, no need for repair		
No evidence of erosion		
No evidence of any blockages		
7. Integrity of Filter Bed (Annual)		
Filter bed has not been blocked or filled inappropriately		

Comments:

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Actions to be Taken:

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Open Channel Operation, Maintenance, and Management Inspection Checklist

Project:
Location:
Site Status:
Date:
Time:
Inspector:

MAINTENANCE ITEM	SATISFACTORY / UNSATISFACTORY	COMMENTS
1. Debris Cleanout (Monthly)		
Contributing areas clean of debris		
2. Check Dams or Energy Dissipaters (Annual, After Major Storms)		
No evidence of flow going around structures		
No evidence of erosion at downstream toe		
Soil permeability		
Groundwater / bedrock		
3. Vegetation (Monthly)		
Mowing done when needed		
Minimum mowing depth not exceeded		
No evidence of erosion		
Fertilized per specification		
4. Dewatering (Monthly)		
Dewaters between storms		
5. Sediment deposition (Annual)		
Clean of sediment		

MAINTENANCE ITEM	SATISFACTORY / UNSATISFACTORY	COMMENTS
6. Outlet / Overflow Spillway (Annual)		
Good condition, no need for repairs		
No evidence of erosion		

Comments:

Actions to be Taken:

Appendix D9: Distributed Runoff Control Methodology - Pond Outlet Structure Design Example

The following design example illustrates a step-by-step methodology for the design of a weir for the control of instream erosion potential using a Stormwater Management (SWM) wet pond design based on the Distributed Runoff Control (DRC) approach. The DRC approach incorporates boundary material composition and its sensitivity to erosion (entrainment and transport) into the design protocol. The boundary materials are characterized at the point of maximum boundary shear stress on the bed and the point of secondary maximum boundary shear stress on the bank. By examining the channel at selected sites downstream of the SWM facility the DRC protocol provides a pseudo 3-dimensional assessment of the impact of development and the SWM facility on the receiving channel. In the following design example the facility will serve the hypothetical Cole's Colony that was presented in Appendix C1.

This design example involves 5 Steps as listed in Table D.6

Table D.6. Overview of Key Steps in the DRC Design Approach

1) Determine the "stability" and "mode-of-adjustment" of the receiving channel.
2) Complete a Diagnostic Geomorphic Survey of the receiving channel.
3) Determine channel sensitivity to an alteration in the sediment-flow regime.
4) Approximate the elevation-discharge curve for the pond.
5) Size the DRC weir.

Step 1. Determine Channel "Stability" and "Mode-of-Adjustment"

Channel stability is determined using a Rapid Geomorphic Assessment (RGA) of the channel downstream of the outlet of the proposed Stormwater Management (SWM) pond. The RGA protocol involves the identification of the presence of in-stream features resulting from a variety of geomorphic processes to provide a semi-quantitative assessment of a stream's stability and mode-of-adjustment. The processes are represented by four factors: aggradation (AF), widening (WF), downcutting (DF), and planimetric form adjustment (PF). Each Factor is composed of 7 to 10 indices for which a present or absent response is required. The total number of "yes" responses is summed and divided by the total number of responses (both "yes" and "no") to derive a value for each factor. An index that is not relevant is not assigned a response. An example of an RGA Form is provided in Table D.7

A Stability Index (SI) value is determined from the factor values using the following equation:

$$SI = \frac{\{AF + DF + WF + PF\}}{m} \quad \text{...[EqnD.1]}$$

where ‘m’ is the number of factors (typically 4 for alluvial streams).

Table D.7. Rapid Geomorphic Assessment Form

FORM/ PROCESS	GEOMORPHIC INDICATOR No.	Description	PRESENT No	Yes	FACTOR VALUE
Evidence of Aggradation (AI)	1	Lobate bar	1		1/7=0.143
	2	Coarse material in riffles embedded		1	
	3	Siltation in pools	1		
	4	Medial bars	1		
	5	Accretion on point bars	1		
	6	Poor longitudinal sorting of bed materials	1		
	7	Deposition in the overbank zone	1		
Evidence of Degradation (DI)	1	Exposed bridge footing(s)	-	-	2/6=0.333
	2	Exposed sanitary/storm sewer/pipeline/etc.	-	-	
	3	Elevated stormsewer outfall(s)	-	-	
	4	Undermined gabion baskets/concrete aprons/etc.	-	-	
	5	Scour pools d/s of culverts/stormsewer outlets	1		
	6	Cut face on bar forms	1		
	7	Head cutting due to knick point migration	1		
	8	Terrace cut through older bar material		1	
	9	Suspended armor layer visible in bank		1	
	10	Channel worn into undisturbed overburden/bedrock	1		
Evidence of Widening (WI)	1	Fallen/leaning trees/fence posts/etc.		1	3/10=0.30
	2	Occurrence of Large Organic Debris		1	
	3	Exposed tree roots		1	
	4	Basal scour on inside meander bends	1		
	5	Basal scour on both sides of channel through riffle	1		
	6	Gabion baskets/concrete walls/armor stone/etc. out flanked	1		
	7	Length of basal scour >50% through subject reach	1		
	8	Exposed length of previously buried pipe/cable/etc.	1		
	9	Fracture lines along top of bank	1		
	10	Exposed building foundation	1		
Evidence of Planimetric Form Adjustment (PI)	1	Formation of cute(s)	1		0/7=0
	2	Evolution of single thread channel to multiple channel	1		
	3	Evolution of pool-riffle form to low bed relief form	1		
	4	Cutoff channel(s)	1		
	5	Formation of island(s)	1		
	6	Thalweg alignment out of phase with meander geometry	1		
	7	Bar forms poorly formed/reworked/removed	1		
STABILITY INDEX (SI) = (AI+DI+WI+PI)/m				SI=	0.19

The Stability Index (SI) provides an indication of the stability of the creek channel at a given time based on the guidelines provided in Table D.8. The SI Value, however, does not differentiate between current and past disturbances.

Table D.8. Interpretation of the RGA Stability Index Value

Stability Index Value	Stability Class	Description
0.0<SI<0.25	Stable	Metrics describing channel form are within the expected range of variance (typically accepted as one standard deviation from the mean) for stable channels of similar type
0.25<SI<0.4	Transitional	Metrics are within the expected range of variance as defined above but with evidence of stress
0.4<SI<1.0	In Adjustment	Metrics are outside of the expected range of variance for channels of similar type.

The guidelines presented in Table D.8 for the interpretation of the SI Value will vary with the field experience and the bias of the observer. The SI Values however, have been shown to be consistent between observers indicating that the protocol, once calibrated to the observer provides a reliable means of screening the channel for stability and mode-of-adjustment.

The RGA protocol is applied to channel segments of two meanders in length or the equivalent of 20 bankfull channel widths (the width of the channel at the geomorphically dominant discharge, recurrence interval of between 1 and 2 years or 1.5 years on average).

The segment chosen for application of the RGA assessment is selected to be representative of the morphology of the channel for some distance up and downstream of the surveyed segment. That is, the parameters defining channel cross-section and plan form (e.g. width, depth, meander wavelength, etc.) are within a consensual level of variance for this reach of channel. An acceptable level of variance is typically defined as within one standard deviation of the mean. These reaches are referred to as being of “like” morphology. Since the morphology of the channel will vary in the longitudinal direction with changes in flow, slope, physiography, etc., it will be necessary to re-apply the RGA protocol where the parameters characterizing the morphology of the channel have changed beyond the consensual level of variance from the previous survey reach. In this manner the channel is divided into a series of reaches of “like” morphology.

Having determined the length of the survey reach, the longitudinal profile can be plotted from topographic mapping as illustrated in Figure D.15 (Topo). Examination of Figure D.15 (topographic map data) suggests that the channel can be differentiated into three distinct reaches. In the first reach (length (L) = 146 ft, slope (S) = 0.00385, and the channel has a meander-pool-riffle morphology. In the middle reach (L≈356 ft;

S≈0.0142) the channel has cascade morphology. The third reach (L≈258 ft; S≈0.00794) returns to the meander-pool-riffle form.

Land use through the study reach is homogeneous (forest) and there are no other features (e.g. bridges, dams, weirs, instream works, etc.) that would affect the hydraulic characteristics of the active channel. Consequently, a preliminary definition of "like" reaches includes the three morphologies described above.

A synoptic geomorphic survey was conducted through the subject reach with an RGA assessment completed for each of the three reaches of "like" morphology. The results of the RGA assessment for the first reach (Reach 1) are reported in Tables D.7 and D.9. Referring to Table D.7, the Stability Index (SI) value was found to be SI=0.19, which is less than 0.25, therefore the channel is considered to be "stable" (Table D.8).

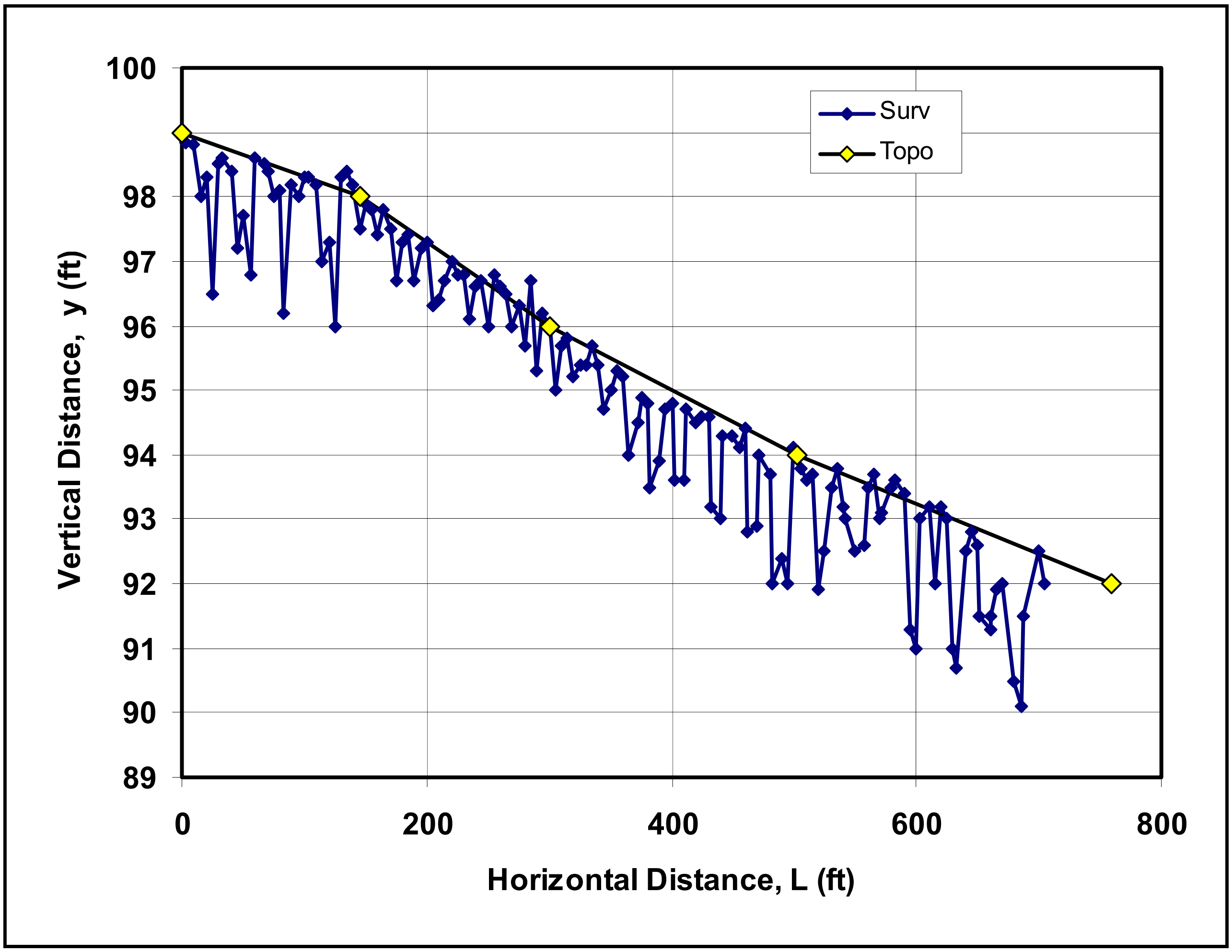

Figure D.15. Longitudinal Profile from Topographic Mapping and Field Survey of Channel Thalweg

Table D.9. Summary of Average Longitudinal Slope and Pool-Riffle Dimensions

Parameter	Reach 1	Reach 2	Reach 3
Longitudinal Gradient, S	0.00385	0.0142	0.00794
Riffle Length, LRIF (ft)	16	34	27
Pool Length, LPOL (ft)	37	10	18
Total Pool-Riffle Length, LTOT (ft)	53	44	45

Step 2. Diagnostic Geomorphic Survey

Following completion of the identification of reaches of "like" morphology and the synoptic survey to finalize the delineation of the "like" reaches, a diagnostic geomorphic survey is undertaken to characterize the morphological attributes of the channel. This information has two primary functions.

1. The optimization of the erosion control benefit of the pond
2. The provision for establishing a baseline condition from which it is possible to assess the performance of the SWM measures.

A detailed diagnostic survey includes a collection of a comprehensive set of parameters to assess and evaluate stream geomorphic conditions. A complete survey is typically required when:

1. A post-construction monitoring program is mandated; and,
2. Data are required for the design and construction of instream works.

Only a partial diagnostic survey is needed where the above issues are not relevant to the project. The following lists those parameters required for the partial diagnostic survey:

1. In the absence of flow measurements, a field estimate of Manning's 'n' value is obtained for comparison with sediment computed estimates.
2. Detailed survey of the channel cross-section, including the floodplain, to determine hydraulic geometry metrics at a so called "Master cross-section" and the relative location of bank material strata.
3. The longitudinal profile of the bed along the channel thalweg and the water surface at the time of survey over a distance of one meander wavelength or 10 bankfull widths. These data are used to determine the longitudinal gradient of the channel from riffle crest to riffle crest and to determine the dimensions of the pool-riffle complex.
4. At least one estimate of bankfull depth (the depth of flow at the dominate discharge) at the Master cross-section and all ancillary cross-sections (3

alternative methods are described in this example for illustrative purposes).

5. Bed material characteristics based on pebble counts of the bed material at a riffle crossover. These data are collected to help assess roughness coefficients, bed material resistance, and provide an alternate method for the estimation of bankfull depth.
6. Soil pits in the banks to map bank stratigraphy and to determine bank material composition using soil consistency tests (stickiness, plasticity and firmness) or particle size analysis (percent silt clay) with Atterberg Limits (Plasticity Index) for each stratigraphic unit. These data are required to help assess historic degradation or aggradation patterns and determine bank material resistance.
7. Map riparian vegetation and root zone characteristics in the soil pits for assessment of the affect of root binding on bank material resistance.

The cross-section data and bank material characterization is completed at a Master cross-section within the representative segment of each "like" reach. The Master cross-section is typically located at a riffle crossover on a straight reach between meander bends. Ancillary cross-sections are located in the lower one third of the meander bends and riffle crossover points up and downstream of the Master cross-section. Data collected at the ancillary cross-sections includes a cross-section profile (typically 7 to 9 ordinates) and estimates of bankfull stage. The longitudinal profile is collected throughout the survey segment along with characterization of plan form geometry.

Design Case: Diagnostic Geomorphic Survey for the Cole's Colony Channel

The longitudinal survey of the channel along the thalweg is presented in Figure D.15 ("Survey" data points). This profile more clearly demonstrates the differences between the three reaches as represented by slope and pool-riffle dimensions (Table D.9). Other parameter values derived from the geomorphic survey are summarized in Table D.10. These data are combined with the cross-section, soils and sediment data to generate values for key parameters as described in the following series of calculations.

The following calculations are required to determine the 3 different estimates of the dominant discharge.

Estimate of Geomorphic Referenced Dominant Discharge

1. The longitudinal data are plotted to generate estimates of the channel gradient in order of priority as follows:
    a. Water surface profile based on estimates of bankfull stage from the Master and ancillary cross-sections
    b. Bed slope (riffle crest to riffle crest)
    c. Water surface profile (dry weather flow at the time of the survey)

Table D.10. Summary of Hydraulic & Sediment Parameters for Cole's Colony Channel

Reach No.	Rosgen Stream Type	Parameter: 2 Year Flow Q_{2YR} (cfs)	W/d Ratio	Width W_{BFL} (ft)	Depth d_{BFL} (ft)	Flow Q_{BFL} (cfs)	Base B (ft)	Wetted Perimeter P (ft)
1	C3	8.9	3.00	3.00	1.00	4.76	2.00	4.24
2	B3	9.54	3.23	2.75	0.85	5.10	1.90	3.80
3	C3	10.1	2.87	2.83	0.99	5.40	1.85	4.06

Reach No.	Parameter: Bed Material Mean Particle Size ϕ_{50} (in)	Bed Material Mean Particle Size ϕ_{84} (in)	Area A_{BFL} (ft^2)	Hydraulic Radius R (ft)	Slope S (ft/ft)	Velocity v (fps)	Riparian Vegetation Type
1	2.8	3.3	2.50	0.590	.00385	1.90	Woody
2	5.1	7.5	1.99	0.521	.0142	2.57	Woody
3	3.7	5.2	2.32	0.570	.00794	2.35	Woody

Reach No.	Parameter: Bank Material Composition: Soil Class: Class	Soil Class: Unit No.	Soil Consistence Test: X1	X2	X3	SCORE	Critical Shear Stress: Bank (*) τ_{CRT} (lbs/ft^2)	Critical Shear Stress: Bed τ_{CRT} (lbs/ft^2)	Depth of Stratigraphic Unit h (ft)	Excess Boundary Shear Stress τ_{CRT} (lbs/ft^2): Bank	Excess Boundary Shear Stress: Bed
1	SiLm	1	1	2	1	4		0.548	$0.36<h\leq1.00$	0.057	-0.334
	SiSa	2	0	0	1	1	0.120		$0.10<h\leq0.36$		
	CoGr	3	N/a	N/a	N/a	N/a			$0.0<h\leq0.10$		
2	CoBo	1	N/a	N/a	N/a	N/a	0.573	1.206	$0.39<h\leq0.85$	-0.016	-0.526
	GrCo	2	N/a	N/a	N/a	N/a			$0.0<h\leq0.39$		
3	SiLm	1	2	1	3	6		0.878	$0.32<h\leq0.99$	0.03	-0.446
	SiCl	2	2	2	2	6	0.329		$0.12<h\leq0.32$		
	SiCl	3	2	3	2	7			$0.0<h\leq0.12$		

(*) Least resistant lower bank stratigraphic unit corresponding to the zone of secondary maximum boundary shear stress.

2. The pebble count data (length, width and breadth) are transformed into an equivalent diameter and used to generate a mass curve wherein cumulative percent finer by mass is plotted as a function of particle diameter.

3. The ϕ_{50} and ϕ_{84} particle size values (the particle diameter below which 50% and 84% of the particles are finer by mass, respectively) are determined from the mass curve.

4. Manning's roughness coefficient is estimated at bankfull stage using: Standard field guides, and Empirical relations such as the Strickler (1923) and Limerinos (1970) equations.

5. The cross-section ordinates collected at the Master cross-section are plotted to produce a cross-section profile and a stage-area curve.

6. The stage-area curve is combined with the longitudinal gradient (S) and the estimate of Manning's roughness coefficient (n) to generate the stage-discharge curve for the cross-section using Manning's equation:

$$Q = \frac{1.49}{n} AR^{(\frac{2}{3})} S^{\frac{1}{2}} \text{.......................................[Eqn D.2]}$$

   in which Q represents the flow rate (cfs) at depth 'y' above the thalweg, 'A' is the cross-section area of the channel at depth 'y', 'R' represents the hydraulic radius at depth 'y' and 'S' is the longitudinal gradient of the channel. An example of a stage-discharge curve is provided in Figure D.16.

7. The dominant discharge (Q_{GEO}) is determined from the stage-discharge curve and field estimate of bankfull stage (d_{BFL}). For Reach 1 in the Cole's Colony example, d_{BFL}=1.0 ft, consequently Q_{GEO}=4.76 cfs (Figure D.16). This procedure is repeated for each cross-section within the reach and the flow rate most common to all cross-sections is adopted as the geomorphic referenced estimate of the dominant discharge. If a wide disparity exists between estimates of (Q_{GEO}) than the determination of slope, Manning's 'n' value and the geomorphic indicators of bankfull stage are revisited to determine if a miss-interpretation of the data or an error in calculations has occurred.

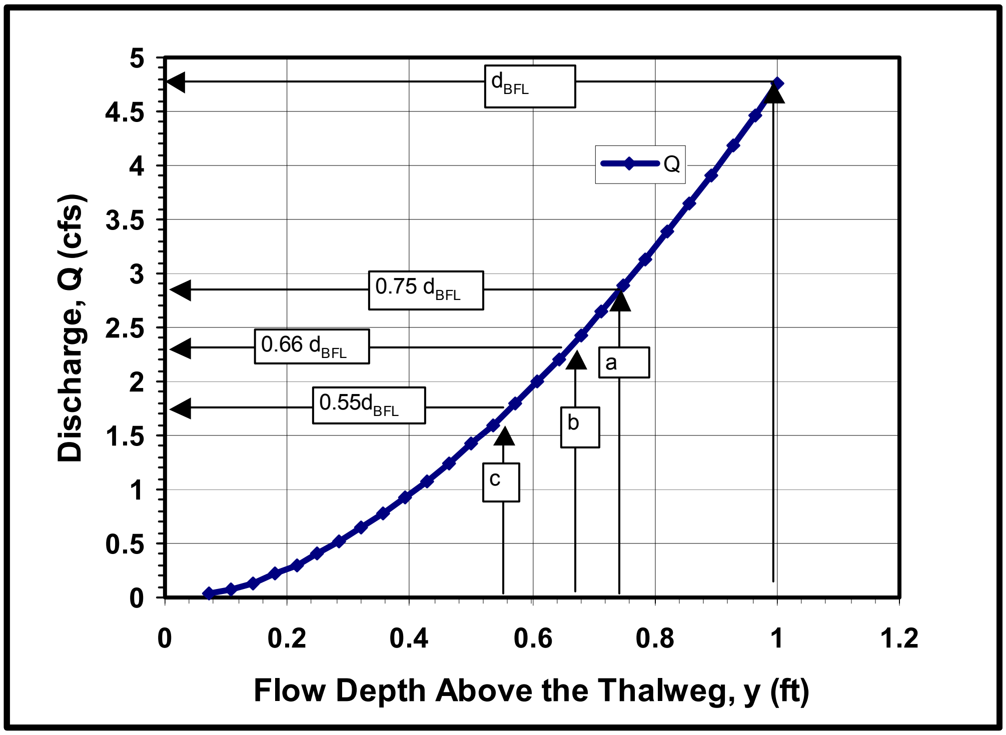

Figure D.16. Stage-Discharge Curve for Reach 1 Downstream of the Proposed Cole's Colony Development

Estimate of Bed Material Critical Shear Stress

8. Critical shear stress is estimated for the ϕ_{84} particle size value of the bed material using procedures such as:

   a. The modified Shield's equation (Vanoni, 1977), or

   b. Various empirical relations (from the literature) that express critical shear stress as a function of particle size, one such is Eqn D.3 proposed by Lane (1955)

$$(\tau_{CRT})_{BED} = 0.164\phi_{84} \quad \text{[Eqn D.3]}$$

in which ϕ_{84} is the particle size for which 84% of the materials are finer (inches) and τ_{CRT} represents the critical shear stress (lbs/ft^2). Applying Eqn D.3 to the Cole's Colony example,

$(\tau_{CRT})_{BED}= 0.164\phi_{84} = 0.164\ (3.34\text{ in}) = 0.548\text{ lbs/ft}^2$

at the Master cross-section (Reach 1).

Estimate of Instantaneous Bed Shear Stress

9. A stage-shear stress curve is generated for the Master cross-section using DuBoy's relation for average shear stress and a channel shape adjustment factor proposed by Lane (1955) as follows:

$$\tau_0 = k_b \rho g (d - d_P) S \text{ ....[Eqn D.4]}$$

and,

$$k_b = 0.000547\left(\frac{B}{d}\right)^3 - 0.0121\left(\frac{B}{d}\right)^2 + 0.092\left(\frac{B}{d}\right) + 0.75 \text{ ....[Eqn D.5]}$$

in which τ_0 represents the instantaneous boundary shear stress at point 'P' on the bed (lbs/ft s^2), k_b is a channel shape adjustment factor (dimensionless; Figure D.17), ρ is the density of the sediment-water mixture being conveyed by the channel (62.4 lbs/ft^3), 'g' is acceleration due to gravity (32.2 ft/s^2), 'd' is the depth of the flow above the thalweg (ft), d_P is the depth of flow above the thalweg at point 'P' (ft), 'S' represents the longitudinal gradient of the flow at depth 'd' and 'B' is the bottom width of the channel (assuming a trapezoidal configuration). In the Cole's Colony design case, a mapping of the isovels through the Master cross-section indicates that the point of maximum boundary shear stress occurs at the thalweg. Since the thalweg is the deepest part of the channel, the term d_P=0 in Eqn. D.4. A stage-shear stress curve for Reach 1 of the Cole's Colony study is illustrated in Figure D.18. Note that the units for τ_0 are reported in lbs/ft^2 to be consistent with the estimate of critical shear stress reported in Task 8. To obtain units of lbs/ft^2 remove 'g' from Eqn. D.4.

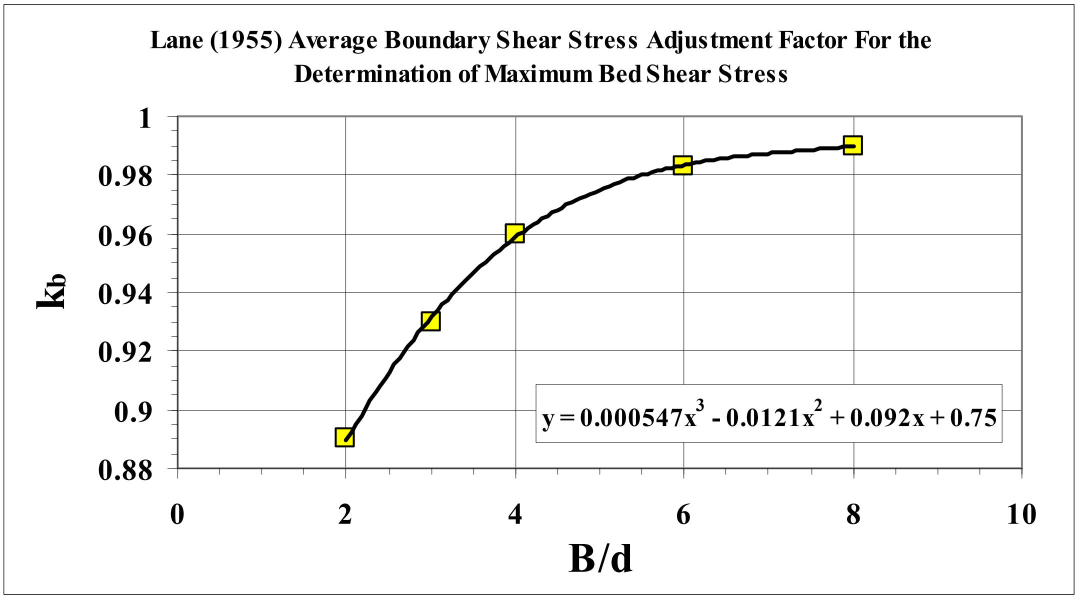

Figure D.17. Determination of k_B for the Adjustment of Average Boundary Shear Stress For Variations in Channel Shape Assuming A Trapezoidal Channel Cross-Section Configuration

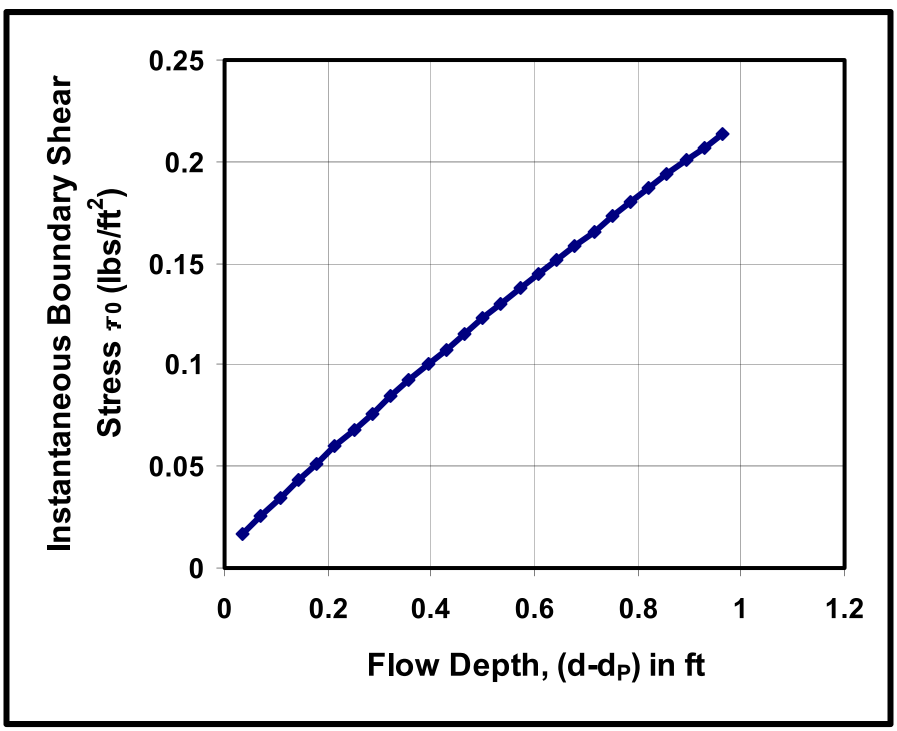

Figure D.18. Stage-Shear Stress Curve For Cole's Colony, Reach 1 (Master Cross-section): Bed Station

Estimate the Sediment Referenced Dominant Discharge

10. The stage-shear stress curve is used to determine the depth of flow at which the boundary shear stress on the bed is equal to the critical shear stress of the ϕ_{84} particle size fraction. This depth is transformed into an estimate of flow rate from the stage-discharge curve (Task 5 above), providing a second, independent estimate of the dominant discharge (Q_{SED}). This calculation also provides a basis for determination of the sensitivity of the bed material to an alteration in the sediment-flow regime. This assessment is described in Task 21 below;

Estimate The Flow Recurrence Interval of the Referenced Dominant Discharge

11. A flow time series is generated using:
    a. Flow gauge data if available, or
    b. A continuous hydrologic model to generate a synthetic flow time series of 6 to 13 years in length.
12. The flow time series is used to derive a flood frequency curve from which a third independent estimate of the dominant discharge (Q_{RI}) is determined as the flow having a recurrence interval between 1 and 2 years (average RI=1.5 years).

Finalize the Estimate of Dominant Discharge

13. The three estimates of dominant discharge are compared for consistency. If consistent (e.g. the range is equal to or less than 20% of the mean), then the mean value of the dominant discharge can be accepted with a higher degree of confidence.

Step 3. Determine the Sensitivity of the Boundary Materials

Sensitivity of the Bed Material

14. Using the stage-shear stress relationship developed in Task 9 and the estimate of flow depth (d_{BFL}, Task 10) from the dominant discharge (Task 13), determine the boundary shear stress $(\tau_0)_{BED}$ being applied to the bed at point 'P' at the dominant discharge. Point 'P' is located on the bed within the zone of maximum boundary shear stress. In the Cole's Colony example the value of maximum instantaneous boundary shear stress at a depth of d_{BFL}= 1.0 ft was found to be $(\tau_0)_{BED}$ = 0.214 lbs/ft^2 at the Master cross-section in Reach 1 (Figure D.18). Similarly, for Reaches 2 and 3 the maximum value of instantaneous boundary shear stress was found to be $(\tau_0)_{BED}$ = 0.680 and 0.432 lbs/ft^2 respectively.

15. Compute the value of $(\tau_e)_{BED}$ for the Master cross-section knowing $(\tau_0)_{BED}$ and $(\tau_{CRT})_{BED}$ as,

$$(\tau_e)_{BED} = (\tau_0 - \tau_{CRT})_{BED} \text{.......................................[Eqn D.6]}$$

in which $(\tau_e)_{BED}$ represents the effective boundary shears stress, τ_0 is the instantaneous boundary shear stress at the dominant discharge and τ_{CRT} is the critical shear stress of the bed material at point 'P'.

16. Repeat the bed shear stress analysis for all Master cross-sections in all reaches of "like" morphology.

17. Compare the value of $(\tau_e)_{BED}$ for all Master cross-sections through the study reach and select the Master cross-section for which the value of $(\tau_e)_{BED}$ is greatest. The reach represented by the Master cross-section having the highest value of $(\tau_e)_{BED}$ is referred to as the "Control Reach".

In the Cole's Colony example, effective boundary shear stress on the bed was found to range from between -0.526 and -0.334 (Table D.10). The negative values infer that the channel bed is armored and the bed material is mobile under flood flow events in excess of the dominant discharge. However, of the three Master cross-sections the value of $(\tau_e)_{BED}$ was greatest for Reach 1, consequently, Reach 1 was identified as the "Control Reach".

Sensitivity of the Bank Material

18. The bank material for the "Control Reach" is classified according to soil type for each stratigraphic unit using:
    (1) Soil consistency tests
    (2) Particle size analysis and Atterberg Limits
    In the Cole's Colony example the bank materials were mapped and differentiated into stratigraphic units as summarized for the three reaches in Table D.10. The soil consistency test results determined using standard soil classification guidelines (as quantified by MacRae, 1991)), are summarized below and reported in Table D.10.
    i) Assign a value for the stickiness of the material, e.g. not sticky, (X1=0) to extremely sticky (X1=4)
    ii) Assign a value for the plasticity of the material, e.g. not plastic (X2=0) to extremely plastic (X2=4)
    iii) Assign a value for the firmness of the material, e.g. loose, no structure (X3=0) to stiff (X4=4)
    iv) Sum the consistency test values

$$SCORE = \sum_{i=1}^{3} x_i \text{ ....[Eqn D.7]}$$

in which SCORE represents the sum of the values assigned for stickiness, plasticity and firmness.

19. Construct stage-shear stress curves for selected bank stations approximated by $0.25d_{BFL}$, $0.33d_{BFL}$, $0.4d_{BFL}$. More than one bank station may be required in a stratigraphic unit depending upon the thickness of the unit. The curves may be approximated as follows:

$$\tau_0 = k_S(\rho g(d - d_P)S) \text{ ....[Eqn D.8]}$$

in which k_S is a correction factor for points on the channel bank determined as a function of channel shape (see Eqn. D.9, Figure D.19), 'd' is the depth of flow (ft), ρ is the density of water (62.4 lbs/ft^3), 'g' is acceleration due to gravity (32.2 ft/s^2) and d_P is the depth of flow at the elevation of the boundary station (ft).

$$k_S = 0.7236\left(\frac{B}{d}\right)^{0.0241} \text{ ....[Eqn D.9]}$$

in which B is the channel bottom (ft) width and 'd' is the depth of flow (ft). Note, to obtain units of lbs/ft^2 remove the constant 'g' from Eqn. D.8.

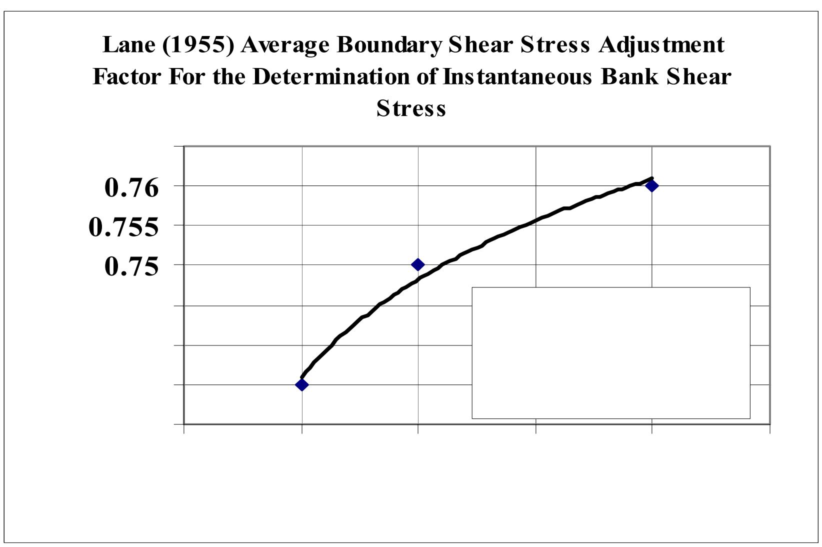

Figure D.19. Adjustment Factor k_S for Bank Shear Stress For Channels Approximating a Trapezoidal Shape

20. Estimate the critical shear stress (τ_{CRT}) within each stratigraphic unit using available empirical relationships. These relations are typically based on percent silt and clay content, degree of compaction, particle size (Vanoni, 1977) or the SCORE value (MacRae, 1991).

21. Compute the excess boundary shear stress for each bank station at a flow depth of between 0.6 and 0.75 feet by reading the boundary shear stress off the stage-shear stress curve for each boundary station and subtracting the critical shear stress as described in DuBoy's relation:

$$(\tau_e)_{BNK} = (\tau_0 - \tau_{CRT})_{BNK} \quad \text{[Eqn D.10]}$$

in which $(\tau_e)_{BNK}$ represents the excess boundary shear stress (lbs/ft^2) at the selected boundary station (P), τ_0 is the instantaneous boundary shear stress (lbs/ft^2) at any specified depth of flow at point P and τ_{CRT} represent the critical shear stress (lbs/ft^2) of the boundary material at point P.

22. Compare the estimates of excess boundary shear stress $(\tau_e)_{BNK}$ at each bank station and select that station having the highest value of $(\tau_e)_{BNK}$ as the bank station controlling bank response (controlling stratigraphic unit) to a change in the flow regime. Using the guidelines presented in Table D.11 determine channel sensitivity to an alteration in the sediment-flow regime and the corresponding Over Control (OC) curve and Inflection Point

Bank materials may be grouped according to the SCORE value if the soil consistency tests apply (i.e. fine-grained material with few stones). For coarse-grained materials, resistance can be determined from observation of bank erosion following a high flow event. As an alternative the resistance of the coarse-grained stratigraphic unit can be inferred from bank form and shear stress distribution through comparison with adjoining strata of fine-grained material.

Finally, relations expressing critical shear stress as a function of particle size are available in the literature. Many of these relations were derived from flume experiments using disturbed material that has been re-compacted. These relations tend to underestimate the resistance of the material as it is observed in the field. Consequently, these relations should be employed with caution or corrected to account for root binding, imbrication, compaction and structurization.

Table D.11. Guidelines for the Application of the DRC Approach Based on Bank Material Sensitivity Using SCORE Values

BANK SENSITIVITY		BED SENSITIVITY				DRC PARAMETERS	
Excess Shear Stress $(\tau_e)_{BED}$	Sensitivity Class	Excess Shear Stress $(\tau_e)_{BNK}$	Bank Resistance: Soil Class	Bank Resistance: SCORE	Sensitivity Class	Over Control Multiplier R_{OC}	Inflection Point
<0		<0	Very Stiff	N/a	L	1.0 –0.9	a
		≈0	Stiff	10-12	ML	0.9 - 0.7	a
			Firm	7-9	M	0.7 - 0.5	b
			Soft	≤6	H	0.5 - 0.2	c
		>0	**N/a**			**0.5 - 0.2**	**c**
≈0	ML	<0	N/a			0.9 - 0.7	a
		≈0	Stiff	10-12	ML	0.9 - 0.7	a
			Firm	7-9	M	0.7 - 0.5	b
			Soft	≤6	H	0.5 - 0.2	c
		>0	N/a			0.5 - 0.2	c
	M	<0	N/a			0.7 - 0.5	b
		≈0	Stiff	N/a		0.7 - 0.5	b
			Firm	7-9	M	0.7 - 0.5	b
			Soft	≤6	H	0.5 - 0.2	c
		>0	N/a			0.5 - 0.2	c
	H	N/a				0.5 - 0.2	c
>0	H	N/a				0.5 - 0.2	c

The multiplier (R_{OC}) in Table D.11 is used in the following manner:

a) The 2 year peak flow attenuation technique is used to derive the stage-discharge curve for the erosion control component of the SWM pond.

b) A multiplier of unity is equivalent to the traditional 2-year peak flow attenuation approach.

c) The multiplier is used to adjust the 2-year stage-discharge curve to account for differences in the erodability of the boundary materials. The adjustment is performed by multiplying each ordinate of the stage-discharge curve by R_{OC}. For stiff materials, the multiplier approaches unity ($R_{OC} \rightarrow 1.0$). For very sensitive materials, the multiplier is between 0.2 and 0.3, which is equivalent to 80%OC to 70%OC respectively.

Step 4. Approximate the Elevation-Discharge Curve For the DRC Pond.

The DRC outflow control structure can be constructed as set of pipes or nested weirs. This design example is for a nested, sharp crested weir.

Determine the stage-discharge curve for the flow rate having a recurrence interval of 2 years for the baseline land use condition. For the Cole's Colony development, the baseline condition is the reforested land use scenario. The flow having a recurrence interval 2 years was determined previously as between 8.9 and 10.1 cfs for Reaches 1 through 3 respectively (Table D.10).

Construct the 2-year stage-discharge curve using an equation for sharp crested weirs with end contractions:

$$Q = C_e L_e h_e^{\left(\frac{3}{2}\right)} \quad \text{[Eqn D.11]}$$

in which, Q represents the rate of flow (cfs), C_e is the effective weir coefficient (C=3.19, Brater and King, 1982), L_e is the effective length of the weir (ft) and h_e is the effective depth of flow above the weir crest (ft). Set the invert of the weir at 628.0 ft. The terms L_e, C_e and h_e are adjusted to account for losses due to end contractions (Brater and King, 1982). In this illustration it is assumed that the stage-volume curve has already been derived and that the approximate head at QBFL=8.9 cfs is h=2.25 ft.

Re-arranging Eqn. D.11 and solving for L_e at Q=(Q2YR)PRE=8.9 cfs yields,

$$L_e = \frac{Q}{C_e h_e^{\left(\frac{3}{2}\right)}} = \frac{8.9}{3.19(2.25)^{\left(\frac{3}{2}\right)}} = 0.83\text{ft} \quad \text{[Eqn D.12]}$$

Compute the stage-discharge curve for the 2-year weir using Eqn. D.11 as illustrated in Figure D.20 (Q_{2YR}, curve AB) for the Cole's Colony development. This stage-discharge curve represents the rating curve for the 2-year post-development to pre-development peak flow attenuation approach.

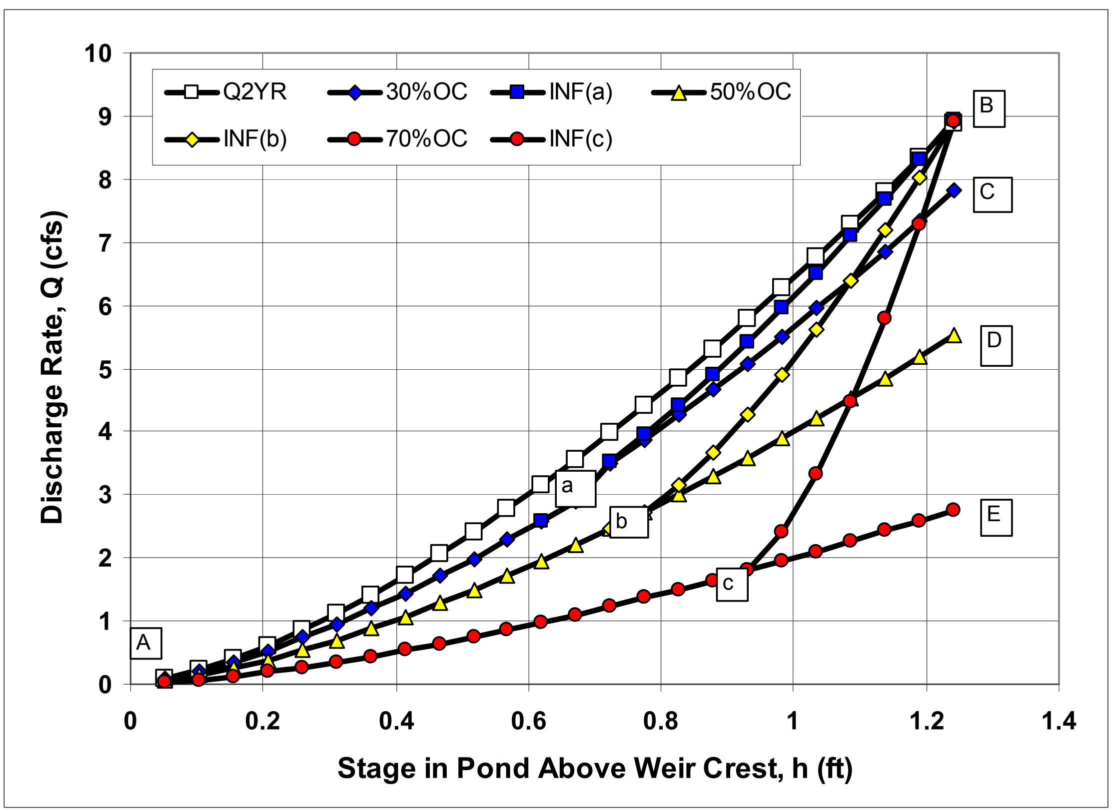

Figure D.20. The 2 Year Peak Flow Attenuation and DRC Rating Curves For 30%OC, 50%OC and 70%OC for Cole's Colony Design Case

Construct the DRC stage-discharge curve as follows:

- Determine the level of OC control and the inflection point from Table D.11
  - Since $(\tau_e)_{BED}<0$ (Table D.10) then the bed is classified as "Low" sensitivity (shaded boxes in the first two columns of Table D.11);
  - The value of $(\tau_e)_{BNK}>0$ consequently, Row 3 of Column 3 (shaded box in Table D.11) was selected;
  - The bank material was classified as soft (SCORE=1), consequently, the 4th Row of Column 4 was chosen providing a range of R_{OC} between 0.5 and 0.2 with an inflection point at "c". In this case R_{OC}=0.3 was selected in accordance with the guidelines in Table D.11. Note: 70%OC means that the multiplier for the 2 year curve is R_{OC}=0.3
  - The 70%OC curve (designated as curve AE in Figure D.20) is created by multiplying the ordinance of the 2 year stage-discharge curve (Q_{2YR} in Figure D.20) by the multiplier R_{OC}=0.3.
  - The inflection point (c) is determined using the guidelines provided in Table D.12.

Table D.12. Guidelines For Determination of the Flow Rate For the DRC Curve Inflection Point: Cole's Colony Design Case (Reach 1)

Inflection Point	Ratio of Inflection Point Depth to Bankfull Depth d_i/d_{BFL} (dim)	Bankfull Depth d_{BFL} (ft)	Inflection Point Depth d_i (ft)	Dominant Discharge Q_{BFL} (cfs)	Flow Rate at Inflection Point Q_i (cfs)
a	.75	1.0	.75	4.76	2.88
b	.67		.67		2.30
c	.55		.55		1.74

In the Cole's Colony design example the point d_c=0.55 ft, d_{BFL}=1.0 ft, characterize the Control Reach, consequently the ratio,

$$\frac{d_c}{d_{BFL}} = \frac{0.55\,ft}{1.0\,ft} = 0.55 \qquad \text{[Eqn D.13]}$$

- The flow rate at d_c/d_{BFL}=0.55 was estimated from Figure D.20 to be Q_c=1.74 cfs.
- Point (c) can be located on curve AE at a flow corresponding to Q_c=1.74 cfs.

- The DRC stage-discharge curve follows the curve A(c)B in Figure D.20. For the purpose of illustration, the stage-discharge curves for 30%OC (inflection point a) and 50%OC (inflection point b) are also provided in Figure D.20.

Step 5. Sizing the DRC Weir

After establishing the DRC stage-discharge curve the next step is to size the DRC weir. This is done using a nested weir configuration as illustrated in Figure D.21. The equation for the nested weir can be approximated from Eqn. D.14 for sharp crested weirs as,

$$Q = \left(C_e L_e h_e^{\left(\frac{3}{2}\right)} \right)_{INSET} + \left(C_e (L_e^* - L_e)(h_e^* - h_e)^{\left(\frac{3}{2}\right)} \right) \text{..........................[Eqn D.14]}$$

in which Q represents the discharge from the nested weir, 'C_e' is a coefficient (3.19) adjusted to account for end contractions, L_e is the length of the inset weir, h_e represents the height of the inset weir where $0 \le h_e \le h_2$ (h_2 represents the total height of the nested weir) and h_e^* is the depth of flow through the nested weir above the inset weir ($h_e \le h_e^* \le h_2$).

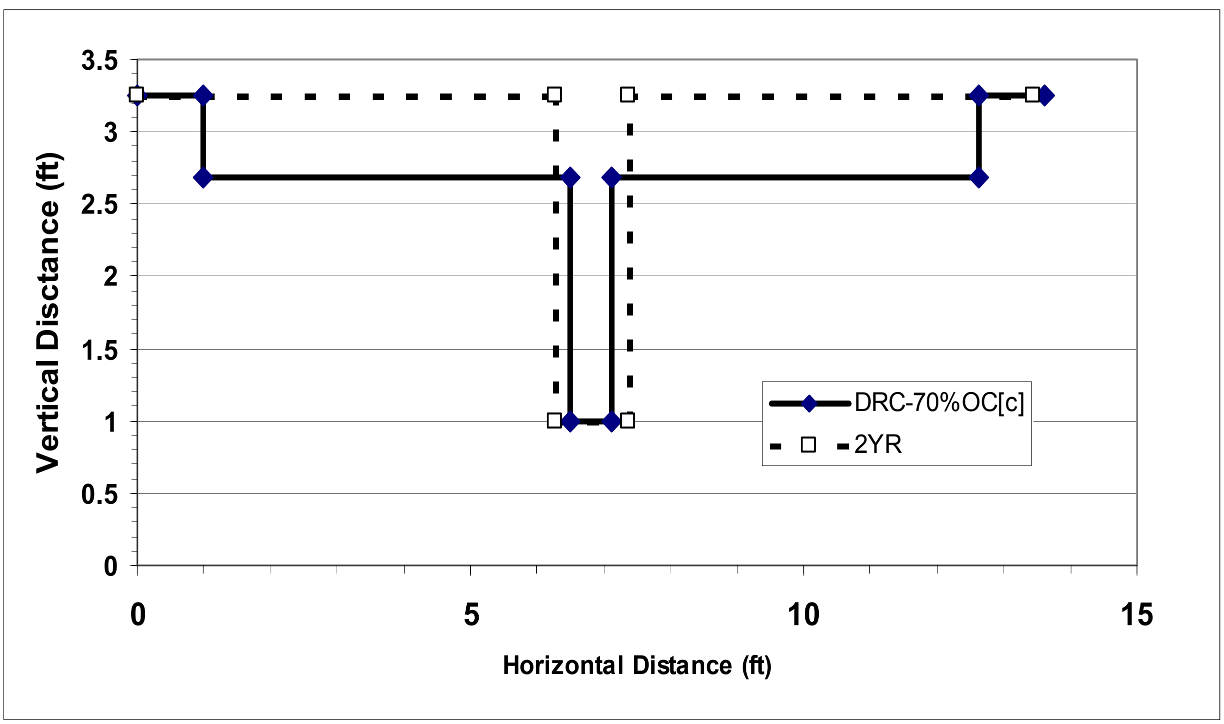

Figure D.21. Comparison of the 70% OC DRC weir with inflection point at [c] and the traditional 2-year peak flow attenuation weir.

Solving Eqn. D.14 for the Cole's Colony example results in the dimensions and flow values reported in Table D.13.

Table D.13. Summary of Dimensions and Flow Characteristics for a Nested DRC Weir: Cole's Colony Design Case (Reach 1).

Parameter	DRC Weir			2-Year Weir
	Inflection Point (a)	Inflection Point (b)	Inflection Point (c)	
L_e (ft)	1.77	1.00	0.62	N/a
h_e (ft)	0.67	0.78	0.93	
Q_i at h_e (cfs)	2.89	2.21	1.74	
L_e^* (ft)	0.80	4.32	11.0	0.83
h_2 (ft)	2.25			
Q at h_2 (cfs)	8.94			

Parameters in Table D.13 are defined in the preceding text.

Note: the weir dimensions for DRC stage discharge curves 30%OC (inflection point 'a') and 50%OC (inflection point 'b') are provided for comparison with the selected option (inflection point 'c').

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Appendix D10: Cold Climate Sizing Guidance

(Excerpt from Caraco, 1997)

Traditional BMP sizing criteria are based on the hydrology and climatic conditions of moderate climates. These criteria are not always applicable to cold climate regions due to snowmelt, rain-on-snow events and frozen soils. This chapter identifies methods to adjust water quality sizing criteria for cold climates.

Water Quality Sizing Criteria

The water quality volume is the portion of the BMP reserved to treat stormwater either through detention, filtration, infiltration or biological activity. Base criteria developed for BMP sizing nationwide are based on rainfall events in moderate climates (e.g., Schueler, 1992). Designers may wish to increase the water quality volume of BMPs to account for the unique conditions in colder climates, particularly when the spring snowfall represents a significant portion of the total rainfall. Spring snowmelt, rain-on-snow and rain-on-frozen ground may warrant higher treatment volumes. It is important to note that the base criteria required by a region must always be met, regardless of calculations made for cold climate conditions.

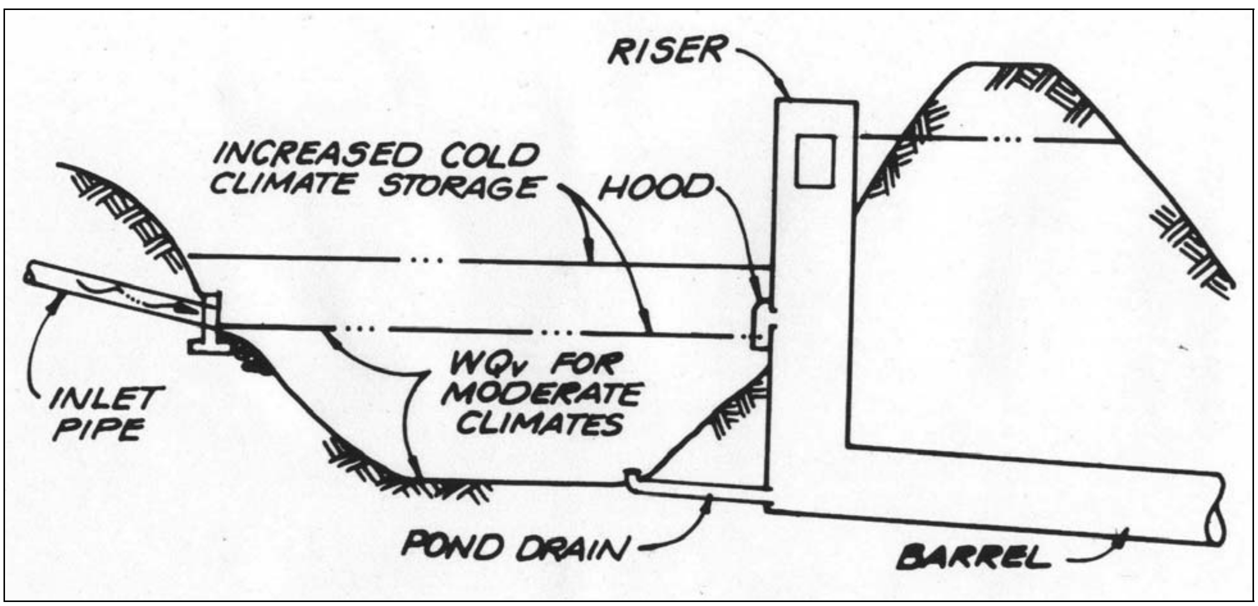


Figure D.22. Increased water quality volume in cold climates.

The goal of treating 90% of the annual pollutant load (Schueler, 1992), can be applied to snowmelt runoff and rain-on snow events. In the following conditions, cold climate sizing may be greater than base criteria sizing:

- Snowfall represents more than 10% of total annual precipitation. This value is chosen because, at least some portion of the spring snowmelt needs to be treated in order to treat 90% of annual runoff in these conditions. Using the rule of thumb that the moisture content of snowfall has about 10% moisture content, this rule can be simplified as:

Oversize when average annual snowfall depth is greater than or equal to annual precipitation depth.

- The area is in a coastal or Great Lakes region with more than 3' of snow annually. In these regions, rain-on-snow events occur frequently enough to justify oversizing stormwater BMPs for water quality.

The following caveats apply to the sizing criteria presented in this section:

- These criteria are not appropriate for very deep snowpacks (i.e., greater than 4') because the volume to be treated would be infeasible, and often unnecessary.
- Sizing for snow storage areas is described in Appendix C.
- Snowmelt is a complicated process, with large annual variations. While the criteria presented here address the affects of snowmelt and rain-on-snow, several simplifying assumptions are made. Where local data or experience are available, more sophisticated methods should be substituted.

Water Quality Volume for Snowmelt

In order to treat 90% of annual runoff volume, sizing for snowmelt events needs to be completed in the context of the precipitation for the entire year. In relatively dry regions that receive much of their precipitation as snowfall, the sizing is heavily influenced by the snowmelt event. On the other hand, in regions with high annual rainfall, storm events are more likely to carry the majority of pollutants annually. The sizing criteria for this section are based on three assumptions: 1) BMPs should be sized to treat the spring snowmelt event 2)Snowmelt runoff is influenced by the moisture content of the spring snowpack and soil moisture 3) No more than five percent of the annual runoff volume should bypass treatment during the spring snowmelt event and 4) BMPs can treat a snowmelt volume greater than their size.

- *BMPs should be sized to treat the spring snowmelt runoff event.*

  Snowmelt occurs throughout the winter in small, low-flow events. These events have high concentrations of soluble pollutants such as chlorides and metals, because of “preferential elution” from the snowpack (Jeffries, 1988). Although these events have significant pollutant loads, the flows are very low intensity, and generally will not affect BMP sizing decisions.

  The spring snowmelt, on the other hand, is higher in suspended solids and hydrophobic elements, such as hydrocarbons, which can remain in the snowpack until the last five to ten percent of water leaves the snowpack (Marsalek, 1991). In addition, a large volume of runoff occurs over a comparatively short period of time (i.e., approximately two weeks). Most BMPs rely on settling to treat pollutants, and the pollutants carried in the spring snowmelt are more easily treated by these mechanisms. In addition, the large flow volume during this event may be the critical water quality design event in many cold regions.

- *Snowmelt runoff is influenced by the moisture content of the spring snowpack and soil moisture.*

Because of small snowmelt events that occur throughout the winter, losses through sublimation, and management practices such as hauling snow to other locations, the snowpack only contains a fraction of the moisture from the winter snowfall. Thus, the remaining moisture in the snowpack can be estimated by:

$$M = 0.1 * S - L_1 - L_2 - L_3 \qquad \text{[Eqn D.15]}$$

Where: M=Moisture in the Spring Snowpack (inches)
S=Annual Snowfall (inches)
L_1, L_2 and L_3 = Losses to Hauling, Sublimation and Winter Melt, respectively.

The volume of snow hauled off site can be determined based on available information on current plowing practices. In most regions, sublimation to the atmosphere is not very important, but this volume should be calculated in dry or southern climates, such as in the Sierra Nevada region.

The design examples in this section use a simple “rule of thumb” approach, to estimate winter snowmelt for simplicity (Table D.14). The method assumes that winter snowmelt is influenced primarily by temperature, as represented by the average daily temperature for January. One half of the snow (adjusted for plowing and sublimation) is assumed to melt during the winter in very cold regions (Average T_{max} <25°F) and two thirds is assumed to melt during the winter in moderately cold regions (Average T_{max} <35°F). Winter snowmelt can be estimated using several methods, such as the simple degree-day method, or through more complex continuous modeling efforts.

Snowmelt is converted to runoff when the snowmelt rate exceeds the infiltration capacity of the soil. Although the rate of snowmelt is slow compared with rainfall events, snowmelt can cause significant runoff because of frozen soil conditions. The most important factors governing the volume of snowmelt runoff are the water content of the snowpack and the soil moisture content at the time the soil freezes (Granger et al., 1984). If the soil is relatively dry when it freezes, its permeability is retained. If, on the other hand, the soil is moist or saturated, the ice formed within the soil matrix acts as an impermeable layer, reducing infiltration. The section *Calculating the Snowmelt Runoff*, below, outlines a methodology for computing snowmelt runoff based on this principle.

Table D.14. Winter Snowmelt*

Adjusted Snowfall Moisture Equivalent	Winter Snowmelt	Winter Snowmelt (January
2"	1.0"	1.3"
4"	2.0"	2.7"
6"	3.0"	4.0"
8"	4.0"	5.3"
10"	5.0"	6.7"
12"	6.0"	8.0"

* Snowmelt occurring before the spring snowmelt event, based on the moisture content in the annual snowfall. The value in the first column is adjusted for losses due to sublimation and plowing off-site.

- *No more than 5% of the **annual runoff volume** should bypass treatment during spring snowmelt.*

In order to treat 90% of the annual runoff volume, at least some of the spring snowmelt, on average, will go un-treated. In addition, large storm events will bypass treatment during warmer months. Limiting the volume that bypasses treatment during the spring snowmelt to 5% of the annual runoff volume allows for these large storm events to pass through the facility untreated, while retaining the 90% treatment goal.

The resulting equation is:

$$T=(R_s-0.05R)A/12 \quad \text{[Eqn D.16]}$$

Where:

T = Volume Treated (acre-feet)
R_s = Snowmelt Runoff [See Section below: Calculating the Snowmelt Runoff]
R = Annual Runoff Volume (inches) [See Section below: Base Criteria/Annual Runoff]
A = Area (acres)

- *BMPs can treat a volume greater than their normal size.*

Snowmelt occurs over a long period of time, compared to storm events. Thus, the BMP does not have to treat the entire water quality treatment volume computed over twenty-four hours, but over a week or more. As a result, the necessary water quality volume in the structure will be lower than the treatment volume. For this manual, we have assumed a volume of ½ of the value of the computed treatment volume (T) calculated in equation D.16.

Thus,

$$WQ_v = ½ T \text{ .... [Eqn D.17]}$$

Base Criteria / Annual Runoff

The base criterion is the widely-used, traditional water quality sizing rule. This criterion, originally developed for moderate climates, represents the minimum recommended water quality treatment volume. In this manual, the runoff from a one-inch rainfall event is used as the base criteria. The basis behind this sizing criteria is that approximately 90% of the storms are treated using this event. This value may vary nationwide, depending on local historical rainfall frequency distribution data. However, the one-inch storm is used as a simplifying assumption. The base criteria included in this manual is chosen because it incorporates impervious area in the sizing of urban BMPs, and modifications are used nationwide. The cold climate sizing modifications used in this manual may be applied to any base criteria, however.

Runoff for rain events can be determined based on the Simple Method (Schueler, 1987).

$$r = p(0.05+0.9I) \text{ .... [Eqn D.18]}$$

Where: r = Event Rainfall Runoff (inches)
p = Event Precipitation (inches)
I = Impervious Area Fraction

Thus, the water quality volume for the base criteria can be determined by:

$$WQ_v = (0.05+.9I)\ A/12 \text{ .... [Eqn D.19]}$$

Where: WQ_v = Water Quality Volume (acre-feet)
I = Impervious Fraction
A = Area (acres)

The Simple Method can also be used to determine the annual runoff volume. An additional factor, P_j, is added because some storms do not cause runoff. Assume P_j = 0.9 (Schueler, 1987). Therefore, annual runoff volume from rain can be determined by:

$$R = 0.9\ P\ (0.05+.9I) \text{ .... [Eqn D.20]}$$

Where: R = Annual Runoff (inches)
P = Annual Rainfall (inches)

Calculating the Snowmelt Runoff

To complete water quality sizing, it is necessary to calculate the snowmelt runoff. Several methods are available, including complex modeling measures. For the water quality volume, however, simpler sizing methods can be used since the total water quality volume, not peak flow, is critical. One method, modified from Granger et al. (1984) is proposed here. Other methods can be used, particularly those adjusted to local conditions.

According to Granger et al. (1984) the infiltration into pervious soils is primarily based on the saturation of the soils prior to freezing. While saturated soils allow relatively little snowmelt to infiltrate, dry soils have a high capacity for infiltration. Thus, infiltration volumes vary between wet, moderate and dry soil conditions (Figure D.23).

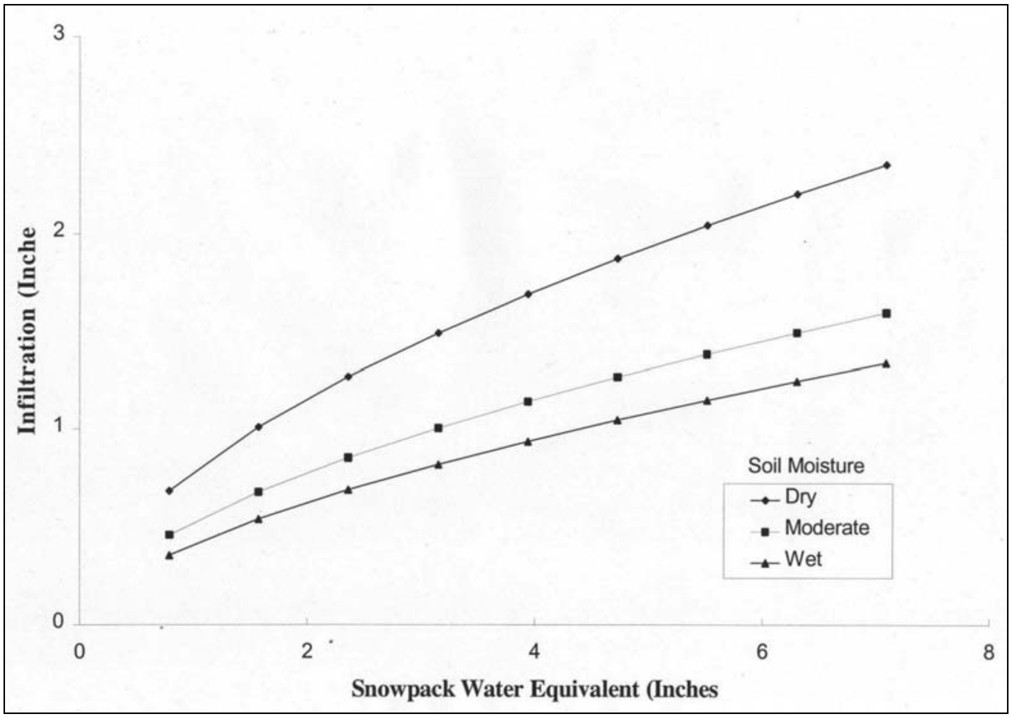


Figure D.23. Snowmelt infiltration based on soil moisture.

Assume also that impervious area produces 100% runoff. The actual percent of snowmelt converted to runoff from impervious areas such as roads and sidewalks may be less than 100% due to snow removal, deposition storage and sublimation. However, stockpiled areas adjacent to paved surfaces often exhibit increased runoff rates because of the high moisture content in the stockpiled snow (Buttle and Xu, 1988). This increased contribution from pervious areas offsets the reduced runoff rates from cleared roads and sidewalks.

The resulting equation to calculate snowmelt runoff volume based on these assumptions is:

R_S = [runoff generated from the pervious areas] + [runoff from the impervious areas]

R_S = [(1 - I)(M-Inf)] + [(I)(1)(M)] .... [Eqn D.21]

where:

R_S = Snowmelt Runoff

I = Impervious Fraction
M = Snowmelt (inches)
Inf = Infiltration (inches)

Sizing Example 1: Snowpack Treatment

Scenario: 50 Acre Watershed
40% Impervious Area
Average Annual Snowfall= 5'=60"
Average Daily Maximum January Temperature= 20E
Average Annual Precipitation = 30"
20% of snowfall is hauled off site
Sublimation is not significant
Pre-winter soil conditions: moderate moisture.

Step 1: Determine if oversizing is necessary
Since the average annual precipitation is only ½ of average annual snowfall depth, oversizing is needed.

Step 2: Determine the annual losses from sublimation and snow plowing.
Since snow hauled off site is about 20% of annual snowfall, the loss from snow hauling, L_1, can be estimated by:
$L_1 = (0.2)(0.1)S$
Where: L_1 = Water equivalent lost to hauling snow off site (inches)
S = Annual snowfall (inches)
0.1 = Factor to convert snowfall to water equivalent

Therefore, the loss to snow hauling is equal to:
$L_1 = (0.2)(0.1)(60")$
$L_1 = 1.2"$

Since sublimation is negligible, $L_2 = 0$

Step 3: Determine the annual water equivalent loss from winter snowmelt events
Using the information in Step 2, the moisture equivalent in the snowpack remaining after hauling is equal to:
$60" \times 0.1 - 1.2" = 4.8"$

Substituting this value into Table D.14, and interpolating, find the volume lost to winter melt, L_3.
$L_3 = 2.4"$

Step 4: Calculate the final snowpack water equivalent, M
$M = 0.1 \times S - L_1 - L_2 - L_3$ (Equation D.1)
$S = 60"$
$L_1 = 1.2"$
$L_1 = 0"$
$L_3 = 2.4"$

Therefore, M = 2.4"

Step 5: Calculate the snowmelt runoff volume, R_s

$R_s = (1\text{-}I)(M\text{-}Inf)+ I \cdot M$ Equation D.7

M =2.4"

I =0.4

Inf =0.8" (from Figure D.23; assume average moisture)

Therefore, R_S = 1.9"

Step 6: Determine the annual runoff volume, R

Use the Simple Method to calculate rainfall runoff:

R=0.9(0.05+0.9*I)P (Equation D.20)

I=0.4

P=30"

Therefore, R=11"

Step 7: Determine the runoff to be treated

Treatment, T should equal:

T= (R_s-0.05*R) A/12 (Equation D.16)

R_s=1.9"

R =11"

A = 50 Acres

Therefore, T=5.6 acre-feet

Step 8: Size the BMP

The volume treated by the base criteria would be:

WQ_v=(.05+.9*.4)(1'/12")(50 acres) = 1.7 acre-feet (Equation D.19)

For cold climates:

WQ_v=1/2(T) = 2.8 acre-feet (Equation D.17)

The cold climate sizing criteria is larger, and should be used to size the BMP.

Rain-on-Snow Events

For water quality volume, an analysis of rain-on-snow events is important in coastal regions. In non-coastal regions, rain-on-snow events may occur annually but are not statistically of sufficient volume to affect water quality sizing, especially after snowpack size is considered. In coastal regions, on the other hand, flooding and annual snowmelt are often driven by rain-on-snow events (Zuzel et al., 1983). Nearly 100% of the rain from rain-on-snow events and rain immediately following the spring melt is converted to runoff (Bengtsson, 1990). Although the small rainfall events typically used for BMP water quality do not produce a significant amount of snowmelt (ACOE, 1956), runoff produced by these events is high because of frozen and saturated ground under snow cover.

Many water quality volume sizing rules are based on treating a certain frequency rainfall event, such as treating the 1-year, 24-hour rainfall event. The rationale of treating 90% of the pollutant load (Schueler, 1992) can also be applied to rain-on-snow events, as shown in the following example.

Sizing Example 2: Rain-on-Snow

Scenario: Portland, Maine
50 Acre Watershed
30% Impervious Area
Data Requirements: Snowfall, Precipitation

Step 1: Develop a rain-on-snow data set.
Find all the rainfall events that occur during snowy months. Rainfall from December through April were included. Please note that precipitation data includes both rainfall and snowfall, and only data from days without snowfall should be included. Exclude non-runoff-producing events (less than 0.1"). Some of these events may not actually occur while snow is on the ground, but they represent a fairly accurate estimate of these events.

Step 2: Calculate a runoff distribution for rain-on-snow events
Since rain-on-snow events contribute directly to runoff, the runoff distribution is the same as the precipitation distribution in Figure D.24.

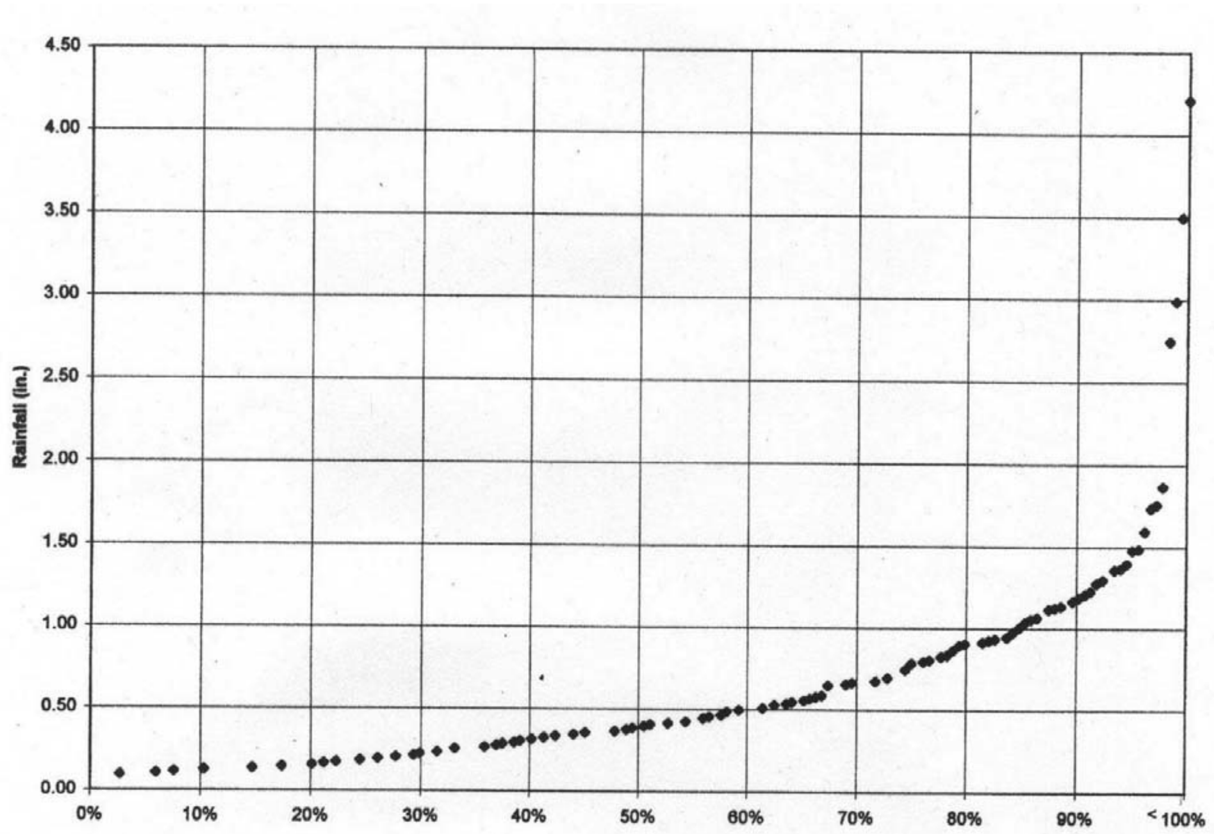


Figure D.24. Rainfall Distribution for Snowy Months

Step 3: Calculate a rainfall distribution for non-snow months.
Develop a distribution of rainfall for months where snow is not normally on the ground. The rainfall distribution for May through November is included in Figure D.25.

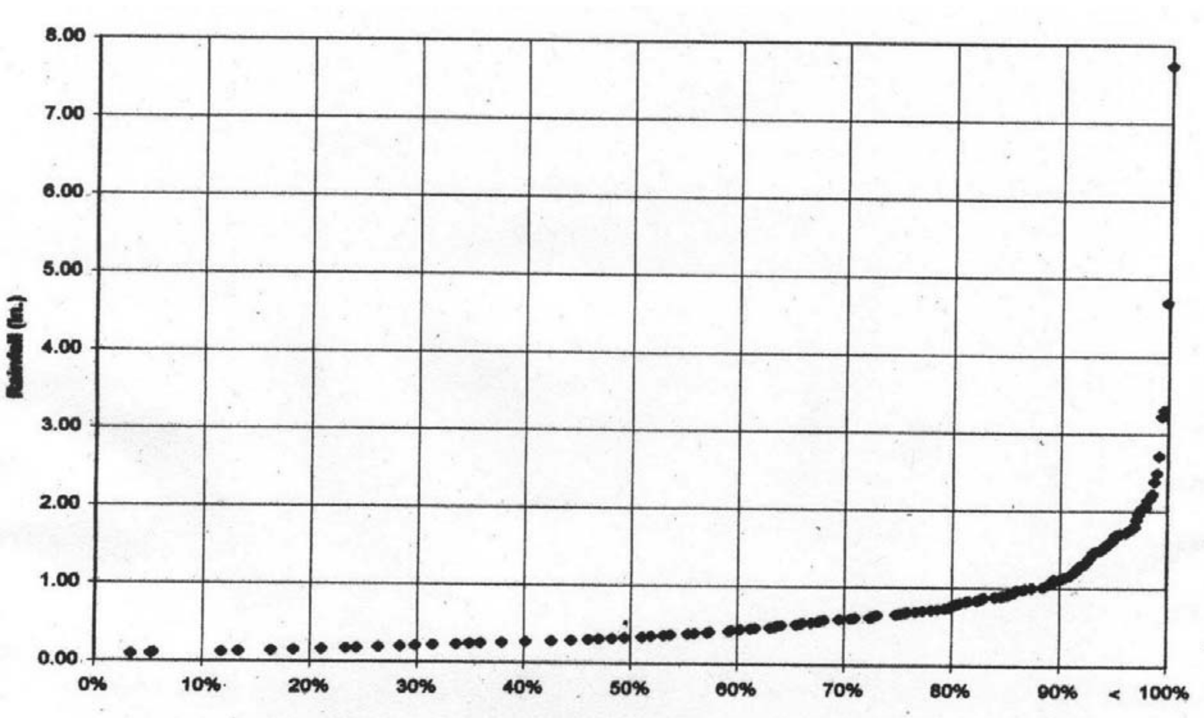


Figure D.25

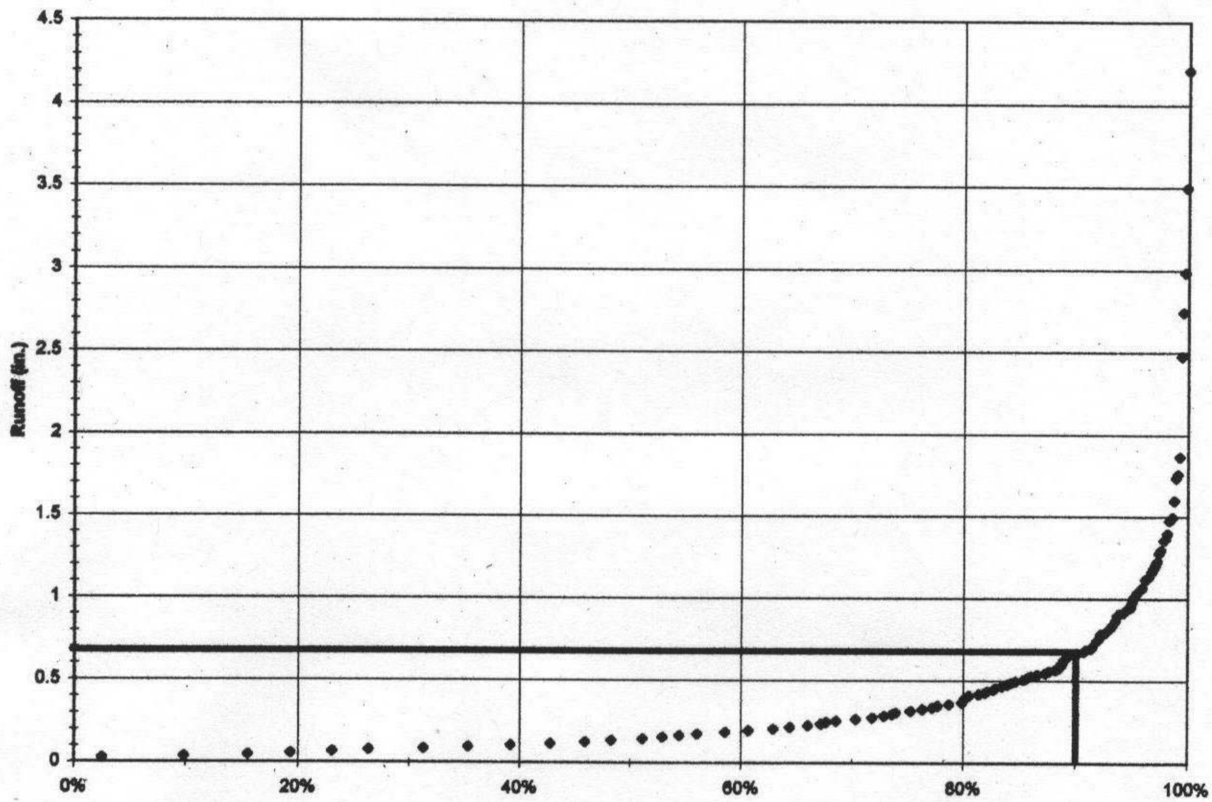
Step 4: Calculate the runoff distribution for non-snow months.
Use a standard method to convert rainfall to runoff, particularly methods that are calibrated to local conditions. For this example, use the Simple Method. Runoff is calculated as:

r=(0.05+0.9 I)p (Equation D.18)

For this example, I=0.3 (30% impervious area), so: r=0.32 p

The runoff distribution for non-snow months is calculated by multiplying the rainfall in Figure D.25 by 0.32.

Step 5: Combine the runoff distributions calculated in Steps 2 and 4 to produce an annual runoff distribution. The resulting runoff distribution (Figure D.26) will be used to calculate the water quality volume.



Step 6: Size the BMP.
In this case, use the 90% frequency runoff event (Figure D.25), or 0.65 watershed inches. This value is greater than the base criteria of 0.32 watershed inches (1" storm runoff). Therefore, the greater value is used.

WQ_v=(0.65 inches) (1 foot/12 inches) (50 acres) = 2.7 acre-feet