G3691-P

The Wisconsin Storm Water MANUAL

Technical Design Guidelines for Storm Water Management Practices

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Disclaimer

This manual is intended for use by engineers, planners, government administrators and other professionals involved in storm water management. It provides guidance on the design of management practices to achieve both water quality and water quantity control, with a particular emphasis on the water quality considerations of storm water management. Historically, storm water facilities were built for flood control. Today, however, it is widely recognized that the pollutant load associated with storm water runoff is a significant problem and that storm water facilities must be built to improve water quality.

The authors have drawn from an extensive literature review, the experience of others and their own personal experience with demonstration projects and state monitoring sites to develop this material. The manual should be considered a tool to help designers understand concerns about storm water management and approaches to designing appropriate storm water management practices.

Keep in mind that parameter values used in the *Wisconsin Storm Water Manual* are for illustration only. Local ordinances and state regulations may establish specific design techniques, such as the method used to determine peak flow rates or runoff volumes, as well as parameter values such as design-level storm return period and duration. Users of the manual must check with local and state authorities to determine local controls for design, and obtain any local, state or federal permits required by law.

The *Wisconsin Storm Water Manual* will be updated periodically and the authors welcome any comments or corrections from professionals involved in storm water management. Appropriate comments will be incorporated into future revisions to the manual.

Introduction

The *Wisconsin Storm Water Manual* was developed to provide guidance to storm water management professionals who select, site, design, construct and maintain storm water management practices. The manual focuses on the applicability, technical design, construction and maintenance of a range of storm water management practices. It presents suggested performance standards for storm water discharge quality and quantity, and provides enough information for an engineering professional to design the water quality components of a storm water management practice or combination of practices. It focuses primarily on controlling the water quality of storm water discharged from relatively small rainfall events, which are responsible for the majority of the annual pollutant loading. While it is recognized that control of peak flow discharges associated with larger rainfall events (10- and 100-year recurrence intervals) is a necessary part of a storm water management program, technical design guidelines addressing these larger rainfall events are not covered here. Design guidelines for controlling the runoff from these larger rainfall events can be found in traditional engineering reference materials.

The *Wisconsin Storm Water Manual* is composed of seven publications, each covering a specific storm water quality topic or practice.

Overview and Screening Criteria (G3691-1) presents basic storm water management practice performance guidelines relating to discharge quality and quantity. These performance guidelines are taken from the standards section (S.07) of the State Model Storm Water Zoning Ordinance. The publication compares the strengths, weaknesses and siting limitations of various structural storm water best management practices.

Hydrology (G3691-2) presents two methods for developing runoff hydrographs for predicting runoff from the relatively small rainstorms responsible for frequent storm water discharges. An alternative table-top method is presented for calculating runoff volumes for small rainfall events so that management practices can be sized to be cost-effective. The remaining five publications in the series provide information on the general principles, planning guidelines, design guidelines, construction guidelines and maintenance considerations for the following management practices: *Infiltration Basins and Trenches* (G3691-3), *Wet Detention Basins* (G3691-4), *Artificial Wetlands* (G3691-5), *Filter Strips* (G3691-6) and *Grassed Swales* (G3691-7).

G3691-1

The Wisconsin Storm Water MANUAL

Overview and Screening Criteria

This manual is intended for use by engineers, planners and other professionals involved in storm water management.

Historically, storm water facilities have been built for flood control. Now, however, it is widely recognized that the pollutants associated with storm water runoff are a significant problem and that facilities must be built to improve water quality and control peak flows.

The guidelines presented in this manual will help professionals design management practices that achieve control over both water quality and quantity, although special emphasis is given to water quality considerations.

An extensive literature review, the experience of other states and personal experience gained from demonstration projects and state monitoring sites were all used in compiling this information. While the manual offers guidelines for the designers of storm water practices, bear in mind that even strict adherence to these guidelines does not relieve the user from the need to obtain local, state or federal permits required by law.

The information covered here is not all-inclusive. Storm water management practices undergo continuous evaluation and this material will be updated periodically as new information becomes available. Comments from professionals involved in storm water management are welcomed and will help guide revisions to future editions.

Urbanization often increases the volume, peak discharge, temperature and pollutant content of storm water. All of these factors add stress to urban streams, lakes and wetlands, and in some cases exacerbate other problems created by channel modification, illicit point source discharges and spills. As a result, many of the state's urban water resources only partially support the recreational opportunities, aquatic life and aesthetic values of which they were once capable.

Professional understanding of storm water management problems and solutions varies greatly depending on which facet of this complicated issue is being examined. For example, the processes by which storm water pollutants are generated and conveyed to surface waters is fairly well established, as are procedures for estimating annual storm water pollutant loadings and determining the relative significance of individual source areas (such as rooftops and parking lots).

However, the processes through which specific pollutants interact with aquatic systems to cause environmental damage are less well understood. The techniques for selecting and designing the most cost-effective and practical best management practices are constantly being refined as the effects of both pollutants and treatment methods become better understood.

In any effort to manage a complicated natural resource problem such as storm water management, a point is reached at which action is taken, even though the knowledge base is incomplete. Management decisions are usually based on an assessment of the relative merits of doing nothing and having environmental problems grow worse, versus committing financial resources for solutions that might be deficient or inefficient in addressing the problem.

Although there is still much to learn about cost-effective approaches to storm water management, the point has been reached at which educated steps can be taken toward reducing storm water impacts.

Developing areas are an obvious choice for implementing storm water management practices. The costs of incorporating management practices into new developments is usually less than trying to retrofit practices into established urban areas. New developments also present opportunities for incorporating comprehensive land use planning, storm water planning and education into both site-specific and regional development plans.

Established urban areas present different opportunities. Retrofitting storm water practices into developed areas can be very costly and the choice of control measures is limited. Low-risk efforts in these areas include such non-structural approaches as information and education programs, pollution prevention, curbing illicit dumping and illicit connections from business and industry into storm sewer systems, and modifying existing flood and drainage control practices where needed to address water quality concerns.

More than 40 communities and thousands of industries statewide are involved in storm water management through the Wisconsin Priority Watersheds Program and the Wisconsin Pollutant Discharge Elimination System. This pattern is being repeated in many other states. As a result, there is a growing wealth of experience in dealing with storm water management issues.

This manual presents some technical and regulatory tools that can be used to address storm water management problems in existing and developing areas. The tools are not perfect. Storm water performance criteria, cost-effective management practices and effective administrative and financial measures will continue to evolve.

Storm water discharge performance guidelines

Storm water management guidelines should address concerns about the quantity and quality of storm water runoff. Guidelines define the level of treatment that stormwater management measures should achieve. They help ensure that the hydrologic regime and pollutant burden to receiving waters do not limit recreation, aquatic life or aesthetics, or endanger property, public health and safety.

Note: All hydrologic design parameters and performance guidelines used in this manual are generic. State and local regulations must be consulted to determine legal requirements in specific areas.

Guidelines may be of two types. Some are performance-based, such as a requirement to remove 80% of a particular pollutant. Others are prescriptive, such as a requirement for pretreatment or a separation distance to water supply wells.

Several factors must be recognized before using the guidelines presented here:

- The guidelines in this manual do not constitute regulatory standards. Guidelines used in the design chapters are generic, and should be used only as aids in selecting and designing stormwater management practices. Detailed storm water management plans or ordinances for specific areas may contain modifications or additions to these generic guidelines. In these circumstances, adjustment of some design parameters might be appropriate.
- The examples in this manual address water quality and peak flow control for the runoff from relatively small design storms with return frequencies of two years or less. Such storms are commonly used to design practices for water quality control. Guidelines for controlling runoff from larger design storms (10-year to 100-year) can be found in other manuals.

Guidelines for quantity discharge

Controlling peak flow discharge rates

Storm water practices should be designed so that the peak flow discharge rates from developing areas are maintained at pre-development levels or at flow rates designated by local ordinances.

The term pre-development conditions implies that the land is under good management practices. This criterion should be met for a series of design storms, including at a minimum the 2-, 10- and 100-year design storms.

The 2-year event is included to help control changes in the morphology of receiving streams and to control frequent scouring of benthic habitat. The 10-year event is included to reduce surcharging of minor drainage system components which can lead to inconvenience and property damage. The 100-year event is included to prevent increases in the regulatory floodplain that may result in damage to property, threaten human health and safety, and lead to solutions such as channel lining and realignment that may be catastrophic to aquatic habitat.

Controlling runoff volume

Storm water runoff volumes should be kept as close to pre-development conditions as practical. This requires maintaining the natural infiltration capacity of land development sites or creating infiltration zones to handle runoff from impervious areas. Maintaining infiltration capacity will help maintain stream base flows and limit the duration and frequency of bank-full flood flows for streams. Maintaining infiltration capacity will also help avoid significant changes to wetlands.

Maintaining runoff volumes at pre-development levels will be very difficult in many situations, and should probably be considered more of a qualitative guideline than those established for peak flow rate maintenance. However, the goal of maintaining pre-development runoff volumes should be pursued.

Studies in the Midwestern United States have shown that 90% of the average annual rainfall depth is produced from rains equal to or less than about 1 inch (Roesner, L., et al, 1991; Pitt, R., 1991). Practices that encourage infiltration of storm water from these numerous small rainfall events may also effectively improve water quality.

Guidelines for discharge quality

It is well-established that relatively small storms are responsible for the majority of the annual pollutant loads in urban runoff (Schueler, 1987; Pitt, 1989; Roesner, 1991) and that the runoff volume is the critical determinant of pollutant loading and control (Pitt, 1989). Therefore, management practices designed for water quality control need to adequately treat these frequent, relatively small storms.

The federal Coastal Nonpoint Source Control Program indicates that treating the 2-year, 24-hour storm event is the appropriate volume to achieve an 80% reduction in the total suspended solids washing off urban surfaces (U.S. EPA, 1993).

However, other research indicates that a smaller design storm can be used to attain the same reduction levels more cost effectively (Roesner, 1991; Pitt, 1989). The Wisconsin Department of Natural Resources conducted a study using two different computer models, the P8-Urban Catchment Model and the Source Loading and Management Model, to select an appropriate rainfall event. They concluded that rainfall events of 1.25 to 1.5 inches in magnitude were adequate to achieve the 80% reduction goal (WDNR, 1997).

Control of pollutant loads is critical to environmental management. Natural wetlands should not be used as primary treatment systems for storm water runoff, and should be protected against significant impacts from pollutants. When storm water discharges to wetlands are unavoidable, the discharges should be pre-treated to remove particulate pollutants, oily residues and other pollutants.

> **Note:** Design storms should be selected to balance economic benefits with the risks to the environment and human health. Therefore, it is necessary to make an engineering judgement for each site. These guidelines are offered as suggestions for design under most conditions.

Groundwater should be protected against contamination from polluted storm water. Direct infiltration of storm water should be restricted to runoff from relatively clean areas such as lawns, rooftops, sidewalks and driveways.

Storm water runoff from the more contaminated urban areas, such as residential and commercial streets, commercial parking lots and non-manufacturing industrial areas, may be safely infiltrated as long as adequate precautions are taken. These include pre-treating the storm water from these sources prior to discharge to the infiltration device, reviewing site characteristics and practice designs, monitoring practice construction, and operation and maintaining the pre-treatment and infiltration devices in good working order.

Consideration must also be given to protection of drinking water. Underground injection of storm water is prohibited under state law. Generally speaking, this prohibition includes discharge of storm water into the ground via openings or excavations, such as french drains or drywells, if such holes are deeper than they are wide. It is unclear at this time whether the Federal Underground Injection Control Program will extend this prohibition to buried, horizontal, perforated piping.

In addition, legal separation distances must be observed when siting management practices near water supply wells. Detention ponds and storm water infiltration devices should not be located within a 1,200-foot radius of a community water supply well or within the defined recharge area surrounding a community water supply well for which a wellhead protection plan has been established. Ponds and infiltration practices should be separated by at least 100 feet from private water supply systems.

Summary of design recommendations

Peak control

Peak flow rates should be based upon the 2-, 10- and 100-year storm events. The 2-year event is included to help prevent damage to the morphology of receiving streams and to control frequent scouring of benthic habitat. The 10-year event is included to reduce surcharging of minor drainage system components that can lead to inconvenience and property damage. The 100-year event is included to protect against increases in the regulatory floodplain that may result in damage to property, threaten human health and safety, and lead to solutions such as channel lining and realignment that may be catastrophic to aquatic habitat.

Water quality control

Recommendations for volume control range from the 2-year, 24-hour storm to storms of 1.25 to 1.50 inches in magnitude. The Wisconsin Department of Natural Resources currently recommends a design value of 1.50 inches for management practices whose primary function is water quality improvement.

Screening criteria for individual management practices

At the heart of urban runoff management is the ability to control the quantity and quality of the storm water. Given the variety of practices available to the planner or engineer, it is important that an initial screening of available practices takes place to match the best practices to the site conditions and overall management objectives. The management objectives of the community and the developer should include:

- Removing pollutants to a specified level
- Reproducing as closely as possible the pre-development hydrologic conditions
- Selecting a management practice that is cost-effective, will not represent an excessive maintenance burden and will not have detrimental side-effects on the environment
- Fitting the practice to the site

A planner or engineer should first consult local storm water ordinances or municipal storm water management plans. Information from these sources will cover the storm water management objectives, design criteria and some site considerations.

A storm water plan should contain the following information:

- The plan's specific, quantified goals; for example, the improvement or control level in the water resource desired in the watershed
- Identification of the natural or environmentally sensitive areas that could be affected by the project. These may include waterways, environmental corridors, lakes and streams, floodplains, wetlands and other significant natural areas
- Existing and proposed land uses for the area and likely pollutants from these uses.
- Existing and planned storm water conveyance system
- General soil types in the area
- Nominal land slopes of the area and any unusually steep or erosive sections

When the water resource management needs of the watershed are known, the types of structures that will best protect them should be determined. Material presented in this chapter helps evaluate alternatives by providing tables to compare physical suitability, storm water benefits, pollutant removal benefits and environmental amenities of the various management practices. Prior to consulting the tables, however, the engineer should collect the following site information.

Hydrologic conditions. Hydrologic conditions should be analyzed, including time of concentration of the watershed and potential runoff from the 2-year, 24-hour, 10-year, 24-hour, and 100-year, 24-hour events as well as the water quality control runoff volume for the design storm. The water quality control event as defined in this manual is the maximum sized storm which must be controlled to achieve 80% removal of suspended solids.

A 1.5-inch storm is used for illustrative purposes throughout this manual. Identification of the type of receiving water, tributary drainage area, estimation of the management practice storage requirement and the need for diversion structures to protect the practice or divert the flow into storage should be included in the hydrologic analysis.

Soils. An investigation of the soils at potential structure sites with regard to the infiltration rates, depth to groundwater and bedrock and structural stability will be needed. Infiltration rates are important in determining if practices using infiltration as the primary treatment mechanism will be possible, or if the soil limits infiltration or permits ponding of runoff. In some cases, the structural stability may limit the choice of management practices.

Site. Land that may be selected for a storm water practice should be evaluated to determine whether it can be acquired economically and is of sufficient size for the proposed management practice.

Ordinances. Local ordinances that require safety and aesthetic features as well as water quantity or quality controls should be reviewed, along with other state or federal requirements or regulations that might apply.

With the storm water management objectives and site information in hand, the following screening tables and notes can be used to assess each practice's applicability and limitations. These tables are modifications of tables and charts taken from *Fundamentals of Urban Runoff* Management by Horner et al. (1994) and are similar to charts developed by Schueler (1987) and modified by the British Columbia Research Corporation (1992).

Physical suitability

Tables 1–3 assist in determining if a practice is physically suited to the site. The size of the drainage area and soil characteristics are two conditions that influence siting. If the tables indicate marginal applicability for certain practices, design changes or local standards or conditions may still make the practices viable.

Catchment area

The catchment or watershed area served is a significant parameter in the selection of an appropriate management practice. For example, ponds require a minimum area of about 10 acres to ensure sufficient flow to maintain water levels, to allow for economy of scale during construction, to accommodate the physical limitations of earth-moving equipment, and to allow the pond to operate effectively.

Other practices such as oil/grit separators or infiltration systems, are limited by the maximum runoff and, therefore, the drainage area they can handle. In some cases the watershed area may be too large or too small for certain management practices.

If an infiltration management practice is desired, for example, a large watershed may need to be subdivided into smaller catchment areas with individual treatment systems. In other cases it may be possible to combine flows from subwatersheds to create flows that meet minimum standards for the practice.

Soil characteristics

Soil characteristics are important for most management practices. Selection can be limited by soils, bedrock or depth to groundwater.

Infiltration practices have a very restrictive range in which they will operate effectively. The soils must be able to infiltrate the water quickly enough to be economically feasible, but slowly enough to provide treatment.

Ponds can function in a broader range of soil types than infiltration structures, but they must also be checked for water storage capability and structural stability. In cases of high infiltration rates, ponds can be lined to maintain a minimum depth in the permanent pool.

Table 1. Applicability of treatment practices relative to catchment areas

	Catchment area (acres)	
Practice	**Feasible**	**Marginal**
Oil-grit separators	0–5	5–7.5
Wet-pond/artificial wetland	18–100	10–18
Vegetated swale/filter strip	0–5	5–10
Infiltration basin	4–20	0.5–4 and 20–50
Infiltration trench	0–5	5–10
Porous pavement	1.5–8	0–1.5 and 8–18

Table 2. Applicability of treatment practices relative to infiltration rate

	Minimum infiltration rate (inches/hr)	
Practice	**Feasible**	**Marginal**
Wet-pond/artificial wetland	0.02–0.7	0.7–8.0*
Vegetated swale/filter strip	0.08–2.4	0.02–0.08 & 2.4–7.0
Infiltration basin	0.02–0.3	0.3–0.5
Infiltration trench	0.02–0.3	0.3–0.5
Porous pavement	0.02–.0.3	0.3–0.5
Oil-grit separators/sand filters	0.02–8.3	———

*May require lining

Table 3. Constraints on treatment practices

Practice	**Slope**	**High water table**	**Close to bedrock**	**Proximity to foundation**	**Space consumption**	**Maximum depth limitation**	**High sediment input**	**Thermal impacts**
Oil-grit separators	++	++	+	+	++	++	+	++
Wet pond/ artificial wetland	++	++	+	++	0	0	+	0
Vegetated swale	+	0	+	+	++	++	0	++
Vegetated filter strip	+	+	+	+	++	++	0	++
Infiltration basin	+	0	0	+	+	0	0	++
Infiltration trench	0	0	0	0	++	0	0	++
Porous pavement	0	0	0	0	0	0	0	++
Sand filter	++	+	+	++	++	+	0	++

++ Generally not a restriction + Can be overcome with careful design 0 May preclude the use of the practice

Source: Horner, 1994

Slope

The slope of the drainage area has a minor effect on pond practices, but a significant effect on most infiltration practices and grass filters. Problems occur when channelized flow makes it difficult to reduce the velocity and spread the inflow across the practice's surface area, or when uniform infiltration cannot be achieved because the land slopes.

High water table

A seasonally high groundwater table is a severe limitation to infiltration systems if the water table is within 5 feet of the bottom of the practice. A high water table may restrict the ability of the site to infiltrate water, but more importantly it increases the potential for groundwater contamination. Ponds may need to be lined to prevent groundwater contamination if the water table is too near to the land surface.

Proximity to bedrock

Bedrock within five feet of the bottom of the management practice is also a major limitation in siting. If the flow downward is blocked by bedrock, infiltration practices will not drain in a reasonable period of time. A related concern with shallow, fractured bedrock is that contaminated surface water may enter a fracture and move rapidly to the groundwater.

Proximity to foundations and wells

Given the potential for groundwater contamination from infiltration practices, locating them close to a drinking water well is not advisable. In general, infiltration practices need to be located at least 100 feet from a private well. For municipal water supply wells, no storm water infiltration structure should be located within 1,200 feet of the well or as otherwise specified in a wellhead protection area. Local ordinances should be checked, as they may specify even greater separation distances. It is recommended that ponds be no closer than 100 feet up gradient or 10 feet down gradient of a building foundation.

Space consumption

Developed areas or sites bordered by restricted areas may not lend themselves to the use of ponds or artificial wetlands that require larger surface areas. Management practices that require less land area may be more appropriate under these conditions.

Maximum depth limitation

The maximum depth limitation varies with the practice, the soil type and maintenance requirements. For example, infiltration practices should drain in 72 hours, which may limit their depth. Ponds deeper than 8 feet tend to stratify and release pollutants from the anoxic zone.

A pond's depth may be further limited by the need to remove sediment and the availability of equipment for this task. Maintenance accessibility also may limit depth for mechanical units such as oil/grit separators and sand filters.

High sediment input

Very few management practices can withstand heavy sediment loading. Sediment basins designed for high sediment loads are often larger than permanent storm water ponds but can be converted into permanent ponds after construction.

Conversions should be considered during the initial design. If a sediment pond is converted to a permanent wet pond after construction, it should be excavated to the final design depth and regraded. The upland area should be stabilized before receiving storm water flows. If this is not done, significant pond volume can be lost to excessive sediment.

Infiltration management practices will fail if they receive high sediment loads. The infiltration practice location must be protected from all traffic or storm water flow until the construction site is stabilized. Pre-treatment is advisable for many management practices, in the form of either a sediment forebay for detention ponds or in a pre-treatment unit before the infiltration practice.

Thermal impacts

Some practices raise the water temperature to undesirable levels during the hot summer months, and/or long dry periods. Water in shallow ponds or artificial wetlands may experience as much as a 5- to 10-degree rise in temperature. If these devices discharge to a stream classified as a cold water fishery, the resulting rise in water temperature in the stream can have a negative impact on the aquatic biota. Infiltration practices normally do not raise water temperatures and will not have adverse thermal effects on cold water fisheries.

Water quantity

The second area to consider is the water quantity control capabilities of the selected management practices. Different practices provide different levels of control for peak discharge, volume, groundwater recharge or streambank protection. Table 4 provides a comparison of the individual management practices and their ability to provide water quantity control.

Peak discharge control

Ponds are the best management practice for controlling peak discharge. Peak discharge can be controlled by holding the runoff volume and releasing it at the pre-determined flow rate. The same pond can have several outlet control devices at different elevations to control the peak discharge from 2-, 10- and 100-year storms.

If the 2-year, 24-hour event is controlled, the shape and form of the receiving channel will be maintained and stream degradation slowed. Control of the 10-year, 24-hour storm is often used for downstream storm water conveyance capacity considerations and may vary with local requirements and local conditions. The 100-year storm is often used as the model to design the emergency spillway.

Volume control

Infiltration devices, which release water to the subsoil, are the best way to reduce storm water runoff volume for small and possibly medium-sized storms. Ponds are designed to detain flows for later release, so they are not good practices for volume control. Volume control is usually not practical or economically feasible for large storms.

Groundwater recharge/low flow maintenance

As a result of development, storm water that once found its way to the groundwater typically runs off impervious surfaces into storm drains, and then flows into a surface water body. However, if the runoff is directed into infiltration structures, the water will help recharge groundwater and, in turn, help maintain base flow levels in headwater streams during the dry summer months.

Streambank erosion control

The bankfull flow condition defines basic stream morphology. Without development, the bankfull condition is believed to occur on the average of once every one to two years. If flows increase due to development , the stream will cut a new bed to reach a new equilibrium, often resulting in excessive streambank erosion. The peak discharge control on the 2-year, 24-hour design storm will help to maintain the pre-development peak flow rate and help control stream bank erosion.

This is a desired goal for runoff control design. However, this peak shaving for the 2-year storm event may not prevent smaller storms from generating more runoff than in the pre-development condition. As a result, the frequency of the bankfull condition may increase.

A management practice that can provide streambank erosion control should be able to store enough of the runoff volume from rains less than the 2-year, 24-hour rain to preserve the frequency of the pre-development bankfull condition.

Not all management practices can provide both peak shaving and bankfull frequency control. Bankfull frequency control can be achieved by extending the detention time of the water. Schueler (1987) provides a procedure to analyze bankfull flooding frequency and summarizes design considerations to prevent an increase in frequency.

Pollutant removal

Variations in management practice design, such as increasing detention time or surrounding a wet pond with a shallow marsh, can improve pollutant removal capability. Pollutant removal can be accomplished most cost-effectively by capturing and treating the volume of runoff generated by a relatively frequent storm over the drainage area. This quantity of runoff generated is referred to as the water quality volume for storm water practices.

Pollutants are removed through a variety of mechanisms, but no single management practice is capable of using all the mechanisms. As a result, treatment trains should be considered when more than one kind of pollutant is of concern.

Table 4. Comparative quantity control benefits provided by water qualtity control practices.

Practice	Peak Discharge Control 2-yr storm	Peak Discharge Control 10-yr storm	Peak Discharge Control 100-yr storm	Volume control	Groundwater recharge/low flow maintenance	Streambank erosion control
Oil-grit separators	0	0	0	0	0	0
Wet pond	++	++	++	0	0	++
Artificial wetland	++	++	++	+	+	++
Vegetated swale/filter strip/urban forestry	+	0	0	+	+	0
Full infiltration basin	++	+	0	++	++	++
Combined infiltration detention basin	++	++	++	++	++	++
Off-line infiltration basin	0	0	0	++	++	++
Full infiltration trench/ porous pavement	++	+	0	++	++	++

++ Usually provided + Sometimes provided with careful design. 0 Seldom or never provided.

Source: Horner, 1994

Table 5. Summary of pollutant removal mechanisms

Mechanism	Pollutants affected	Promoted by
Physical sedimentation	Solids, BOD, pathogens, particulate COD, P, N., metals, synthetic organics	Low turbulence
Filtration	Same as sedimentation	Fine, dense herbaceous plants; constructed filters
Soil incorporation	All	Medium-fine texture
Chemical precipitation	Dissolved P, metals	High alkalinity
Adsorption	Dissolved P, metals, synthetic organics	High soil Al, Fe, high soil organics (Met.); circumneutral pH
Ion exchange	Dissolved metals	High soil cation exchange capacity
Oxidation	COD, petroleum hydreocarbons, synthetic organics	Aerobic conditions
Photolysis	Same as oxidation	High light
Volatilization	Volatile petroleum hydrocarbons and synthetic organics	High temperature and air movement
Biological degradation	BOD, COD, petroleum hydrocarbons, synthetic organics	High plant surface area and soil organics
Plant uptake and metabolism	P, N, metals	High plant activity and surface area
Natural die-off	Pathogens	Plant excretions
Nitrification	NH_3-N	Dissolved oxygen >2 mg/L, low toxicants, temperature >5–7°C, circumneutral pH
Denitrification	$NO_3 + NO_2$-N	Anaerobic, low toxicants, temperature >15°C

Source: Horner, 1994

Table 5 is a summary of pollutant removal mechanisms and the pollutants they control. Table 6 presents a summary of the individual management practices and their pollutant removal mechanisms. Table 7 highlights the potential pollutant removal effectiveness of the treatment. For design purposes, the pollutant removal goal is 80% reduction of total suspended solids on an average annual basis.

Particulate reduction will result in control of not only the suspended solids, but pollutants such as phosphorus, particulate COD and BOD, some metals such as lead, copper and zinc, some pathogens and synthetic organic compounds. The percent removal differs with the removal mechanism employed. Many contaminants are attached to the particulate fraction and will be removed with the solids.

Table 6. Individual practices and pollutant removal mechanisms

Practice	Removal mechanisms
Wet ponds	Physical sedimentation, absorption, adsorption, volatilization, (biological removal could occur if algae is removed)
Artificial wetlands/ shallow marshes	Physical sedimentation, filtration, biodegradation, chemical precipitation, plant uptake, volatilization, nitrification, (biological removal may depend on whether plants are harvested)
Infiltration basins	Adsorption, filtration, biodegradation, plant uptake, ion exchange, soil incorporation, physical sedimentation (this should occur in the pre-treatment unit)
Infiltration trenches	Filtration, biodegradation, plant uptake, ion exchange, soil incorporation, sedimentation (same as basins), nitrification/denitrification (may be possible if engineered with anaerobic zone)
Filter strips	Filtration, physical sedimentation, soil incorporation, ion exchange, adsorption, plant uptake
Vegetated swales	Physical sedimentation, filtration, soil incorporation, adsorption, ion exchange, plant uptake
Filters	Filtration (if grass is on top then include mechanisms used by filter strips), if media include an organic layer then add adsorption, biodegradation, absorption, nitrification, chemical interaction
Oil/grit separators	Physical sedimentation

Source: Summary table; see individual sections of this manual for sources of information

Environmental amenities

Aside from the water quantity and quality control provided by individual management practices there may be environmental or human benefits to consider. Potential auxiliary benefits of individual treatment practices are suggested in table 8.

Sometimes these benefits are a natural part of the management practice and in other cases they can be easily included. In a few cases the amenity is so desirable that it weighs heavily in the decision-making process, requiring significant planning and design effort. However, at no time should the incorporation of an amenity compromise the management practice's ability to perform its water quality or quantity control functions, nor should the potential amenities of a practice (such as portraying a detention pond as a lake to nearby residents) be overstated.

Aquatic habitat

Wet ponds and artificial wetlands are particularly good for attracting waterfowl, marsh birds and other aquatic wildlife. Landscaping that uses appropriate plantings along with open water areas will make the management practice attractive to wildlife. However, too many ducks and geese at a wet pond or artificial wetland site can increase the biological oxygen demand beyond the original design capacity.

Wildlife habitat

Buffer strips around larger management practices, as well as the vegetative practices such as swales and filter strips can become home to a variety of wildlife if planted and managed appropriately. Mowings timed to avoid the nesting season will help protect the wildlife attracted to these areas. Diverse plant species and use of native grasses will encourage wildlife diversity. Volunteer plants less attractive to wildlife may dominate despite an extensive effort to control them.

Landscaping and aesthetics

Management practices should not detract from the aesthetic qualities of a neighborhood. Using existing land contours, retaining natural vegetation and designing for a more natural look will allow ponds to enhance the urban landscape. Efforts to conceal outlet control

Table 7. Treatment practices' effectiveness in removing potential pollutants

Practice	Suspended solids	Oxygen demand	Total lead	Total zinc	Total phosphorus	Total nitrogen	Bacteria
Oil-grit separators	0	-	-	-	-	-	-
Wet pond	++	+*	++	+	+*	0*	-
Artifical wetland	++	++*	++	++	++*	++*	-
Vegetated swale	++	0	++	+	0	0	-
6-meter wide turf filter strip	0	0	0	0	0	0	-
30-meter wide forested filter strip	++	++	++	++	+	+	-
Infiltration practices	0	++	++	++	++	+	++

++ High potential for removal. + Moderate potential for removal. 0 Low potential for removal. - Insufficent knowledge

* May be subject to exports of nutrient-enriched and deoxygenated water

Source: Horner, 1994

Table 8. Potential auxiliary benefits of treatment practices

Practice	Aquatic habitat creation	Wildlife habitat creation	No temperature increase	Landscape enhance & aesthetic	Recreational benefits	Public safety	Community acceptance
Oil-grit separators	0	0	++	0	0	++	++
Wet pond	++	++	0	++	++	+	++
Artifical wetland	++	++	0	+	+	+	+
Vegetated swale	+	+	+	+	0	++	++
Vegetated filter strip	0	++	++	+	+	++	++
Infiltration basin	0	++	++	+	+	++	+
Infiltration trench	0	0	++	0	0	++	++
Porous pavement	0	0	++	0	0	++	++

++ Usually provided + Sometimes provided with design modifications. 0 Seldom provided.

Source: Horner, 1994

structures with vegetation or contouring the embankment to hide the outfall riser pipe will enhance the appearance of ponds. Regular maintenance, especially removal of debris, is critical to maintaining the appearance of the management practices.

Recreational benefits

Storm water management practices generally do not provide recreational activities such as boating, fishing or swimming because of health hazards from accumulated pollutants or the potential for damaging the practice. If properly landscaped, however, biking or jogging trails, birdwatching or relaxation areas can be incorporated into the site plan for the practice. These amenities work well for ponds and artificial wetlands. Unfortunately, the vegetative monoculture of infiltration devices does not easily lend itself to wildlife habitat or even natural beauty for sight-seeing. Landscaping added to improve recreational benefits should not interfere with the normal operation of the device or access for maintenance.

Public safety

Management practices most likely to pose a public safety concern include ponds and artificial wetlands because of the risk of accidental drowning. These management practices should be built with safety shelves and gentle slopes. Infiltration devices are not normally a public safety problem.

Community acceptance

The best way to encourage community acceptance of a management practice is to maintain it so that it does not become a nuisance. Many times the perception that a management practice produces odors, breeds mosquitoes or generates weeds discourages installation in a new location. Proper design, operation and maintenance of practices, however, will help overcome these perceptions. Some management practices simply cannot be made visually pleasing no matter how well designed. In these cases a buffer of trees and/or shrubs may improve public acceptance.

Pretreatment

Some of the management practices described in this manual are most appropriate as pretreatment devices, while others require pretreatment to operate effectively. Still others can serve either as primary treatments or pretreatment devices. Table 9 indicates which management practices can be used for pretreatment, which are primary treatment devices and which can be used for source area controls in the upland watershed.

A primary treatment device is able to provide all required treatment in a single unit. If a management practice is used for pretreatment, its design is modified to reflect the reduced performance requirement. Some management practices are sensitive to rainfall intensity and cannot handle the variability of storm water flows without compromising their treatment capability or even structural integrity. As a result, these management practices may need to be constructed off-line from the primary flow.

Other management practices can accommodate only limited flows and are best located at the site of the pollution where they can provide source control. Source controls can also be considered for retrofit conditions in developed areas where space is limited. Highly urbanized areas require creative solutions and innovative thinking. Pollution prevention measures such as street sweeping, catch basin cleaning and leaf pickup should remain a major component of any storm water management program in an urban area.

Table 9. Suitability of practices for pretreatment, primary treatment or source control

Practice	Provides pretreatment	Requires pretreatment	Source control	Primary treatment
Oil/grit separators	X		X	
Wet ponds	X			X
Artificial wetlands		X		X
Vegetated swales	X		X	
Filter strips	X		X	
Infiltration basins		X		X
Infiltration trenches		X	X	X
Sand filters		X	X	
Street sweeping			X	
Porous pavement		X	X	

Land use considerations

The management practices discussed in this manual have unique applications and limitations. Certain land uses restrict the range of management practices a municipality can consider. This is especially true when land is limited by existing development. Detention ponds, infiltration basins and artificial wetlands require major land commitments and may not be appropriate for densely developed areas. If the drainage area includes parks or green space these areas are potential sites for large practices. Residential areas may lend themselves to larger management practices and infiltration devices.

Pollution "hot spots" such as service stations and maintenance shops, downtown areas and industrial parking lots need oil/grit separators, filter strips or sand filters. These can often be located under existing structures provided access for maintenance is incorporated in the design.

Commercial strip areas also may need source controls, unless land is available for larger management practices. The pollutant loads from these areas are higher on a per-acre basis than a residential area. Regional treatment is a consideration when land is available in some areas but not in others. Planners should first consider pollution prevention to minimize the land needed for treatment facilities. This is especially critical if land is limited.

Treatment trains

A final consideration is the development of treatment trains. A treatment train is a group of management practices that handle storm water flows in series, each providing its unique pollution control capability. A treatment train may not result in additional sediment removal but rather a modified sediment removal rate based on the particle size distribution received by each unit. For example, while a wet pond may be capable of removing 75% of the sediment load, if it follows a sedimentation chamber it may only remove 40% of the incoming sediment load since the chamber has already captured the larger particles.

An infiltration device will continue to reduce the pollutant loads by 100% if those flows do not reach surface water, but the loads coming to the device will be modified by any device located upstream. The advantage of treatment trains comes from each management practice's ability to remove certain pollutants more effectively than others, thus providing better removal of a variety of pollutants.

Summary

The guidelines given here should help in the initial selection of appropriate management practices. Additional data collection may be necessary to finalize the best alternative. There may also be political, social, physical or regulatory considerations not covered here that should be explored when choosing the site and practice. The planner or design engineer needs to explore all factors when selecting a practice

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The Wisconsin Storm Water Manual: Overview and Screening Criteria (G3691-1)

G3691-2

The Wisconsin Storm Water MANUAL

Numerous methods are used to predict runoff peaks and volumes and routing flows (See for example Pitt, 1989; USDA-SCS, 1986; Walker, 1990). Two commonly used methods, Small Storm Hydrology and TR 55, are described in this manual for illustrative purposes. Check with local or state codes to determine if specific methods and storm events have been prescribed for your location.

Hydrology

Hydrology is the study of the movement of water through the environment. We refer to water's movement from the atmosphere through the earth and back to the atmosphere as the *hydrologic cycle*. In this manual, we restrict the discussion of hydrology to those portions of the cycle that affect stormwater management and water quality—precipitation, runoff and infiltration. For an overview of storm water hydrology and how urbanization affects surface water runoff, refer to the *Wisconsin Storm Water Manual*, Part 1 (Prey, 1994).

Water quality criteria

Certain hydrologic methods for sizing storm water management practices (for example, grassed swales, infiltration structures and wet detention basins) accomplish two goals:

1. To remove a pollutant to a desired performance standard. For example, a widely accepted standard is to remove a minimum of 80% of the total suspended solids generated from the tributary drainage area on an average annual basis. A Wisconsin Department of Natural Resources (DNR) study has established that this level of performance can be achieved in urban areas by designing for removal of the 5-micron particle from runoff from a 1.5-inch, 4-hour rainfall event. (WI-DNR, 1997)
2. To control runoff peak or volume to a desired design level for water quality management. While management practices aimed primarily at water quality management may also have flood control objectives, this manual does not address flow control for storms greater than the 2-year rainfall and, therefore, does not address practices associated with flood control. Contact the U. S. Department of Agriculture-Natural Resources Conservation Service (NRCS), the Army Corps of Engineers or seek other sources for guidance on flood control concerns.

Design process overview

To size water quality management practices, the Small Storm Hydrology method has been used to predict the runoff from a 1.5-inch, 4-hour rainfall. This method, developed by Pitt and explained in *Small Storm Hydrology: The Integration of Flow with Water Quality Management Practices* (Pitt, 1989), is used to predict runoff from small storms and areas with short concentration times.

To determine the management practice storage volume and peak discharge from the 2-year, 24-hour storm, the method described in *Urban Hydrology for Small Watersheds, Technical Release 55* (USDA-SCS, 1986) is used.

The water quality design process uses the drainage area's hydrologic characteristics to determine the design runoff. After assessing the site data and watershed characteristics, the management practice is designed. A brief overview of the recommended design procedure is given below to assist the reader in understanding the overall process. Not all steps may be necessary for every design. A more detailed description, along with an example, is presented later in this section of the manual.

The steps in the design process for water quality management practices are as follows.

Step 1. Before design begins, the designer should consult with local officials, regional planning agencies and the DNR regional office to determine zoning restrictions, watershed requirements and/or surface and groundwater requirements that may apply to the development site or watershed. Local ordinances and state codes should also be checked to determine if design storm designations and prediction methods have been prescribed.

Step 2. Determine the viability of various water quality practices by collecting and analyzing watershed data and site characteristics using screening criteria such as those contained in the introductory section of this manual.

Step 3. Calculate the expected runoff volume from the developed drainage area for the design storms. Two design storms are considered here. A 1.5-inch, 4-hour storm is used for water quality control and the 2-year, 24-hour storm is used for flow control at bank full conditions.

Step 4. Create hydrographs for the drainage area for the water quality control design rainfall and the bank full conditions. These hydrographs become the inflow hydrograph for design. If the structure is to be used for flood control, the designer may also want to develop hydrographs for additional storms at this time. These hydrographs may be required for flow routing purposes later in the design process.

Step 5. From the information gathered from steps 1–4 above, select the water quality management practice(s) that is/are best suited to the drainage area and the management goals.

Step 6. Develop a preliminary management practice design for the design storm.

Step 7. For those practices that require routing, route the design hydrograph through the structure as a check of the water quality discharge and runoff storage requirements. If the water quality criteria are not satisfied, or the structure is excessively large, repeat step 6 and redesign the structure to correct design flaws. Recheck the design and repeat this loop as required.

Step 8. If not previously determined as a part of step 4, determine the peak discharge for the drainage area in its existing condition and develop the 2-year, 24-hour hydrograph using the tabular method from *TR-55 Urban Hydrology for Small Watersheds* (USDA-SCS, 1986).

Step 9. Develop a preliminary design that limits the post-developed peak discharge to the pre-developed peak discharge for the 2-year, 24-hour rain event or to the peak as prescribed in local ordinances incorporating management practice design for water quality control.

Step 10. Route the 2-year, 24-hour rainfall hydrograph from the developed area through the management practice to check that the practice meets the peak flow and runoff storage requirements. If the peak discharge from the management practice exceeds the pre-developed peak discharge, or if the structure is excessively large, return to step 8 and redesign the structure to correct the design flaws. Repeat steps 8, 9 and 10 as required.

Step 11. Design and route other control discharges as needed. Some management practices, such as infiltration basins and trenches, may not handle flows above the 2-year, 24-hour storm. In such cases, larger flows will need to be diverted around the practice.

Step 12. Assess the effects on the watershed and stream for flows expected after the installation of the management practice.

Step 13. Design details of the management practice, including safety, maintenance and operational features.

Step 14. Develop plans and specifications for the design and construction of the management practice and a maintenance plan.

Hydrologic principles for the design of water quality management practices

Precipitation is the driving force in the hydrologic cycle. During some rainfall events, all precipitation is either intercepted by vegetation or infiltrated into the soil. In such cases no surface runoff occurs. However, when rainfall or snowmelt exceeds the soil's infiltration rate, excess water begins to accumulate as surface storage in small topographic depressions. As the depth of water in surface detention increases, overland flow may occur. Overland flow quickly concentrates into small rills or channels, which then flow into larger streams.

Rainfall can also infiltrate into the soil and move laterally through upper soil zones until it again appears on the soil surface or enters a stream channel. This shallow lateral flow is known as interflow. A portion of the precipitation may percolate to the water table. Percolation will contribute to stream base flow if the water table intersects the stream channel.

Factors such as antecedent soil moisture, surface cover, variable infiltration rate and seasonal variations make the development of rainfall-runoff relationships difficult. Although a number of methods to calculate runoff from a known rainfall event have been developed, these methods must be used with caution. The advantages, disadvantages and limitations of each method should be known if an appropriate model choice is to be made.

Computation method

The storm water components of concern in this manual are the water quality component and the peak discharge reduction necessary to protect stream banks and stream biota from increased runoff due to urbanization. Runoff volume is usually the most important hydrologic parameter in design of management practices for water quality, while peak flow discharge and time of concentration are the most important hydrologic parameters for flood control. Runoff models for water quality investigations, therefore, may differ from runoff models for flood control.

The storms to be assessed from a water quality concern are high frequency storms of relatively small magnitude. These small storms are responsible for the majority of the pollutant loads generated from urban areas on an annual basis (EPA 1983, Pitt 1989). For "bank full" conditions, peak discharge from the 2-year, 24-hour storm from the drainage area in a fully developed condition should be limited to the peak discharge from a 2-year, 24-hour storm in the pre-developed condition, or to the level specified in local ordinances.

Models other than those described in this manual may be applicable. The designer is encouraged to explore a variety of models to determine those most appropriate for the situation, provided local regulations allow their use. Most models are for single event design storms. Continuous simulation models (for example, see Bicknell et. al., 1997) are alternatives for assessing the pollutant removal efficiency of management practices. These models typically use an annual rainfall event file and analyze pollutant loading and runoff volume from a specified rain file on a given land-use. Pollutant loads are then summed on a mass basis and a theoretical removal rate calculated. The complexity of these simulation models requires that a computer be used in the assessment.

Water quality—small storm hydrology

To simplify the management practice design process for water quality control, a single event approach will be discussed in this document. For purposes of illustration, the runoff volume produced by the 1.5-inch storm is used. By treating all runoff from storms up to and including the 1.5-inch design storm size, a storm water management practice should achieve approximately 80% removal of the annual total suspended solids loading delivered to that practice. By using the 1.5-inch storm, the designer can estimate the design runoff volume and the required storage volume to size controls. Water quality peak flow should not be confused with the 2-year, 24-hour peak used for discharge rate control for bank full flow control.

Some management practice situations require the use of a flow splitting device. For example, a flow splitter might be required for an off-line management practice when retrofitting practices in an existing developed area. The hydraulic design flow for an off-line water quality management practice would be determined using the 1.5-inch design storm and a 4-hour duration in concert with a triangular hydrograph method to calculate water quality peak flow. The splitter would then be used to bypass the remaining flow.

Water quality runoff volume

To estimate storm runoff volumes, peak flows and hydrographs for small storms, the Small Storm Hydrology method devised by Pitt (1989) is used here. This method uses volumetric runoff coefficients to calculate runoff from urban land-uses for small rainfalls.

The method is particularly useful in describing the contributions of individual source areas to the total runoff or the effectiveness of individual source area controls. Land development characteristics (landscaping, streets, drainage system type, etc.) are usually critical when determining small-storm flows and the variable urban source areas contributing pollutants.

The volumetric runoff coefficients, R_v, in small storm hydrology are calibrated to account for various rainfall depths. Small rains tend to have small volumetric runoff coefficients that increase as the rainfall depths increase. Pervious areas are less responsive to rainfall depths than mostly impervious areas. The approach used in calculating the water quality volume is to establish the R_v values and runoff volumes from the various source areas for the design rainfall. Source areas considered are consistent with those used in the Source Loading and Management Model (SLAMM) computer model (Pitt, 1994). Six different land use types defined in table 1.

Calculation of the design runoff volume from a watershed uses the area, the weighted volumetric runoff coefficients for various source areas (table 2) for the 1.5-inch storm, the individual land use source areas, and a conversion factor to convert acre-inches into acre-feet. If the design rainfall is different than 1.5 inches, the runoff coefficients must be adjusted as in Pitt (1989).

Land use runoff volume (cu.ft.) =
$(1.5\text{ in})(1\text{ ft}/12\text{ in})(R_v)(43560\text{ sq.ft/ac})(\text{Area acres})$

Table 1. Land use and pollutant source area definitions

Residential land uses

- High Density Residential without Alleys (HRNA): Urban single family housing at a density of greater than 6 units/acre. Includes house, driveway, yard and streets.
- High Density Residential with Alleys (HRWA): Same as HRNA except alleys exist behind the houses where the back yards join.
- Medium Density without Alleys (MRNA): Same as HRNA except the density is between 2-6 units/acre.
- Medium Density with Alleys (MRNA): Same as HRWA except alleys exist behind the houses where the back yards join.
- Low Density (LR) : Same as HRNA except the density is 0.7 to 2 units/acre.
- Multiple Family (MF): Housing for three or more family units from 1-3 stories in height. Units may be adjoined up-and-down, side-by-side or front-and-rear. Includes building, yard, parking lot and driveways.
- High Rise (HIR): Same as MF except buildings are apartments 4 or more stories in height.
- Trailer Parks (MOBR): For a mobile home or trailer park, includes all vehicle homes, the yard, driveway and office area.
- Suburban (SUBR): Same as HRNA except the density is between 0.2 and 0.6 units/acre.

Commercial land uses

- Strip Commercial (CST): Those buildings for which the primary function involves the sale of goods or services. This category includes some institutional lands found in commercial strips, such as post offices, court houses and fire and police stations. This category does not include buildings used for the manufacture of goods or warehouses. This land use includes the buildings, parking lots and streets. It does not include nursery, tree farms or lumberyards.
- Shopping Centers (SC): Commercial areas where the related parking lot is at least 2.5 times the area of the buildings' roof area. The buildings in this land use are usually surrounded by the parking area. This land use includes the buildings, parking lot and the streets.
- Office Parks (OP): Land use where non-retail business takes place. The buildings are usually multi-story, surrounded by larger areas of lawn and other landscaping. This land use includes the buildings, lawn and road areas. Establishments that may be in this category include: insurance offices, government buildings and company headquarters.
- Downtown Commercial (CDT): Highly impervious downtown areas of commercial and institutional land use.

Industrial land uses

- Manufacturing (MI): Those buildings and premises devoted to the manufacture of products. This category includes utility power plants.
- Non-Manufacturing (LI): Those buildings used for the storage and/or distribution of goods awaiting further processing or sale to retailers. This category includes warehouses and wholesalers. This category also includes businesses such as lumberyards, auto salvage yards, junk yards, oil tank farms, coal and salt storage areas, grain elevators, agricultural coops and areas for bulk storage of fertilizers and pesticides.

Institutional land uses

- Hospitals (HOSP): Medical facilities that provide inpatient overnight care. Includes nursing homes, state, county or private facilities. Includes buildings, grounds, parking lots and drives.
- Education (SCH): Includes any public or private primary, secondary, or college educational institutional grounds. Includes buildings, playgrounds, athletic fields, roads, parking lots and lawn areas.
- Miscellaneous Institutional (MISC): Churches and large areas of institutional property not part of CST and CDT.

Open space land uses

- Cemeteries (CEM): Includes cemetery grounds, roads, and buildings located on the grounds.
- Parks (PARK): Outdoor recreational areas including municipal playgrounds, botanical gardens, arboretums, golf courses and natural areas.
- Undeveloped (OSUD) : Lands that are private or publicly owned with no structures and have a complete vegetative cover. This includes vacant lots, transformer stations, radio and TV transmission areas, water towers and railroad rights-of-way.

Freeway land uses

- Freeways (FREE): Limited access highways and the interchange areas.

Table 2: Runoff coefficients (R_v) for urban source areas assuming a 1.5-inch rain depth*

Source area	R_v
Roof areas	
Flat, connected **	.88
Flat, disconnected, AB soil	.04
Flat, disconnected, CD soil, low density	.23
Flat, disconnected, CD soil, med/high density, no alleys	.21
Flat, disconnected, CD soil, med/high density, alleys**	.86
Pitched, disconnected, AB soil	.04
Pitched, disconnected, CD soil, low density	.23
Pitched, disconnected, CD soil, med/high density, no alleys	.23
Pitched, disconnected, CD soil, med/high density, alleys	.96
Pitched, connected	.98
Parking & storage areas	
Paved, connected **	.94
Paved, disconnected, AB Soil	.04
Paved, disconnected, CD soil, low density	.23
Paved, disconnected, CD soil, med/high density, no alleys	.22
Paved, disconnected, CD soil, med/high density, alleys unpaved, connected **	.85
Unpaved, disconnected, AB soil	.04
Unpaved, disconnected, CD soil, low density	.23
Unpaved, disconnected, CD soil, med/high density, no alleys	.20
Unpaved, disconnected, CD soil, med/high density, alleys	.84
Playground areas	
Connected **	.94
Disconnected, AB soil	.04
Disconnected, CD soil, low density	.23
Disconnected, CD soil, med/high density, no alleys	.22
Disconnected, CD soil, med/high density, alleys	.93
Driveway areas	
Connected **	.94
Disconnected, AB soil	.04
Disconnected, CD soil, low density	.23
Disconnected, CD soil, med/high density, no alleys	.22
Disconnected, CD soil, med/high density, alleys	.93
Sidewalk areas	
Connected **	.94
Disconnected, AB soil	.04
Disconnected, CD soil, low density	.23
Disconnected, CD soil, med/high density, no alleys	.22
Disconnected, CD soil, med/high density, alleys	.93

* *These R_v values will change as a function of rain depth, and should only be used to calculate the volume of runoff from the 1.5 inch-rain depth.*

** *For commercial strip and shopping center areas, use these Rv coefficients if the source area is disconnected and draining into a soil in hydrologic group C or D.*

Source area	R_v
Street and alley areas	
Smooth texture **	.84
Intermediate texture **	.79
Rough texture **	.79
Very rough texture**	.79
Landscaped areas	
Large area, AB soil	.04
Large area, CD soil	.23
Small area, AB soil	.04
Small area, CD soil	.23
Undeveloped areas	
Undeveloped area, AB soil	.04
Undeveloped area, CD soil	.23
Other areas	
Directly connected **	.94
Pervious, AB soil	.04
Pervious, CD soil	.23
Partially connected, AB soil	.04
Partially connected, CD soil, low density	.23
Partially connected, CD soil, med/high density, no alleys	.22
Partially connected, CD soil, med/high density, alleys	.93
Freeway areas	
Paved land & shoulder, smooth	.84
Paved land & shoulder, intermediate	.78
Paved land & shoulder, rough	.78
Paved land & shoulder, very rough	.78
Large turf area, AB soil	.04
Large turf area, CD soil	.23
Undeveloped area, AB soil	.04
Undeveloped area, CD soil	.23
Other directly connected areas **	.94
Partially connected, AB soil	.04
Partially connected, CD soil, low density	.23
Partially connected, CD soil, med/high density, no alleys	.22
Partially connected, CD soil, med/high density, alleys	.93

* *These R_v values will change as a function of rain depth, and should only be used to calculate the volume of runoff from the 1.5 inch-rain depth.*

** *For commercial strip and shopping center areas, use these Rv coefficients if the source area is disconnected and draining into a soil in hydrologic group C or D.*

Water quality runoff hydrograph

To establish the peak flow from the 1.5-inch, 4-hour storm used in this manual, a triangular hydrograph method is used. This hydrograph method uses a regression equation developed by Pitt (1994) to establish runoff duration. The equation:

Runoff duration = 0.9 hours + (0.98)(rainfall duration)

Since our design storm duration is four hours, the runoff duration is equal to:

Runoff duration = 0.9 hours + 0.98 (4 hours) = 4.8 hours

The average runoff flow rate is equal to the runoff volume divided by the runoff duration. The design peak flow rate is assumed equal to twice the average runoff flow rate. With the runoff duration and the runoff peak flow, the triangular hydrograph can be created (figure 1). The runoff hydrograph will become the inflow hydrograph for use in sizing management practices and splitting devices.

2-year peak flow control

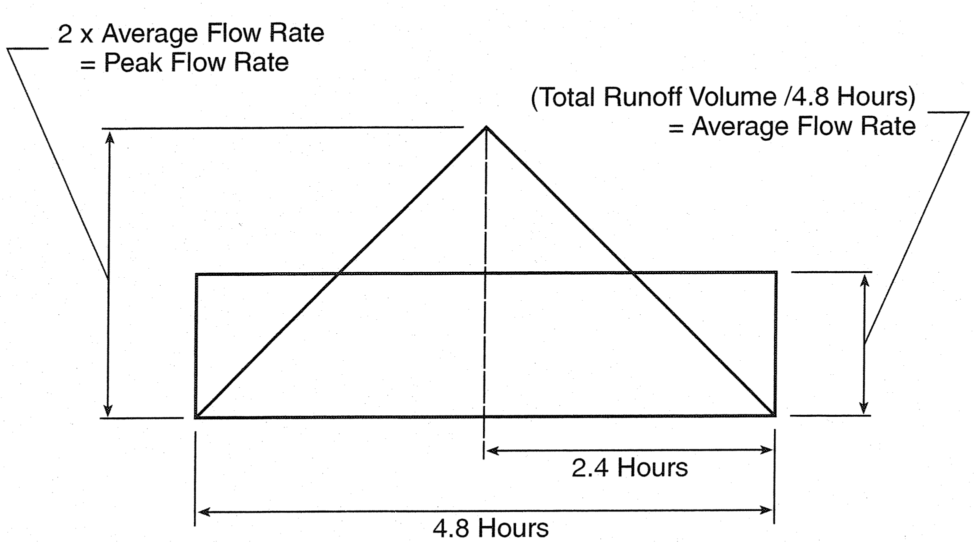
When an area is urbanized, the amount of impervious surface in the drainage area is usually significantly increased. Storm sewers are installed to quickly convey runoff from developed sites, and landscaping and surface grading often compacts the soil reducing infiltration and removing natural surface depressions that provide small storage areas for runoff. These combined changes have a number of detrimental effects on receiving streams:

- Incresed runoff peak discharges
- Increased runoff volumes
- Increased flow velocity during storms
- Decreased time of concentration
- Increased frequency and severity of flooding
- Reduced base flow between storms
- Increased stream bank erosion
- Increased water turbidity due to bank erosion and increased transport capacity
- Increased down stream sediment deposition
- Reduced diversity and abundance of aquatic species

By restricting peak discharges of the post-developed site to the peak discharges that existed before development, or to specified levels, damage to downstream areas can be greatly reduced. Detention basins are an excellent practice that can be used to diminish the destructive effects listed above. By designing detention basins to restrict flows by temporarily storing the increased runoff produced by urbanization, downstream flow quantities and velocities can be more closely controlled.

Stream channel characteristics are largely determined by smaller rainfall events (Leopold, 1968; Wolman and Schick, 1967). Generally, storm events in Wisconsin between the 1-year and 2-year return periods cause what is called bank full flow condition. This flow quantity controls and forms the natural stream channel. By restricting peak flows from these more common rainfall events, damaging effects to the channel from increased runoff produced by urbanization can be greatly reduced. While debate continues over the appropriate return period for these conditions, research to date indicates that the 2-year, 24-hour storm event will cover the wide range of stream flow characteristics and, when used with the water quality design guidelines, will help protect streams from the negative impact of urbanization.

Figure 1. Runoff hydrograph for the 1.5-inch, 4-hour rainfall.



Peak flow limitations warrant concern when streams or water bodies are affected negatively by increased flows. Water bodies that experience only negligible effects from increased flow may not have to conform to these guidelines. For example, increased flows from small tributaries to a large lake such as Lake Michigan would have little or no effect on the lake's water quality.

As mentioned earlier, the design method used to size management practices to limit the 2-year, 24-hour peak flow from developing areas to the pre-developed peak flow is described in the NRCS TR-55 manual, *Urban Hydrology for Small Watersheds* (USDA-SCS, 1986). Because flow routing is used to properly size stormwater management practices, the tabular hydrograph method described in *Urban Hydrology for Small Watersheds* is illustrated in this manual. Because numerous documents are readily available and the calculation procedure is commonly known, detailed coverage is not given here. However, an example demonstrating the use of this procedure is given below. Contact the NRCS to obtain a copy of the TR-55 manual.

Structural storage volume—flow routing

The design active storage volume is the volume in the reservoir available to accommodate the runoff from the design storms. The storage volume required is the difference between the inflow and outflow hydrographs as illustrated in figure 2. To determine the storage volume needed, flow routing procedures should be employed.

A management practice design is subjected to the expected inflow hydrograph, and the storage and discharge are analyzed to determine if the storage required exceeds the available storage volume and if the outlet is properly sized. For water quality considerations, this method is used to develop, test and modify the design to remove 80% of the total suspended solids in storm water runoff on an annual average basis.

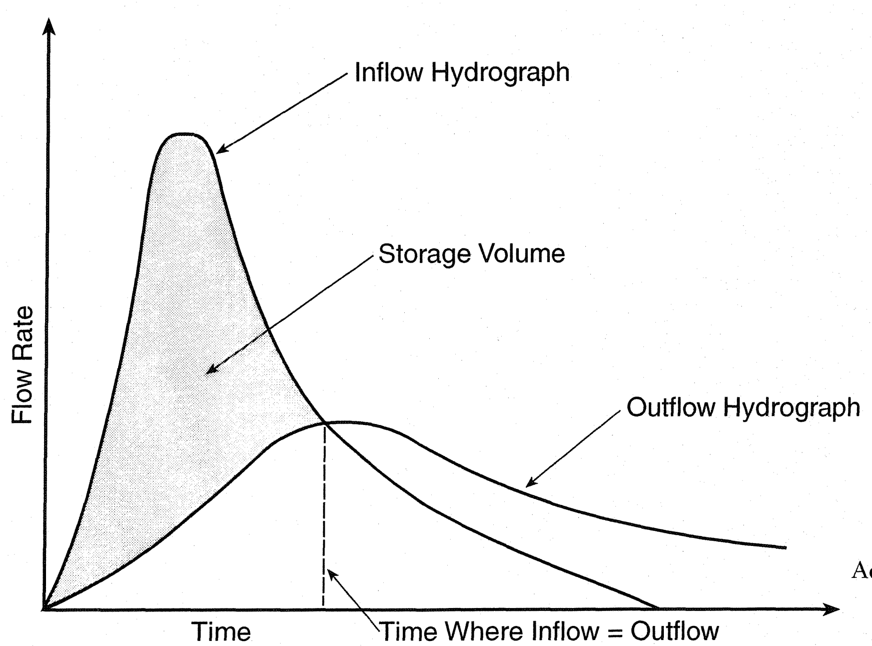
Wisconsin DNR studies indicate that removal of the 5-micron particle from runoff from a 1.5-inch, 4-hour storm will achieve 80% removal on an average annual basis. Flow routing is also used to develop, test and modify the basin storage volume and outlet design for the active storage for larger storms for bank full or peak control. The flow routing method used in this manual is based on the NRCS Technical Release-20 (USDA-SCS, 1992) as presented by McCuen (1982). An example of this procedure using a detention basin as a management practice follows.

Water quality management practice design: an example

The remainder of this section of the storm water manual consists of a detailed example of a procedure for designing a water quality management practice.

In this example, note that all runoff values and field data information, while in the range of acceptable values, are assumed. The designer is responsible for collecting relevant field survey data needed to develop the basin design. Failure to collect survey data relevant to the site characteristics will likely result in designs that fail to meet design specifications and could result in a major failure of the basin facility. The approach given below is an example of what could be done, but may not fit every situation. Often design becomes an iterative process where the design evolves as more information is obtained and alterations correct earlier design assumptions.

Figure 2. Flow difference between inflow and outflow hydrographs.



Adapted from Barfield et al. 1983.

To develop accurate hydrologic assessments the designer should, at a minimum, conduct representative surveys of each sub-drainage area. This includes field surveys to determine how impervious areas are connected to the drainage system, the condition of streets, the type and efficiency of the drainage system, and the percent of impervious surfaces in the sub-drainage area.

Steps 1 & 2
Inventory

Assume that the first two steps of the design process have been completed. The steps consist of: 1) collecting data to establish the zoning and watershed requirements; and 2) assessing the viability of a variety of management practice alternatives and selection of the most appropriate based on design objectives and site conditions. During these steps, the designer has accumulated information about the watershed, the development area and potential sites. The types of source area information include topographic maps of the drainage area, storm sewer drainage system maps, aerial photographs, soils information, and existing and proposed land use. The designer should also know what storm water flow restrictions apply to developing areas. With this information, the designer is ready to continue the design process with the hydrologic assessment.

Step 3
Hydrology: Calculation of the drainage area runoff volume from the 1.5-inch rainfall

The designer begins by assessing the hydrologic characteristics of the site both in its existing and proposed developed states. The drainage area should be divided into subareas that have similar characteristics. Land use maps, topographic maps and aerial photographs are very useful in delineating areas with similar source area characteristics. When delineating the subareas, some key items to consider include:

- Variation in land uses, building densities and the percent of impervious surfaces
- Change in street and/or alley patterns that indicate variations in construction practices and code requirements
- Change in topography
- Variation in street widths
- Historical analysis of building codes and zoning and drainage ordinances

Example: A developer wants to develop 100 acres of a 122-acre watershed. From the proposed plans and from information received from city officials, the designer determines that the planned drainage area will have the following land use breakdown:

Pre-developed characteristics are:

Open farm land—100 acres

Grass and meadow vegetation

Hydrologic soil group—C

Curve number—71

General land slope 3.5%

$R_v = 0.23$

From a filed survey of the site, it was determined that after development the drainage area will increase to 126 acres due to the storm sewer system.

Planned post-developed characteristics are:

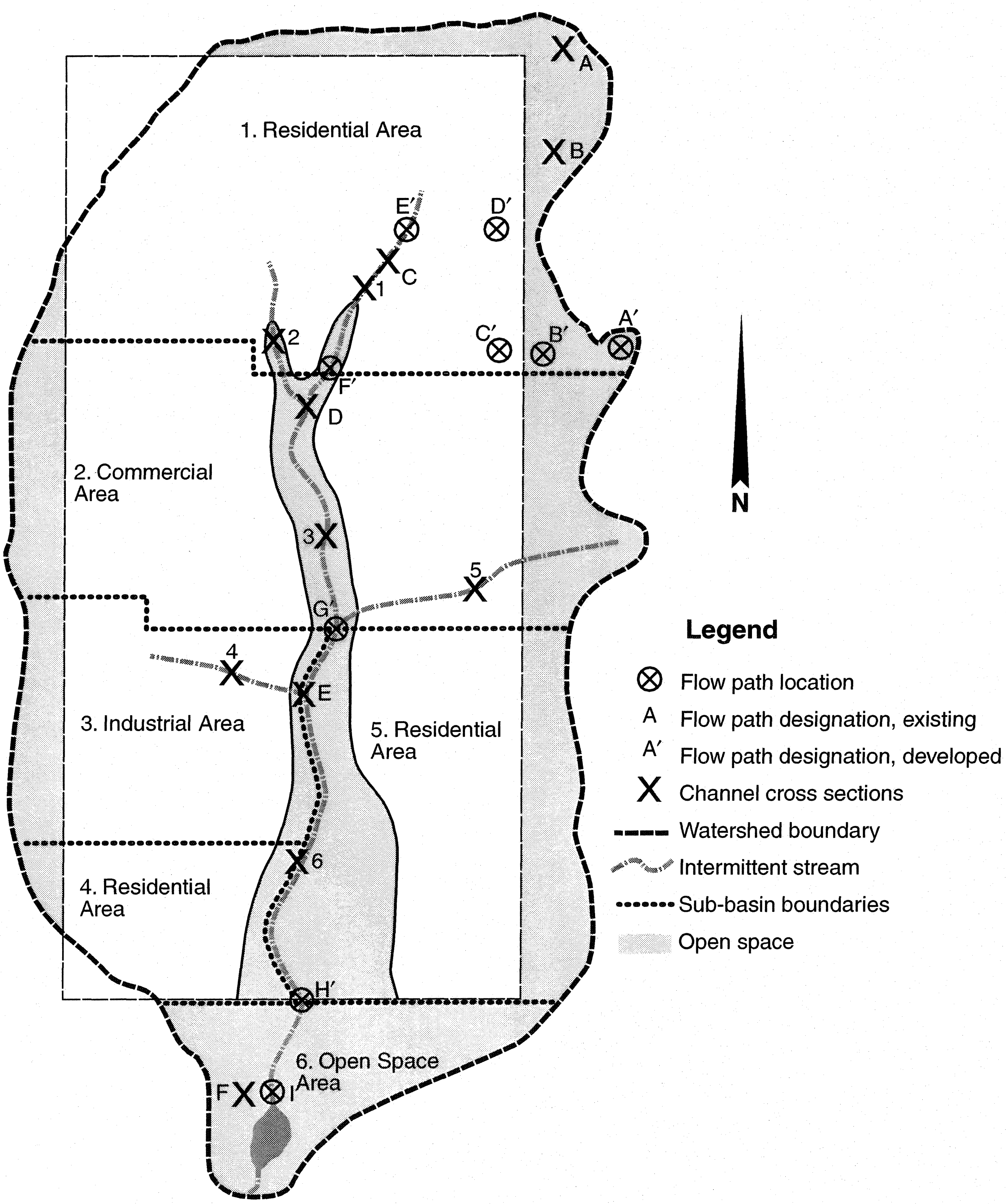
Residential	52 acres
Industrial	13 acres
Commercial	25 acres
Open space	36 acres

Cross-sections and streambed slopes were taken at six locations as shown in figure 3, with the results shown in table 3.

Table 3. Existing channel dimensions.

Channel location	**Channel depth** (ft.)	**Channel width** (ft.)	**Channel slope** (ft./ft.)
1'	0.60	2.20	0.035
2'	0.65	2.25	0.030
3'	0.90	3.60	0.030
4'	0.45	1.30	0.035
5'	0.70	3.20	0.035
6'	0.90	5.95	0.035

Figure 3. Plan view of the watershed post-development.



Using data from soil surveys and field data the designer assigns an NRCS curve number of 71.

With the collected information and knowing the commonly used development practices, the designer divides the developed drainage area into six sub-drainage areas as shown in figure 3. A brief summary of the predominant land use in each sub-drainage area is summarized in table 4.

Area 1. 30.1 acres of low density residential, 4.3 acres of open space—undeveloped on the outside edge, and 1.7 acres of park near the channel. Total area equals 36.1 acres. (Note: Due to the storm sewer system, 2.8 acres are added to the existing drainage area in this sub-watershed area.)

Area 2. 25.0 acres commercial, 6.6 acres of open space—undeveloped on the outside edge, and 1.9 acres of park near the channel. Total area equals 33.5 acres.

Area 3. 13.0 acres industrial, 2.3 acres open space—undeveloped on the outside edge, and 0.8 acres park near the channel. Total area equals 16.1 acres.

Area 4. 8.2 acres low density residential, 0.4 acres open space—undeveloped on the outside edge, and 1.4 acres of park near the channel. Total area equals 10.0 acres (Note: Due to the storm sewer system, 0.8 acres are added to the existing drainage area in this sub-basin area.)

Area 5. 13.7 acres low density residential, 3.3 acres of open space on the outside edge, and 4.2 acres of park near the channel. Total area equals 21.2 acres.

Area 6. 9.1 acres open space—undeveloped.

These sub-drainage areas, grouped according to land uses, have source area characteristics, the corresponding runoff coefficients, and runoff volumes for the 1.5-inch rainfall shown in table 5.

Table 4. Basin drainage areas.

Sub-basin	**Existing undeveloped area** (acres)	**Developed area** (acres)	**Open area on outside edge** (acres)	**Open area near channel** (acres)	**Total area after development** (acres)	**Changes in drainage area** (acres)
res.1	33.3	30.1	4.3	1.7	36.1	+2.8
com.2	33.5	25.0	6.6	1.9	33.5	0.0
ind. 3	16.1	13.0	2.3	0.8	16.1	0.0
res. 4	9.2	8.2	0.4	1.4	10.0	+0.8
res. 5	21.2	13.7	3.3	4.2	21.2	0.0
6	9.1	-	-	9.1	9.1	0.0
Total	122.40	90.00	16.90	10.10	126.0	+3.60

Table 5 : Calculation of runoff volumes for the developed area.

Area 1 — Residential (36.1 acres)

Source area characteristic	**Area in acres**	**Runoff coefficient R_v**	**Product of R_v x acres**	**Runoff vol.= 1.5in.** x **R_v** x (1ft/12in) **43,560 sq.ft./ac.** x **area** (in cu. ft.)
Flat roofs—connected	0.01	0.88	0.009	48
Flat roofs—disconnected	0.08	0.23	0.018	100
Pitched roofs—connected	0.41	0.98	0.402	2,191
Pitched Roofs—disconnected	2.85	0.23	0.656	3,572
Paved parking—connected	0.06	0.94	0.056	305
Driveways—connected	0.57	0.94	0.536	2,919
Driveways—disconnected	0.85	0.23	0.196	1,067
Sidewalks—connected	0.33	0.94	0.310	1,690
Sidewalks—disconnected	0.45	0.23	0.104	566
Street area—smooth	1.20	0.84	1.008	5,489
Street area—surface—intermediate	2.86	0.79	2.259	12,301
Large landscape	8.73	0.23	2.008	10,934
Small landscape	16.74	0.23	3.850	20,965
Other pervious areas	0.96	0.23	0.221	1,202
Total	36.10	——	11.633	63,349

R_v = 11.633/36.10 = 0.32

Area 2—Commercial (33.5 acres)

Source area characteristic	**Area in acres**	**Runoff coefficient R_v**	**Product of R_v x acres**	**Runoff vol.= 1.5 in.** x **R_v** x (1ft/12in) **43,560 sq.ft./ac.** x **area** (in cu. ft.)
Flat roofs—connected	4.14	0.88	3.643	19,837
Flat roofs—disconnected	0.93	0.23	0.214	1,167
Paved parking—connected	7.83	0.94	7.360	40,076
Paved parking—disconnected	3.35	0.23	0.771	4,203
Sidewalks—connected	0.28	0.94	0.263	1,433
Street area—smooth	3.30	0.84	2.772	15,094
Street area—surface—intermediate	4.74	0.79	3.745	20,400
Large landscape	8.50	.23	1.955	10,663
Small landscape	0.43	.23	0.099	539
Total	33.50	——	20.822	113,412

Commercial = 20.82/33.50 = 0.62

Area 3—Industrial (16.1 acres)

Source area characteristic	**Area in acres**	**Runoff coefficient R_v**	**Product of R_v x acres**	**Runoff vol.= 1.5 in.** x R_v x (1ft/12in) **43,560 sq.ft./ac.** x **area** (in cu. ft.)
Flat roofs—connected	1.57	0.88	1.382	7,523
Flat roofs—disconnected	2.65	0.23	0.610	3,324
Paved parking—connected	1.10	0.94	1.034	5,630
Paved parking—disconnected	2.51	0.23	0.577	3,149
Unpaved parking—disconnected	0.56	0.23	0.129	703
Driveways—connected	0.10	0.94	0.094	512
Driveways—disconnected	0.15	0.23	0.035	188
Sidewalks—connected	0.02	0.94	0.019	102
Sidewalks—disconnected	0.04	0.23	0.009	50
Street area—smooth	0.94	0.84	0.790	4,299
Street area—surface—intermediate	1.11	0.79	0.877	4,777
Large landscape	5.02	0.23	1.155	6,298
Small landscape	0.12	0.23	0.028	151
Rail areas (other)	0.21	0.23	0.048	263
Total	16.10	——.—	6.787	36,969

R_v for industrial = 6.79/16.10 = 0.43

Area 4— Residential (10.0 acres)

Source area characteristic	**Area in acres**	**Runoff coefficient R_v**	**Product of R_v x acres**	**Runoff vol.= 1.5 in.** x R_v x (1ft/12in) **43,560 sq.ft./ac.** x **area** (in cu. ft.)
Flat roofs—connected	0.00	0.88	0.000	0
Flat roofs—disconnected	0.02	0.23	0.005	26
Pitched roofs—connected	0.33	0.98	0.323	1,760
Pitched roofs—disconnected	0.62	0.23	0.143	780
Paved parking—connected	0.02	0.94	0.019	105
Driveways—connected	0.16	0.94	0.150	819
Driveways—disconnected	0.22	0.23	0.051	279
Sidewalks—connected	0.11	0.94	0.103	562
Sidewalks—disconnected	0.12	0.23	0.028	152
Street area—smooth	0.33	0.84	0.277	1,507
Street area—surface—intermediate	0.78	0.79	0.616	3,354
Large landscape	2.53	0.23	0.582	3,171
Small landscape	4.56	0.23	1.049	5,711
Other pervious areas	0.20	0.23	0.046	253
Total	10.00	——.—	3.392	18,479

R_v = 3.39/10 = 0.34

Area 5: Residential (21.2 acres)

Source area characteristic	**Area in acres**	**Runoff coefficient R_v**	**Product of R_v** x **acres**	**Runoff vol.= 1.5 in.** x **R_v** x (1ft/12in) **43,560 sq.ft./ac.** x **area** (in cu. ft.)
Flat roofs—connected	0.01	0.88	0.009	48
Flat roofs—disconnected	0.03	0.23	0.007	39
Pitched roofs—connected	0.53	0.98	0.519	2,827
Pitched roofs—disconnected	1.06	0.23	0.244	1,329
Paved parking—connected	0.03	0.94	0.028	152
Driveways—connected	0.15	0.94	0.141	767
Driveways—disconnected	0.49	0.23	0.113	614
Sidewalks—connected	0.13	0.94	0.122	666
Sidewalks—disconnected	0.22	0.23	0.051	279
Street area–smooth	0.58	0.84	0.445	2,422
Street area–surface—intermediate	1.32	0.79	1.043	5,680
Large landscape	8.73	0.23	2.008	10,934
Small landscape	7.64	0.23	1.757	9,566
Other pervious areas	0.34	0.23	0.078	427
Total	21.21	—.—	6.565	35,750

$R_v = 6.56/21.2 = 0.31$

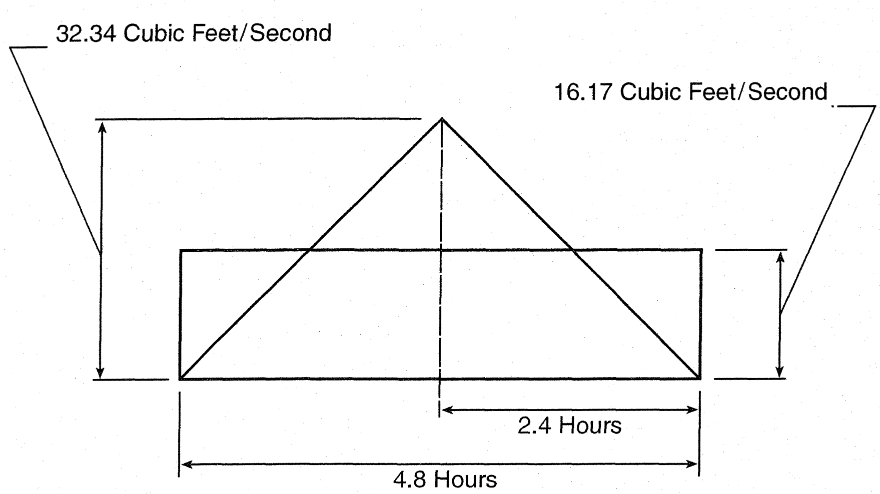
Area 6 is open space undeveloped. The volume of runoff is:

Runoff Vol. = [9.1 acres x (43,560 sq. ft./1 acre)] x [1.5 in. x (1 ft./12in.)] x [0.23] =11,400 cubic feet

Total drainage area runoff volume =

63,350 cu. ft. + 113,410 cu. ft. + 36,970 cu. ft. + 18,480 cu. ft. + 35,750 cu. ft. + 11,400 cu. ft. = 279,360 cu. ft.

Figure 4. Runoff Hydrograph for the 126-acre development (1.5-inch, 4.8-hour rainfall event).



Step 4

Creating the 1.5-inch/hour rainfall hydrograph

Given the calculated runoff volume of 279,360 cubic feet, and the water quality triangular runoff hydrograph procedure described previously, the hydrograph for the 1.5-inch, 4-hour storm can be developed. This runoff hydrograph is used as the inflow hydrograph in the flow routing procedure to obtain a more accurate assessment of the storage needs for management practices.

Average flow rate = 279,360 cubic feet/4.8 hours
= 58,200 cubic feet/hour, or
= 16.17 cubic feet/second

Peak flow rate = (16.17 cubic feet/second) x 2
= 32.34 cubic feet/second

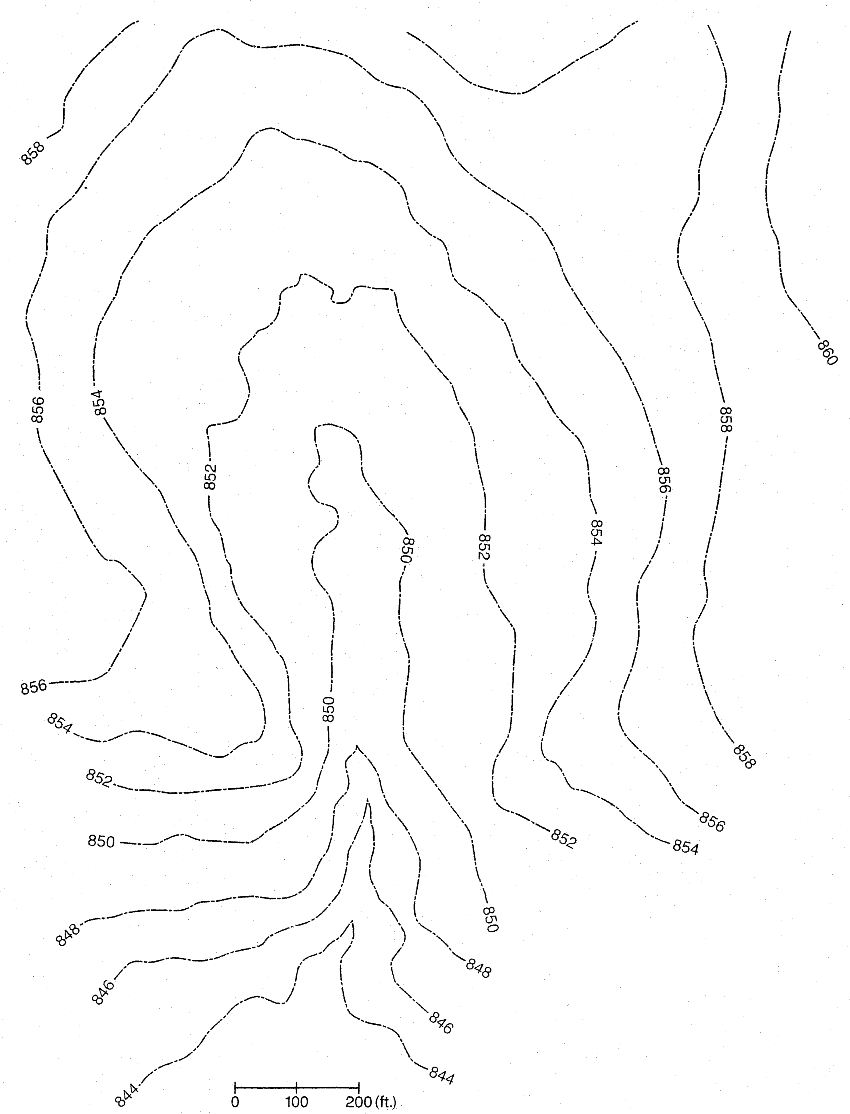
Step 5

Selection of the water quality management practice

Using the information obtained in steps 1 through 4 and information from site and/or watershed maps, the designer is able to determine potential practices and possible site locations.

For this example, assume that preliminary data indicate a site in the southern portion of the drainage area that would serve the entire drainage area and, in the designer's judgement, provide the necessary space to contain the runoff volume. A detailed site investigation shows that the prevalent soil on the site is from Hydrologic Soils Group D, with an infiltration rate of less than 0.03 inches/ hour, making storage feasible. Space and slope limitations prohibit an artificial wetland stormwater management system. From these results and the criteria of the local officials, a detention basin is chosen as the most appropriate water quality practice.

Figure 5. Detention basin site—existing conditions



Step 6

Develop a preliminary size and rough design for the management practice

To create a preliminary design, the designer follows the appropriate design guidelines of the chosen management practice as described in later sections of this manual. In this example the guidance for detention basins is used to determine the preliminary design.

From the drainage area survey, a site on the southern portion of the drainage area is suitable for a detention basin. The site's lowest elevation is the bed of an intermittent stream. The stream is considered a non-navigable stream, and, in its existing condition, has flows only in the spring and after heavy rains. (See figure 5).

According to detention basin design guidelines, two key considerations are the permanent pond volume and the surface area size. The designer now determines the permanent pond surface area.

Calculation of the permanent pond surface area

In most cases the drainage area consists of mixed land uses. The required pond surface area is determined by multiplying the area of the land-use by the recommended drainage area coefficient. An abbreviated table of drainage area coefficients is found in table 6; a more extensive table of coefficients may be found in the wet detention basin section of this manual.

Table 6. Estimated pond surface area as a percent of the tributary drainage area

Land use	% of drainage area
Commercial	1.7
Industrial	2.0
Residential	0.8
Open Space	0.6

The recommended pond size as a percentage of drainage area is calculated as:

Residential
  (52 acres x 0.008) = 0.42 acres
Industrial
  (13 acres x 0.020) = 0.26 acres
Commercial
  (25 acres x 0.017)= 0.42 acres
Open space
  (36.0 acres x 0.006) = 0.22 acres
Total = 1.32 acres of surface area for the permanent pond or
  = 57,500 square feet

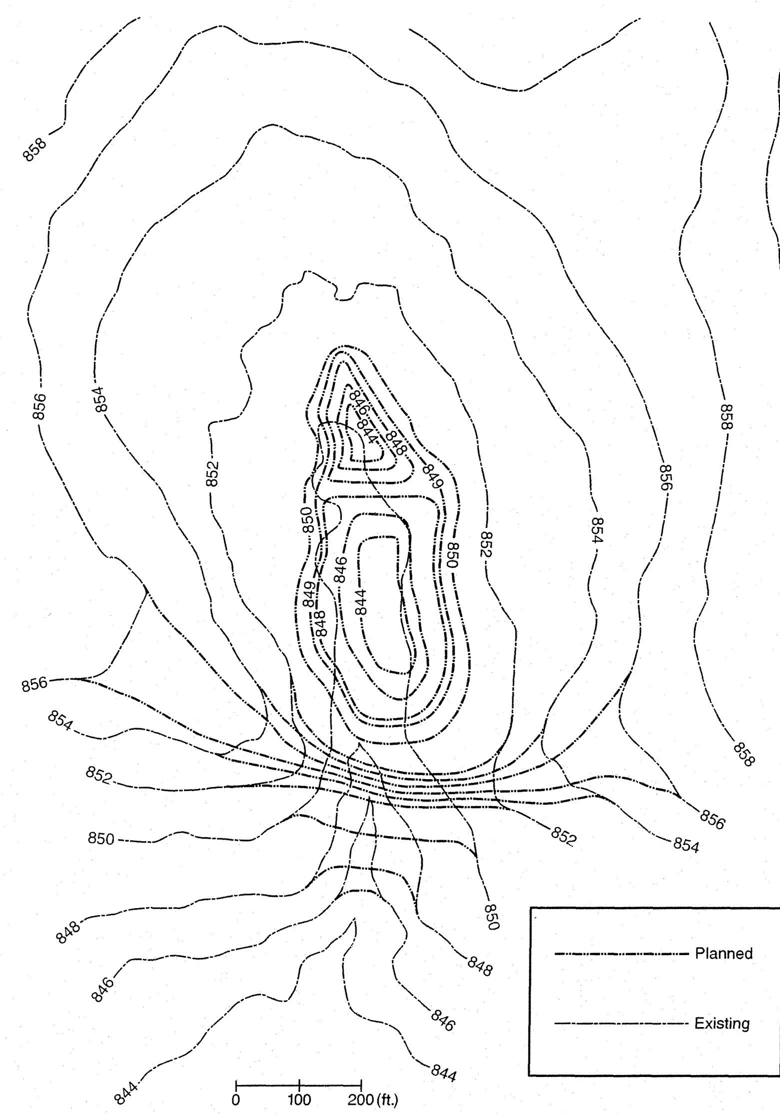
The excavation must be sufficient to achieve the necessary surface area of 57,500 square feet. With a general slope of 2 percent, the surface area requirement appears to plausible with excavation of the pond at an elevation of 850 feet.

The designer decides to use the existing topography to reduce the cost of excavation by constructing a berm across the ravine. The designer needs to check whether this will produce the storage needed to detain storm water for water quality and if needed, for peak flow control. Sufficient space must exist for excavation of existing soils to conform roughly to the wet pond design guidelines to form the detention basin's wet permanent pond. The resulting detention basin plan surface areas for each elevation are given in table 7 and illustrated in figure 6.

Table 7. Basin site surface area before excavation.

Elevation (ft.)	Surface area (sq. ft.)
848	0
850	28,000
852	136,400
854	299,900
856	448,100

Figure 6. Preliminary detention basin site plan.



Check on permanent pond volume

The permanent pond volume is equal to 279,360 cubic feet. The designer now creates the preliminary design to contain the permanent pond volume at elevation 850 feet. The stage-storage table, table 8, indicates the storage volumes for designated elevations, and is used to assess the storage capacity at various elevations. Table 8 will later be used to assess the storage capacities for control of 5-micron sediment particles and for peak flow control as well.

At elevation 850 feet, the surface area is approximately 58,000 square feet, and the volume below this elevation must contain 279,360 cubic feet, or the drainage area runoff volume. Since these values exceed those of the current site, some excavation will be required to achieve design standards. Post excavation surface area and storage values are shown in table 8.

With this information the designer can begin the flow routing procedure to refine the management practice design, with an outlet and permanent pond surface elevation at 850 feet.

Notice at elevation 850.0 feet that the wet pond volume is 273,050 cubic feet or 6,310 cubic feet less than required. However, since this is approximately 2% of the required volume, it should not present a problem especially when considering the sediment storage also included into the design.

Step 7

Flow routing the drainage area runoff hydrograph for the 1.5-inch, 4-hour rain

After creating a preliminary practice design, a flow routing procedure is used to examine the hydrologic criteria of the management practice. This procedure determines the detention structure's storage and discharge characteristics relative to the expected inflow. By using this procedure, the designer can determine if the storage and discharge criteria are met. Flow routing is used in both the 1.5-inch, 4-hour rain and the 2-year, 24-hour storm assessments.

Table 8. Basin surface area and storage after excavation.

Stage elevation (ft.)	**Pond surface area**	**Incremental active storage volume** (cubic feet)	**total storage volume** (cubic feet)
844.0	40,000	-	0
846.0	42,600	82,600	82,600
848.0	45,300	87,900	170,500
849.0	50,900	48,100	218,600
850.0	58,000	54,450	273,050
852.0	136,400	194,400	467,450
854.0	299,900	436,300	903,750
856.0	448,100	748,000	1,651,750

Numerous methods exist for routing flows through small structures. The flow routing method used in this manual is based on NRCS Technical Release - 20 (TR-20) procedures (USDA-SCS 1992), as presented by McCuen (1982). The flow routing procedure uses the continuity equation:

$$I_{(mean)} - O_{(mean)} = \Delta S/\Delta t$$

Where:

$I_{(mean)}$ = mean inflow into the basin during the time increment Δt.

$O_{(mean)}$ = mean outflow out of the basin during the time increment Δt.

ΔS = change in stormwater storage volume in basin during the time increment Δt.

Δt = change in time for the increment or time period.

This equation may be expressed as:

$$\frac{I_1 + I_2}{2} - \frac{O_1 - O_2}{2} = \frac{S_2 - S_1}{\Delta t}$$

where the subscripts 1 and 2 indicate the beginning and the end of some time period. The times 1 and 2 are usually determined by picking some convenient length of time that will divide the rising side of the inflow hydrograph or table into equal time increments. The length of the time increment should be such that the change in inflow over the period is approximately linear. The number of increments usually exceeds three and is less than ten. The peak flow should be one of the points of division. The initial time is zero, and coincides with the time when inflow into the detention basin begins. Reconfiguring the equation so that the known terms are on the left side of the equal sign, and the unknown terms are on the right side of the equal sign, the equation becomes:

$$\frac{(I_1 + I_2)}{2} \times \Delta t + S_1 - \left[\frac{O_1}{2} \times \Delta t\right] = S_2 + \left[\frac{O_2}{2} \times \Delta t\right]$$

In this equation, the inflows are taken from a hydrograph or table and are, therefore, known. The initial storage and discharge at time zero is usually assumed to be zero. This assumes that the water is at the lip of the outlet structure and that only the active storage area will be used in routing. The terms to the right of the equal sign are calculated sequentially from one time segment to the next, beginning with time zero. The storage, S_2, and outflow, O_2, for the current time increment become the storage, S_1, and outflow, O_2 for the next time increment. The sequence continues until the critical storage and discharge characteristics for the management practice are obtained.

In determining the terms to the right of the equal sign, it is necessary to develop a number of tables or graphs. To make this assessment easier, a flow routing procedure has been devised. An outline of the process followed by a detailed description based upon the 1.5-inch, 4-hour storm follows. An example, as it applies to detention basin design, accompanies the description. Each outlet for a management practice will differ in its outlet characteristics and, therefore requires a unique expression for the outflow-rating curve. For example, the discharge rate for infiltration structures would be the infiltration rate, while the discharge rate for artificial wetland storm water management systems would be the infiltration rate, the evapotranspiration rate and the surface outflow.

For infiltration facilities and other management practices, the storage volume needed to limit the post-developed 2-year, 24-hour peak flow may be beyond the storage capacity. In this event a bypass or flow splitter would then be used to channel flow to a facility that would limit the discharge to the 2-year, pre-developed peak flow and store the excess inflow. The flow routing procedure consists of six steps. These steps are first described and then developed as part of the detention basin design example.

Step 7-A: Develop an expected inflow hydrograph or table. Determine a convenient time increment that divides the time period of rising inflow into equal time increments. The time segments should be divided into a minimum of four time increments. One of the points of division should coincide with the peak flow time. The initial time is zero, and coincides with the time when inflow into the management practice begins. In most cases the hydrograph or table will have been developed previously in step 4 of the design process. (Note: For the 2-year, 24-hour peak flow assessment, the time segments will follow the times identified in the TR-55 model.)

Step 7-B: Develop an elevation stage-storage curve or table for the proposed site. This is a graphical representation or tabulation of the storage volume relative to the water level or stage of the structure. It is necessary to survey the site and determine the surface area associated with a height or elevation in the structure. The incremental storage volume is then determined by summing the surface areas at the two elevations, averaging the areas and multiplying by the change in elevation.

Step 7-C: Develop a stage-discharge curve or table for the expected discharge structure. This is a description of the flow discharged from the management practice at an associated water surface stage or elevation in the structure.

Step 7-D: Construct a graph or table describing storage, (S_1), versus discharge, and storage plus discharge, ($S_2 + [O_2/2]\Delta t$), versus discharge.

Step 7-E: Test the outlet and the storage capacity of the rough design using the continuity equation, and the items developed in Steps 7-A through 7-D.

Step 7-F: Determine the maximum needed storage and discharge. Redesign the structure if the water quality criteria are not satisfied.

The flow routing procedure: an example

Step 7-A. Inflow hydrograph or table divided into incremental units of time.

The drainage area runoff becomes the inflow for the management practice. The time to peak inflow is 2.4 hours. In this example, an increment of 28.8 minutes was used to give five equal increments and one increment ending at peak time of 2.4 hours. Using this time increment, table 9 presents the inflow into the detention basin at the break points.(The determination of the length of the time increment is taken from Barfield, et. al., 1983.)

Table 9. Drainage area runoff for the 1.5- inch, 4-hour rainfall.

time (min.)	**inflow rate** (cu. ft./sec.)
0.00	0.00
28.80	6.44
57.60	12.88
86.40	19.32
115.20	25.76
144.00	32.20
172.80	25.76
201.60	19.32
230.40	12.88
259.20	6.44
288.00	0.00

Step 7-B. Develop stage-storage relationship

The stage-storage relationship is a correlation between the stage height or elevation of the pond's surface, and the amount of water stored at the associated water level. These data have been developed in table 7. The storage volume and surface area associated with a particular elevation are needed to determine the discharge characteristics for the 1.5-inch, 4-hour storm. With the proposed outlet elevation at 850 feet, only those values above the 850 foot elevation would apply to this condition. The storage totals have been recalculated in table 10.

Step 7-C. Develop a stage-discharge curve or table for the proposed outlet

To develop the stage-discharge curve for the outlet, the designer must first determine the type and size outlet that will be used. To get an estimate of the outlet height, make a rough calculation of the maximum storage. As a first estimate, roughly one-half of the drainage area runoff volume from the 1.5-inch rain, or 139,680 cubic feet, should be stored. At elevation 852 the volume is 194,400 cubic feet which is more than one-half the drainage area runoff volume of 279,360 cubic feet. Interpolating to find the elevation that stores one-half of 279,360 cubic feet shows:

$$\frac{(852.0\text{ ft} - Z)}{(852.0\text{ft} - 850.0\text{ft.})} = \frac{(194{,}400\text{ cu.ft.} - 139{,}680\text{ cu.ft})}{(194{,}400\text{ cu.ft.} - 0.0\text{ cu.ft.})}$$

$$Z = 851.4\text{ feet}$$

Table 10. Basin storage above the permanent pool.

Stage elevation (ft.)	Pond surface area (sq. feet)	Incremental active storage volume (cubic feet)	total active storage volume (cubic feet)
850.0	58,000		0
852.0	136,400	194,400	194,400
854.0	299,900	436,300	630,700
856.0	448,100	748,000	1,378,700

Assume an elevation of 851.50 feet, and use the design value of a maximum discharge of 0.00013 cubic feet per second for every square foot of pond surface area for removal of the 5-micron particle. Interpolating again to determine the surface area at elevation 851.50 feet gives:

$$\frac{(136{,}400\text{ sq.ft.} - S)}{(136.400\text{ sq.ft.} - 58{,}000\text{ sq.ft.})} = \frac{(852.0\text{ ft.} - 851.50\text{ ft.})}{(852.0\text{ ft.} - 850.0\text{ ft.})}$$

$$S = 116{,}800\text{ square feet}$$

To determine the maximum discharge, the surface area is multiplied by the maximum discharge per square foot of surface area given above:

$$Q_{(max.)} = 116{,}800\text{ sq.ft.} \times [(0.00013\text{ cu.ft./sec.})/(\text{sq.ft. of surface area})]$$

$$Q_{(max.)} = 15.18\text{ cu.ft./sec.}$$

Note: If the discharge rate at the highest expected head for the 1.5-inch, 4-hour event does not exceed the allowable discharge rate (or elevation) for removing the 5-micron particle, it is rare for the allowable discharge rate below the highest expected head to be exceeded unless the slope of the terrain or basin design varies sharply. Should steeper slopes exist in lower portions of the active storage range, a check of the allowable discharge rate should be done in this elevation range.

Total height of the weir in the pollutant removal range is 1.5 feet, and the maximum discharge is 15.18 cubic feet per second at elevation 851.5 feet.

Choosing a Cipolletti weir as the outlet gives a discharge equation (DOI-BREC, 1997)

$$Q_{(cfs)} = 3.367LH^{1.5}$$

A Cipolletti weir is a trapezoidal weir with sides that slope at an angle of 1 foot horizontal to 4 feet vertical. Two of the more important assumptions in using this formula are that the outlet is sufficiently elevated above the downstream water to eliminate backwater effects, and that the edge of the weir has a sharp edge which reduces the friction of water discharging through the weir. The designer should make an accurate assessment of the discharge characteristics of the designed outlet. There are several sources that may assist the designer in this regard (for example, see the water measurement manual, USDI-BREC, 1997).

Using this formula, a weir width is determined by substituting the calculated maximum discharge 15.18 cubic feet per second for $Q_{(cfs)}$, and 1.5 feet for H.

15.18 cu.ft. = 3.367 x L x 1.5 ft.$^{1.5}$

L = 2.45 feet or 29.45 inches wide

Using a 30.00 inch or 2.50 foot weir, the equation becomes:

$Q_{(cfs)} = 8.42\ H^{1.5}$

Table 11 indicates the discharge associated with a given stage height for the proposed weir. The stage height is the height above the 850.0 feet elevation, or the surface of the permanent pond.

Table 11. Stage-discharge for the 2.5-foot weir.

Stage height (ft.)	**Discharge** (cfs)
0.00	0.00
0.20	0.75
0.40	2.13
0.60	3.91
0.80	6.02
1.00	8.42
1.20	11.07
1.40	13.95
1.50	15.47
1.60	17.04
1.80	20.33
2.00	23.81

Table 12. Storage-discharge for 2.5-foot weir.

Row	**Elev.** (ft.)	**Surface Area** (sq. ft.)	**Storage** (cu. ft.)	**Disc. Rate** (cfs)	*** Disc. Vol. for Increment** (cu. ft.)	*** Storage + [(Qave.)(Δ T)]** (cu. ft.)
1	850.0	58,000		0.0		
2	850.20	65,840	12,384	0.75	651	13,035
3	850.40	73,680	26,336	2.13	1,840	28,176
4	850.60	81,520	41,856	3.91	3,381	45,237
5	850.80	89,360	58,944	6.02	5,205	64,149
6	851.00	97,200	77,600	8.42	7,275	84,875
7	851.20	105,040	97,824	11.07	9,563	107,387
8	851.40	112,880	119,616	13.95	12,051	131,667
9	851.60	120,720	142,976	17.04	14,723	157,699
10	851.80	128,560	167,904	20.33	17,568	185,472
11	852.00	136,400	194,400	23.82	20,576	214,976
12	852.20	152,750	223,315	27.48	23,739	247,054
13	852.40	169,100	255,500	31.31	27,048	282,548
14	852.60	185,450	290,955	35.30	30,499	321,454
15	852.80	201,800	329,680	39.45	34,085	363,765
16	853.00	218,150	371,675	43.75	37,801	409,476
17	853.20	234,500	416,940	48.20	41,644	458,584
18	853.40	250,850	465,475	52.79	45,608	511,083
19	853.60	267,200	517,280	57.51	49,691	566,971
20	853.80	283,550	572,355	62.37	53,889	626,244
21	854.00	299,900	630,700	67.36	58,199	688,899
22	854.20	314,720	692,162	72.47	62,618	754,780
23	854.40	329,540	756,588	77.71	67,144	823,732
24	854.60	344,360	823,978	83.07	71,773	895,751
25	854.80	359,180	894,332	88.55	76,505	970,837
26	855.00	374,000	967,650	94.14	81,336	1,048,986
27	855.20	388,820	1,043,932	99.84	86,264	1,130,196
28	855.40	403,640	1,123,178	105.66	91,289	1,214,467
29	855.60	418,460	1,205,388	111.58	96,407	1,301,795
30	855.80	433,280	1,290,562	117.61	101,617	1,392,179
31	856.00	448,100	1,378,700	123.75	106,918	1,485,618

Step 7-D. Storage versus discharge curve, and storage plus discharge versus discharge curve

The storage versus height, and storage plus discharge versus height curve is used to determine the unknown terms on the right side of the continuity equation for the flow routing procedure.

$$\left[\frac{(I_1 + I_2)}{2} \times \Delta t\right] + \left[S_1 - \frac{O_1}{2} \times \Delta t\right]$$

$$= S_2 + \frac{O_2}{2} \times \Delta t$$

Table 12 assists in developing the curve shown in figure 7.

The discharge rate is $Q = 8.42\ H^{1.5}$

Step 7-E. Construction of the flow routing table

With the inflow table or hydrograph, and the storage-discharge curve that have been developed, determine the right hand terms of the continuity equation;

$$\left[\frac{(I_1 + I_2)}{2} \times \Delta t\right] + \left[S_1 - \frac{O_1}{2} \times \Delta t\right] = S_2 + \frac{O_2}{2} \times \Delta t$$

A sequential assessment is done taking a known term on the left side of the equation for a series of time increments to determine the terms on the right side of the equation. By taking the beginning and ending inflow rates for each time increment from the inflow hydrograph or inflow table, averaging them, and multiplying by the change in time, $[(I_1 + I_2)/2 * \Delta t]$ can be calculated. S_1, or storage at the beginning of the time increment, is initially equal to zero, or has been determined by solving the continuity equation for the previous time increment and is therefore known. Discharge, O_1, is initially equal to zero or determined by solving the continuity equation for the previous time increment and therefore known. The right hand terms are determined by using the storage-discharge graph or table. This method is usually easier to perform and record in a tabular form. The tabular method is described and demonstrated using the actual values for the detention basin example given in table 13.

Determining the values for the table involves using the drainage area runoff for the 1.5-inch, 4-hour rainfall, table 9, and the storage-discharge graph, figure 7 or table 12.

Column A in table 13 represents a specific start and/or finish time for a time increment. Values in columns B, G and H represent either a rate of flow or storage volume for a specific time. These columns have values entered in those rows with an identified time in column A only (in this case, all the odd-numbered rows). Columns C, D, E and F are flow rates and volumes relative to the entire time increment. These columns will have values entered in those rows between times specified in column A only (in this case, all the even rows).

The initial start time, or time zero, is the beginning of inflow into the management practice. At the start time, inflow, outflow and storage, columns B, G and H are all assumed to be zero. The times in column A and the inflow rates in column B have been obtained from the inflow table (table 9).

Figure 7. Storage-discharge curve.

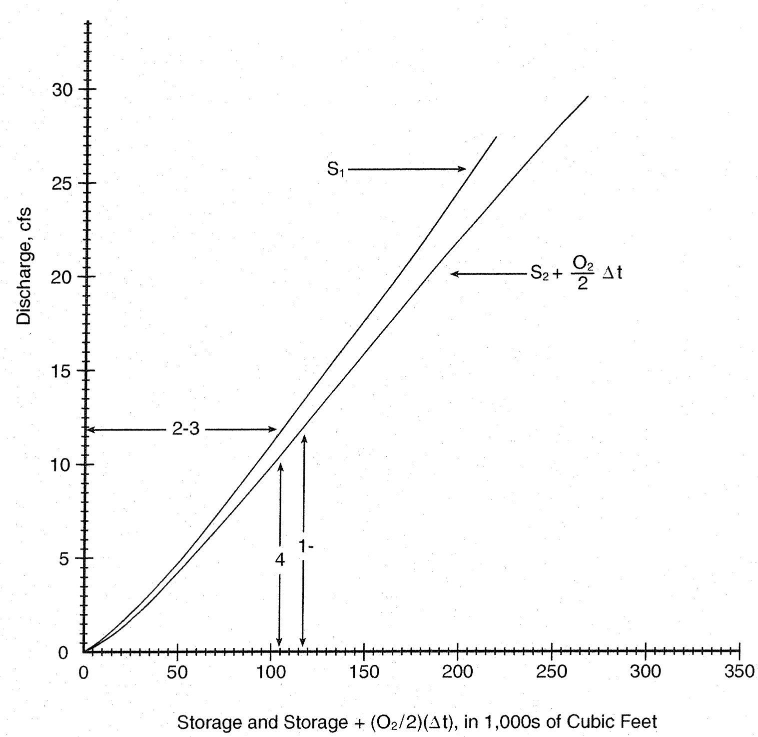


Table 13. Flow routing for 2.5-foot weir.

row	(A) start of time increment (min.)	(B) starting inflow rate (cfs)	(C) average inflow rate (cfs)	(D) 0.5(i1+i2)ti inflow volume (cu. ft.)	(E) previous storage - previous incremental outflow (cu. ft.)	(F) final storage + final incremental outflow (cu. ft.)	(G) outflow rate (cfs)	(H) storage (cu. ft.)
1	0	0		-			0.00	0
2			3.22	5,564	0	5,564		
3	28.80	6.44		-			0.32	5,286
4			9.66	16,692	5,010	21,702		
5	57.60	12.88		-			1.54	20,371
6			16.1	27,821	19,040	46,861		
7	86.40	19.32		-			4.09	43,323
8			22.54	38,949	39,788	78,738		
9	115.20	25.76		-			7.71	72,076
10			28.98	50,077	65,415	115,492		
11	144.00	32.20		-			12.03	105,099
12			28.98	50,077	94,703	144,781		
13	172.80	25.76		-			15.51	131,384
14			22.54	38,949	117,986	156,935		
15	201.60	19.32		-			16.95	142,291
16			16.1	27,821	127,646	155,467		
17	230.40	12.88		-			16.78	140,973
18			9.66	16,692	126,480	143,172		
19	259.20	6.44		-			15.32	129,940
20			3.22	5,564	116,707	122,272		
21	288.00	0.00		-			12.84	111,183
22			0	0	100,093	100,093		
23	316.80			-			10.21	91,272

In table 13 the column headings are defined in this manner:

Column A: Time increment is the start/finish of the time increment.

Example:

In this case the time increment is 28.8 minutes, or 1,728 seconds taken from table 9 for the inflow rates.

Column B: Inflow into the basin at the beginning of the time increment. (This is the flow from the drainage area that the basin serves.) These values are taken from the inflow (Table 9) or hydrograph .

Example:

From table 9, the inflow rates for times 0.0 min. and 28.8 min. are 0.0 cfs and 6.44 cfs respectively and the inflow rates for times 115.2 min. and 144.0 min. are 25.76 cfs and 32.20 cfs respectively. These values are entered into their respective times in column B of table 13.

Column C: Average inflow rate into the basin over the time segment is the average inflow of the time increment, or the left term, $(I_1 + I_2)/2$, of the continuity equation.

Example:

From table 9, the average inflow of rows 1 and 3 is:

(6.44 cfs + 0.0 cfs)/2 = 3.22 cfs.

The average inflow of rows 9 and 11 is (25.76 cfs + 32.20 cfs)/2 = 28.98 cfs.

These values are entered into column C.

Column D: Inflow volume for the time increment, which is calculated by multiplying the time increment by the average inflow.

Example:

Row 2: (3.22 cfs) x [(28.8 min. - 0.0 min.) x (60 sec./min.)] = 5,564 cu. ft.

Row 10: (28.98 cfs) x [(144.0 min. - 115.2 min.) x (60 sec./min.)] = 50,077 cu. ft.

Column E: The $[(S_1-(O_1/2)\ \Delta t]$ term of the continuity equation. The storage volume, (the storage volume at the beginning of the time increment obtained from the adjacent row above in column H), minus the average discharge for the time increment, (the discharge rate at the beginning of the time increment obtained from the adjacent row above in column G divided by 2) times the change in time for the increment. In detention basins, when inflow first begins, the active storage, or water volume above the crest of the outlet, and the discharge are assumed to be zero. Infiltration structures would be empty with no active storage and no discharge. In artificial wetland storm water management systems, the discharge rate is assumed to be the infiltration rate of the structure at the level where surface outflow is zero, and evapotranspiration is assumed to be zero.

Example:

Row 2; from column H, row 1, the initial storage above elevation 850 feet is zero at time zero. From column G, row 1 the discharge is also zero. We have:

(0.0 cu. ft.) - [(0.0 cfs) x (28.8 min. x 60 sec./min.)] = 0.0 cu. ft.

In row 10; (72,076 cu. ft.) - [(7.71 cfs/2) x ((144.0 min. - 115.2 min.) x (60 sec./min.))] = 65,415 cu. ft.

Column F: Is the $[(S_2+(O_2/2)\Delta t]$ term of the continuity equation. This value is just the sum of the values in columns D & E.

Example:

Row 2; (5,564 cu. ft.) + (0 cu. ft.) = 5,564 cu. ft.

In row 10; 50,077 cu. ft. + 65,415 cu. ft. = 115,492 cu. ft.

Column G: The discharge rate at time T in column A. To determine the discharge rate, the previous $S_2 + (O_2/2)\Delta t$ term in column F in table 13 is used with the Storage-Discharge Curve (Figure 7), or Storage-Discharge Table (Table 12). Find the value for $S_2 + (O_2/2)\Delta t$ term on the X-axis of the Storage-Discharge Curve and move vertically until intersecting the graph of $S_2 + (O_2/2)\Delta t$ versus discharge line, and then move left horizontally until intersecting the Y-axis to approximate the discharge rate. This value is the flow rate at the end of the time increment and is placed one row below the value taken in column F.

Using the Storage-Discharge Table (Table 12) in the Storage +($Q_{aver.}$) x Δt column, find a value greater and less than the value of column F in the Flow Routing Table (table 13). Using these values, the corresponding values from the discharge rate column in the Storage-Discharge Table, and the value of column F in the Flow Routing Table, interpolate a discharge value.

Example:

Row 3, using the Storage-Discharge Curve (Figure 7). The value of 5,564 cu. ft.is taken from row 2, column F. Locating this value on the X-axis moving vertically, intersecting the $S_2 + (O_2/2)\Delta t$ versus discharge line and then moving horizontally to the left, the approximate discharge rate is 0.32 cfs.

Row 11: Repeating the above sequence, from row 10, column F, $S_2 + (O_2/2)\Delta t$ = 115,492 cu. ft. and the associated discharge is approximately 12.03 cfs.

Row 3 Storage-Discharge Table. In column F, values 13,035 cu. ft. and 0 are values that contain 5,564 cu. ft. the corresponding values for discharge are 0.75 cfs and 0.00 cfs. Interpolating we have:

$$\frac{0.75 \text{ cfs} - O_2}{0.75 \text{ cfs} - 0.00 \text{ cfs}} = \frac{13{,}035 \text{ cu. ft.} - 5{,}564 \text{ cu.ft.}}{13{,}035 \text{ cu ft.} - 0.00 \text{ cu.ft.}}$$

O_2 = 0.32 cfs. which is placed in row 3 column G

Row 3, using the Storage-Discharge Table; Values 13,035 cu. ft. and 0 are values that contain 5,564 cu. ft. The corresponding values for storage from table 15 are 12,384 cu. ft and 0.00 cu. ft. Interpolating, we have:

$$\frac{12{,}384 \text{ cu.ft} - S_2}{12{,}384 \text{ cu.ft.} - 0.00 \text{ cu.ft.}} = \frac{13{,}035 \text{ cu.ft.} - 5.564 \text{ cu.ft}}{13{,}035 \text{ cu.ft.} - 0.00 \text{ cu.ft}}$$

S_2 = 5,286 cu. ft.

Repeating this sequence for row 11:

O_2 = 12.03 cfs

Column H: The storage at time T indicated in column B. The storage is determined using the Storage-Discharge Curve and the discharge value from column G. Using the discharge value just determined in column G, move horizontally to the right until intersecting the storage curve, S_1. Move vertically down the graph until intersecting the X-axis to determine the approximate storage value.

This value can also be found using the Storage-Discharge Table (table 12). Again in the Storage + (O_{aver}) x Δt column, find a value greater and less than the value of column F in the Flow Routing (table 13). Using these values, the corresponding values from the storage column in the Storage-Discharge Table, and the value of column F in the Flow Routing Table interpolate a discharge value.

Example:

Row 3, using the Storage-Discharge Curve from column G, we have 0.32 cfs. Locating 0.32 cfs on the Y-axis of the Stage-Discharge Curve, move horizontally to the right until intersecting the S_1 discharge curve and then move vertically down, we have an estimated value of 5,300 cu. ft. which we enter into the column H.

Row 11, using the Storage-Discharge Curve; Repeating the above procedure, from column G, discharge = 12.03 cfs, gives an approximate value of 105,000 cu. ft. of storage.

Please note that O_2 and S_2 now become O_1 and S_1 for the next time increment.

Step 7-F. Maximum necessary storage and discharge

From the Flow Routing Table (table 13) the maximum peak storage and flow occurs in line 15. The peak storage is approximately 142,291 cubic feet and the maximum outflow for the outlet is 16.95 cubic feet per second. From this point forward the storage volume and discharge rate decline. The predicted maximum storage volume was 139,690 cubic feet, and the maximum allowable discharge is 15.2 cfs at an elevation of 852.5 feet. The actual results are above the expected elevation and volume. Checking to determine if the pond will still settle the 5-micron particle we have:

Required elevation, Er: (values taken from table 12)

[(851.6 ft. - E_r)/(851.6 ft.- 851.4 ft.)] = [(142,976 cu. ft. - 142,291 cu. ft.)/ (142,976 cu.ft. - 119,616 cu. ft.)]

851.6 ft. - E_r = 0.2 ft. x 0.0293

E_r = 851.6 ft. - 0.0059 ft. = 851.594 ft.

The surface area, A, at this elevation is: (values taken from table 12)

851.59 ft. is approximately 851.6 ft. and the surface area at this elevation is 120,720 sq. ft. allowable discharge, Q, = 120,720 sq. ft. * 0.00013 ft./sec.

Q = 15.69 cfs. (this is less than 16.95 cfs)

The outlet must be reduced to fulfill the requirements of settling velocity for the 5-micron particle. We now must return to step 7-C and choose an outlet size and develop a "stage-discharge table and curve."

By downsizing the weir, the storage height is likely to be higher due to the restriction of flow at lower elevations, so use an elevation of 851.8 ft.

Q = 128,560 sq.ft. x 0.00013 ft./sec
= 16.71 cfs

Q = 16.71 cfs = $3.367LH^{1.5}$

16.71 cfs = 3.367 x L x $(1.8')^{1.5}$

L = 2.06 ft. try 2.00 ft. weir

We now must work through the process, starting with the development of the stage-discharge table (Step 7-B in this example), storage-discharge table and curve, and, finally, checking the BMP design by developing a flow routing table (Step 7-E in this example) as we did earlier. The result of repeating these steps is shown in tables 14, 15 and 16. The results are, from table 16, line 17, a discharge of 15.09 cfs and storage of 156,827 cu.ft. Calculating the corresponding elevation and surface area we have:

E = 851.80 ft. -[(851.80 ft. - 851.60 ft.) x ((167,904 cu.ft. - 156,827 cu.ft.)/ (167,904 cu.ft. - 142,976 cu.ft.))]

E = 851.7 ft.

A = 128,560 sq.ft. - [(128,560 sq.ft. - 120,720 sq.ft.) x ((167,904 cu.ft. - 156,827 cu.ft.)/(167,904 cu.ft. - 142,976 cu.ft.))]

A = 125,076 sq.ft.

Allowable discharge to settle the 5-micron particle size is;

Q = 125,076 sq.ft. x 0.00013 ft./sec = 16.26 cfs

The discharge of 15.09 cfs is less than the allowable discharge of 16.26 cfs for the assumed weir. We could refine the weir size further by repeating the process once more; however, for this example we will assume the design is within design expectations.

Table 14. Stage-discharge table for 2-foot weir

Stage height (ft)	Discharge (cfs)
0.00	0.00
0.20	0.60
0.40	1.70
0.60	3.13
0.80	4.82
1.00	6.73
1.20	8.85
1.40	11.15
1.50	12.37
1.60	13.63
1.80	16.26
2.00	19.05

Step 8: Existing peak flow and hydrograph of developed drainage area peak flows for the 2-year, 24-hour storm

Steps 8 through 10 are practice design features created to limit the 2-year, 24-hour storm. These steps are based on guidance material described in the NRCS manual TR-55 (USDA-SCS, 1986). Only a summary of the procedure and the detention basin example results are given here. Designers should contact NRCS for a copy of TR-55. Because an assessment of the management practice structure requires a flow routing procedure, the tabular hydrograph method of TR-55 is required and designers should structure the assessment and data with this in mind.

To size the basin outlet to limit the 2-year, 24-hour runoff peak flow from the drainage area in fully developed condition to the peak flow in the pre-developed condition, an estimate of the needed storage volume must be made and checked using the flow routing procedure. This will require routing the 2-year, 24-hour runoff from the developed site through a two-stage outlet designed to limit this runoff to the 2-year, 24-hour peak in the existing condition. The lower stage, the first 1.7 feet above the crest of the outlet, is for the active storage to remove the 5-micron particle from the 1.5-inch, 4-hour rain. The second stage of the outlet design is to limit the 2-year, 24-hour storm peak flow. To size the second stage of the basin, an outlet must be designed to incorporate the design characteristics developed in Steps 1 through 7.

The 2-year, 24-hour peak flow with the site in its existing condition is established following the method in TR-55. Using the data collected by the designer and the surveyor and TR-55, we have:

$$Q = \frac{(P - 0.2S)^2}{(P - 0.8S)}$$

Where:

Q = runoff (inches)

P = rainfall (2.7 inches for Wisconsin)

S = potential maximum retention after runoff begins (inches) and

$$S = \frac{1000}{CN} - 10$$

Where:

S = potential maximum retention after runoff begins (inches)

CN = the SCS runoff curve number = 71.

Calculating the runoff yields the following:

$$S = \frac{1000}{71} - 10 = 4.08$$

and

$$Q = \frac{(2.7 - 0.2 \times 4.08)^2}{(2.7 + 0.8 \times 4.08)} = 0.59 \text{ inches}$$

To determine time of concentration and travel time, the designer uses the calculation procedure, the data for the existing site, and the work sheet shown in table 17. The existing channel cross-sections are given in table 3, and the surveyed channel locations along with the flow path are indicated in figure 3.

Existing site time of concentration = 0.73 hours.

Because it is necessary to route the runoff through the designed detention basin, the tabular hydrograph method from TR-55 is required for the developed condition. To be consistent, the existing condition will also use this method. The drainage area is uniform in land-use, soils and land cover and therefore will not be subdivided into sub-drainage areas in the existing condition.

The time of concentration, t_c is 0.73 hours from above, or approximately 0.75 hours

The CN is 71, from survey data

The rainfall is 2.7 inches, type II storm distribution.

Total runoff in inches, Q, is 0.59 inches.

The initial abstraction for CN value of 71, from Table 5-1 in TR-55, is 0.817.

The value $I_a/P = 0.817/2.7 = 0.30$.

Using these results, the designer selects the peak tabular unit discharge of 348 csm/in from page 6 of Exhibit 5-II in TR-55. Using this value the designer calculates a peak flow using the equation on page 5-2 of the TR-55 manual, which is:

$$q = q_t(A_m)(Q)$$

Where: q = hydrograph coordinate (cfs) at hydrograph time t; peak flow is the only point of concern for the existing condition.

q_t = tabular hydrograph unit discharge from exhibit 5 in cubic feet per second for each square per inch of runoff, (csm/in)

Am =drainage area in square miles

Q =depth of runoff in inches

q = (348csm/in)[122.4 acres x (1 sq. mi./640 acres)] [0.59 inches] = 39.3 cfs

Table 15. Stage-discharge table for 2-foot weir.

Row	Elev. (ft.)	Surface Area (sq. ft.)	Storage (cu. ft.)	Disc. Rate (cfs)	Disc. Vol. for Increment (cu. ft.)	Storage [(Qave.)(Δ T)] (cu. ft.)
1	850.0	58,000	0.	0.0	0.	0.
2	850.20	65,840	12,384	0.60	520	12,904
3	850.40	73,680	26,336	1.70	1,472	27,808
4	850.60	81,520	41,856	3.13	2,704	44,560
5	850.80	89,360	58,944	4.82	4,163	63,107
6	851.00	97,200	77,600	6.73	5,818	83,418
7	851.20	105,040	97,824	8.85	7,648	105,472
8	851.40	112,880	119,616	11.15	9,638	129,254
9	851.60	120,720	142,976	13.63	11,775	154,751
10	851.80	128,560	167,904	16.26	14,051	181,955
11	852.00	136,400	194,400	19.05	16,456	210,856
12	852.20	152,750	223,315	21.97	18,985	242,300
13	852.40	169,100	255,500	25.04	21,632	277,132
14	852.60	185,450	290,955	28.23	24,392	315,347
15	852.80	201,800	329,680	31.55	27,260	356,940
16	853.00	218,150	371,675	34.99	30,232	401,907
17	853.20	234,500	416,940	38.55	33,305	450,245
18	853.40	250,850	465,475	42.22	36,476	501,951
19	853.60	267,200	517,280	46.00	39,741	557,021
20	853.80	283,550	572,355	49.88	43,099	615,454
21	854.00	299,900	630,700	53.87	46,545	677,245
22	854.20	314,720	692,162	57.96	50,080	742,242
23	854.40	329,540	756,588	62.15	53,699	810,287
24	854.60	344,360	823,978	66.44	57,402	881,380
25	854.80	359,180	894,332	70.82	61,186	955,518
26	855.00	374,000	967,650	75.29	65,049	1,032,699
27	855.20	388,820	1,043,932	79.85	68,991	1,112,923
28	855.40	403,640	1,123,178	84.50	73,009	1,196,187
29	855.60	418,460	1,205,388	89.24	77,103	1,282,491
30	855.80	433,280	1,290,562	94.06	81,270	1,371,832
31	856.00	448,100	1,378,700	98.97	85,509	1,464,209

Table 16. Flow routing table for 2-foot weir.

row	(A) start of time increment (min.)	(B) starting inflow rate (cfs)	(C) average inflow rate (cfs)	(D) 0.5(i1+i2)ti inflow volume (cu. ft.)	(E) previous storage - previous incremental outflow (cu. ft.)	(F) final storage + final incremental outflow (cu. ft.)	(G) outflow rate (cfs)	(H) storage (cu. ft.)
1	0	0		-			0.00	0
2			3.22	5,564	0	5,564		
3	28.80	6.44		-			0.26	5,340
4			9.66	16,692	5,116	21,809		
5	57.60	12.88		-			1.26	20,720
6			16.1	27,821	19,634	47,455		
7	86.40	19.32		-			3.39	44,523
8			22.54	38,949	41,591	80,540		
9	115.20	25.76		-			6.46	74,956
10			28.98	50,077	69,375	119,453		
11	144.00	32.20		-			10.20	110,635
12			28.98	50,077	101,820	151,898		
13	172.80	25.76		-			13.35	140,362
14			22.54	38,949	128,825	167,775		
15	201.60	19.32		-			14.89	154,910
16			16.1	27,821	142,046	169,867		
17	230.40	12.88		-			15.09	156,827
18			9.66	16,692	143,788	160,481		
19	259.20	6.44		-			14.18	148,226
20			3.22	5,564	135,971	141,535		
21	288.00	0.00		-			12.34	130,868
22			0	0	120,202	120,202		
23	316.80			-			10.27	111,322
24			0	0	102,445	102,445		
25	345.60			-			8.56	95,048
26			0	0	87,653	87,653		
27	374.40			-			7.14	81,483
28			0	0	75,317	75,317		
29	403.20			-			5.97	70,159
30			0	0	65,003	65,003		
31	432.00			-			5.00	60,685
32		0	0		56,367	56,367		
33	460.80						4.21	52,734

Table 17. Time of concentration—existing condition.

Worksheet 3: Time of concentration (Tc) or Travel time (Tt)

Project: Hypothetical # 1 By: Ace Designer Date: 2/08/9

Location Checked by: Trump Designer Date: 2/08/99

Circle one: Present Developed Present

Circle one: Tc Tt Through Subarea T_c

Notes: Space for as many as two segements per flow type can be used for each worksheet. All references to terms and tables are from the SCS TR-55 Manual—2nd ed. June 1986. Include a map, schematic or description of flow segments.

Sheet flow (Applicable to Tc only) Segment ID	AB		
1. Surface description (table 3-1)	Dense Grass		
2. Manning's roughness coeff., n (table 3-1)	0.24		
3. Flow length, L (total L less than or equal to 300 ft,) in feet.	280		
4. Two-yr., 24-hr. rainfall, P2 in inches.	2.7		
5. Land Slope, s in ft./ft.	0.03		
6. $T_t = (0.007 * (nL)^{\wedge}0.8) / (P2^{\wedge 0.5} * s^{\wedge 0.4})$Compute T_t	0.5	—	0.5
Shallow concentrated flow Segment ID	BC		
7. Surface description (paved or unpaved)	Unpaved		
8. Flow length, L in feet	880		
9. Watercourse slope, s in feet/feet	0.03		
10. Average velocity, V (figure 3-1) in feet/second	2.8		
11. $T_t = L/(3600 * V)$ Compute T_t	0.09	—	0.09
Channel flow Segment ID	CD	DE	
12. Cross sectional flow area, a in square feet	1.62	3.24	
13. Wetted perimeter, Pw in feet	3.8	4.8	
14. Hydraulic radius, r = (a / Pw) Computer	0.43	0.68	
15. Channel slope, s infeet/feet	0.03	0.03	
16. Mannings roughness coefficient, n	0.04	0.04	
17. $V = (1.49 * r^{\wedge}(2/3) * s^{\wedge}(0.5)) / n$, Compute V in feet/second	3.65	4.99	
18. Flow length, L in feet	560	920	
19. Tt = L/(3600 * V) Compute Tt in hours	0.04	0.05	0.09
20. Watershed or subarea T_c, or T_t (add T_t in steps 6, 11, and 19) in hours			0.68
Channel flow Segment ID	EF		
12. Cross sectional flow area, a in square feet	5.35		
13. Wetted perimeter, P_w in feet	7.75		
14. Hydraulic radius, $r = (a / P_w)$ Computer	0.69		
15. Channel slope, s in feet/feet	0.035		
16. Mannings roughness coefficient, n	0.035		
17. $V = (1.49 * r^{\wedge(2/3)} * s^{\wedge(0.5)}) / n$, Compute V in feet/second	6.22		
18. Flow length, L in feet	1190		
19. $T_t = L/(3600 * V)$ Compute T_t in hours	0.05	0	0.05
20. Watershed or subarea Tc, or T_t (add Tt in steps 6, 11, and 19) in hours			0.73

T_c is approximately 0.75 hours with an Ia / P of approximately of 0.30 T_t = 0.0 hours

Table 18. Basic watershed data—existing condition

Worksheet 5a: Basic Watershed Data—Existing Condition.

Project: Hypothetical # 1 **Location:** New Discovery, Wisconsin **By:** Ace Designer Date: 2/08/99

Circle one: Present, developed present Frequency (yr.) 2 yr. Checked by: Trump Designer Date: 2/08/99

Notes: All references to terms and tables are from the SCS TR-55 Manual—2nd ed. June 1986. Include a map schematic or description of flow segments.

Subarea Name	Drainage Area A_m, (mi^2)	Time of Concentration T_c, (hr.)	Travel time thru area T_t, (hr.)	Downstream subarea names	Travel time sumation to outlet Sum of T_t (hr.)	24-hr. Rainfall P (in.)	Runoff Curve Number CN	Runoff Q (in.)	AmQ (mi.^{2}-in.)	Initial abstraction I_a (in.)	I_a/P
total	0.191	0.75	—	—	—	2.7	71	0.59	0.113	0.817	0.3

Table 19. Tabular hydrograph discharge summary—existing condition.

Worksheet 5b: Tabular hydrograph discharge summary—Existing Condition

Project: Hypothetical #1 Location: New Discovery, WI. By: Ace Designer Date: 02/08/99

Select one: Present, Developed Present Frequency: (yr.) 2 yr. Checked by: Trump Designer Date: 02/08/99

Notes: All referances to terms and tables are from the SCS TR-55 Manual—2nd ed. June 1986. Include a map, schematic or description of flow segments.

	Basic watershed data used[1/]				Select and enter hydrograph times in hours from exhibit 5- [2/]											
Sub-area name	Sub-area T_c (hr.)	Sumation T_t outlet (hr.)	I_a/P	A_mQ (mi.^{2}-in.)	12.2	12.3	12.5	12.6	12.7	12.8	12.9	13	13.2	13.4	13.6	13.8
unit					30	86	174	266	326	348	328	246	181	138	110	92
existing	0.75	0	0.3	0.113	3.4	9.7	19.7	30.1	36.8	39.3	37.1	27.8	20.5	15.6	12.4	10.4

Table 20. Time of concentration worksheet—developed condition.

Worksheet 3: Time of concentration (Tc) or Travel time (Tt)

Project: Hypothetical # 1

Location: Ideal Wisconsin

Designed by: Ace Designer Date: 2/08/99

Checked by: Trump Designer Date: 2/08/99

Circle one: present, developed Circle one: T_c Tt

Notes: Space for as many as two segments per flow type can be used for each worksheet. All references to terms and tables are from the SCS TR-55 Manual—2nd ed. June 1986. Include a map, schematic or description of flow segments.

Sheet flow (Applicable to Tc only)	Segment ID	AB		
1. Surface description (table 3-1)		Dense Grass		
2. Manning's roughness coeff., n (table 3-1)		0.24		
3. Flow length, L (total L less than or equal to 300 ft.,) in ft.		115		
4. 2-year, 24-hour rainfall, P_2 in inches.		2.7		
5. Land slope, s in ft./ft.		0.03		
6. $T_t = (0.007 * (nL)^{\wedge 0.8}) / (P_2{}^{\wedge 0.5*} s^{\wedge 0.4})$ Compute T		0.25	0	0.25
Shallow concentrated flow	Segment ID	BC	CD	
7. Surface description (paved or unpaved)		Unpaved	Paved	
8. Flow length, L in feet		65	265	
9. Watercourse slope, s in feet		0.02	0.03	
10. Average velocity, V (figure 3-1) in feet/second		2.2	3.6	
11. $T_t = L/(3600 * V)$ Compute T_t		0.01	0.02	0.03
Channel flow	Segment ID	DE		
12. Cross sectional flow area, a, in square feet		0.9		
13. Wetted perimeter, P_w in feet		4.2		
14. Hydraulic radius, $r = (a/P_w)$, Compute r		0.21		
15. Channel slope, s in feet/feet		0.04		
16. Mannings roughness coefficient, n		0.02		
17. $V = (1.49 * (r^{\wedge 2/3}) * s^{\wedge(0.5)})/n$, Compute V in feet/second		4.5		
18. Flow length, L in feet		460		
19. $Tt = L/(3600 * V)$ Compute T_t in hours		0.03		0.03
20. Watershed or subarea T_c, or T_t (add T_t in steps 6, 11, and 19) in hours				0.30

Tables 18 an 19 show the calculations of the peak flow for the existing condition. To develop the hydrograph for the drainage area in the fully developed condition, repeat the TR-55 sequence that was used to create the 2-year, 24-hour peak flow for the drainage area in its existing condition. Calculate the time of concentration using figure 5. The flow times for this example were calculated using both TR-55 and pipe flow calculation. The *Standard Handbook for Civil Engineers*—(Merritt, 1983) was used for the pipe calculations based on the Manning equation. Please note that the longest flow path in the developed condition is different than the flow path in the existing condition, because the drainage area serviced by the storm sewer system changes the flow direction.

Please refer to tables 20 through 22 for the results of the flow table in the developed condition. The information in the tabular hydrograph discharge summary can be used to produce an outflow hydrograph and also informs the designer of the 2-year, 24-hour peak flow, and required maximum storage needed for the management practice with the area in the fully developed condition.

Table 21. Basin watershed data—developed condition.

Worksheet 5a: Basic watershed data—developed condition

Project Hypothetical # 1 Location New Discovery, WisconsinBy: Ace Designer Date: 2/08/99

Circle one: Present, Developed Developed Frequency (yr.) 2 yr. Checked By: Trump Designer Date: 2/08/99 **Notes:** All references to terms and tables are from the SCS TR-55 Manual—2nd ed. June 1986. Include a map, schematic or description of flow segments.

Subarea Name	**Drainage Area**	**Time of Concen-tration**	**Travel time thru area**	**Downstream subarea names**	**Travel time sumation to outlet**	**24-hr. Rainfall**	**Runoff Curve Number**	**Runoff**	**AmQ**	**Initial abstraction**	**I_a/P**
	A_m, (mi^2)	T_c, (hr.)	T_t, (hr.)		Sum of T_t (hr.)	P (in.)	CN	Q (in.)	(mi.^{2}-in.)	I_a (in.)	
1	0.056	0.3	-	2,5,6	0.09	2.7	80.5	1.06	0.060	0.484	0.18
2	0.052	0.3	0.04	5,6	0.05	2.7	88.2	1.57	0.082	0.268	0.10
3	0.025	0.3	0.03	4	0.02	2.7	87.1	1.49	0.037	0.296	0.11
4	0.016	0.2	0.02	6	0	2.7	80.8	1.08	0.017	0.475	0.18
5	0.033	0.3	0.05	-	0	2.7	78.8	0.96	0.032	0.538	0.20
6	0.014	0.5	0.005	-	0	2.7	71.0	0.59	0.008	0.817	0.30

Table 22. Tabular hydrograph discharge summary—developed condition.

Worksheet 5b: Tabular hydrograph discharge summary—developed condition

Project: Hypothetical #1 Location: New Discovery, WI. By: Ace Designer Date: 02/08/99

Select one: Present, Developed Developed Frequency: (yr.) 2 yr. Checked by: Trump Designer Date: 02/08/99

Notes: All references to terms and tables are from the SCS TR-55 Manual—2nd ed. June 1986. Include a map, schematic or description of flow segments.

Sub-area name	Basic watershed data used[1/]: Sub-area T_c	Suma-tion T_t outlet (hr.)	I_a/P (hr.)	A_mQ (mi.^{2}-in.)	Select and enter hydrograph times in hours from exhibit 5- [2/]: 11	11.3	11.6	11.9	12	12.1	12.2	12.3	12.4	12.5	12.6	12.7	12.8	13
					Tabular Hydrograph Unit Discharges (CFS sq. mi./in.)													
	0.2	0	0.1		23	31	47	209	403	739	800	481	250	166	128	102	86	70
	0.3	0	0.1		20	28	41	118	235	447	676	676	459	283	196	146	114	80
	0.3	0.1	0.1		19	26	39	99	189	361	571	641	520	362	251	181	136	89
	0.5	0	0.3		0	0	0	1	9	53	157	314	433	439	379	299	237	159
					Composite Discharges (CFS)													
1	0.3	0.1	0.1	0.06	1.140	1.560	2.340	5.940	11.340	21.660	34.260	38.460	31.200	21.720	15.060	10.860	8.160	5.340
2	0.3	0.1	0.1	0.082	1.558	2.132	3.198	8.118	15.498	29.602	46.822	52.562	42.640	29.684	20.582	14.842	11.152	7.298
3	0.3	0	0.1	0.037	0.004	1.036	1.517	4.366	8.695	16.539	25.012	25.012	16.983	10.471	7.252	5.402	4.218	2.960
4	0.2	0	0.1	0.017	0.391	0.527	0.799	3.553	6.851	12.563	13.600	8.177	4.250	2.822	2.176	1.734	1.462	1.190
5	0.3	0	0.1	0.032	0.003	0.896	1.312	3.776	7.520	14.304	21.632	21.632	14.688	9.056	6.272	4.672	3.648	2.560
6	0.5	0	0.3	0.008	0.000	0.000	0.000	0.008	0.072	0.424	1.256	2.512	3.464	3.512	3.032	2.392	1.896	1.272
	composite				3.096	6.151	9.166	25.761	49.976	95.092	142.582	148.355	113.225	77.265	54.374	39.902	30.536	20.620

Peak Flow = 148

Step 9. Develop a preliminary design to limit 2-year, 24-hour peak.

First make an estimate of the runoff volume to be detained in order to limit the outflow of the 2-year, 24-hour storm from the developed area. A good first estimate can be derived by using Chapter 6 of the TR-55 manual (USDA-SCS,1986). Using that reference, the three components needed to make an estimate are: 1) the peak runoff for the existing condition; 2) the peak runoff for the developed condition; and 3) the runoff volume for the developed condition.

From Step 7

Peak flow in the existing condition = 39.3 cfs (table 18)

Peak flow in the developed condition = 148.4 cfs (table 21)

Estimated volume of runoff is = [(36.1 acres x 1.06 inches) + (33.5 acres x 1.57 inches) + (16.1 acres x 1.49 inches) + (10.0 acres x 1.08) + (21.2 acres x 0.96 inches) + (9.1 acres x 0.59 inches)] [(43560 sq. ft./acre) x (1 ft./ 12 inches)]

= 549,480 cubic feet

The estimated storage volume for the 2-year peak flow control is found using equation 6-2 and figure 6.1 from Chapter 6 of the TR-55 manual.

Eq. 6-2; $V_s = V_r(V_s / V_r)$

Where :

V_s = estimated storage volume max. stage

V_r = runoff volume - developed condition

(V_s/V_r) = a coefficient taken from figure 6-1 and determined by the ratio of peak flow existing over peak flow developed

Peak flow existing/peak flow developed = 39.3 cfs/148.4 cfs = 0.26

from TR-55, figure 6.1: $(V_s/V_r) = 4.2$

V_s = (549,480 cu. ft.) x (0.44) = 230,800 cu. ft.

Estimated storage volume for the 2-year peak = 230,800 cu.ft.

Using interpolation to determine the height of the weir outlet we have from table 5:

$$\frac{856.0 \text{ ft} - \text{Yft.}}{856.0 \text{ ft} - 854.0 \text{ ft.}} = \frac{328.550 \text{ cu.ft.} - 230{,}800 \text{ cu.ft.}}{328.550 \text{ cu.ft.} - 214{,}300 \text{ cu.ft.}}$$

Y = 854.08 feet, or about 854.1 feet

Use 854.1 feet, or a weir stage height of 4.1 feet. The maximum peak flow limit is 39.4 cfs. We now need to devise an outlet that will go to the height of 854.1 feet. Keeping in mind the first outlet characteristics and that the storage volume to the top of the weir to control 1.5-inch runoff volume is 162,299 cubic feet. We have a discharge of 13.74 cfs. at 1.8 feet of stage height. If we extend the weir up 4.1 feet with no changes in side slope the discharge is:

$Q(out) = 5.69\ H^{1.5} = 5.69\ (4.1)^{1.5} =$ 47.23 cfs

which is slightly more than the allowable discharge, but because it is within the acceptable range of peak flow, it should be considered.

Step 10. Route the 2-year, 24-hour rainfall hydrograph from the developed area through the basin.

Using this outlet design, and routing the 2-year, 24-hour hydrograph through the structure we have the resulting discharges and storage (see table 15).

The peak storage and discharge is at time 12.5 hours with a storage of approximately 162,000 cubic feet, and discharge of 16.8 cubic feet per second. These results indicate that peak discharge is much less than the 39.4 cfs that is maximum limit of flow for the 2-year, 24-hour storm. The height associated with 162,000 cubic feet of storage is approximately 852.0 feet.

Redesigning the outlet to accommodate 0.2 feet was, in the designer's opinion, not worth the complexity that it would add to the construction of the outlet.

Step 11. Design and route other flood flow control features

This design feature is not covered in this manual. The designer should consult other sources for this design feature.

Step 12. Assess BMP flow characteristics as it affects the watershed.

This design feature is not covered in this manual. The designer should consult other sources for this design feature.

Step 13. Design the BMP details.

See the following sections in this manual for individual practice design.

Table 23. Flow routing table for 2-foot weir, 2-year

row	(A) start of time increment (hours)	(B) starting inflow rate (cfs)	(C) average inflow rate (cfs)	(D) 0.5(i1+i2)ti inflow volume (cu. ft.)	(E) previous storage - previous incremental outflow (cu. ft.)	(F) final storage + final incremental outflow (cu. ft.)	(G) outflow rate (cfs)	(H) storage (cu. ft.)
1	10	0		-			0.00	0
2			1.55	5,580	0	5,580		
3	11.00	3.10					0.26	5,355
4			4.625	4,995	5,075	10,070		
5	11.30	6.15					0.47	9,664
6			7.66	8,273	9,158	17,431		
7	11.60	9.17					0.93	16,622
8			17.465	18,862	15,613	34,475		
9	11.90	25.76					2.27	32,513
10			29.575	42,588	15,277	57,865		
11	12.00	49.98					4.35	54,114
12			72.535	26,113	52,549	78,662		
13	12.10	95.09					6.28	73,231
14			118.835	42,781	70,969	113,750		
15	12.20	142.58					9.65	105,409
16			145.47	52,369	101,935	154,304		
17	12.30	148.36					13.59	142,566
18			130.79	47,084	137,675	184,760		
19	12.40	113.22					16.53	170,472
20			95.24	34,286	164,521	198,807		
21	12.50	77.26					17.89	183,352
22			65.815	23,693	176,913	200,606		
23	12.60	54.37					18.06	185,002
24			47.135	16,969	178,500	195,468		
25	12.70	39.9					17.56	180,291
26			35.22	12,679	173,968	186,647		
27	12.80	30.54					16.71	172,202
28			25.58	18,418	160,169	178,586		
29	13.00	20.62					15.93	164,817
30			18.55	13,356	153,344	166,700		

Summary

The method described here is devised to calculate the size of control practice structures to accomplish an 80% removal of total suspended solids on an annual basis and limit the peak flow from a drainage area to the peak flow in the pre-developed condition. The method achieves this by calculating the runoff volume, peak flow and hydrographs for the pre- and post-developed conditions. To simplify the procedure, it is necessary that conservatively sized structures be designed to accommodate a wide range of site conditions.

Designers are encouraged to use the SLAMM computer model and other models approved by local governments to design structures that are more economical. All structures must achieve 80% removal of the total suspended solids on an annual basis, and maintain exiting peak flow levels in receiving water bodies.

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The Wisconsin Storm Water Manual: Hydrology (G3691-2)

G3691-3

The Wisconsin Storm Water MANUAL

Infiltration Basins and Trenches

Infiltration structures provide runoff volume control because they detain the runoff, slowly releasing the water into the groundwater. When peak reduction is desired, storage is increased, and the outflow riser elevation and the release rate are controlled. By diverting a significant portion of the runoff into the soil, infiltration structures can recharge groundwater, augment low flows and preserve base flow in streams, protect downstream aquatic biota and help minimize erosion and flooding downstream. Infiltration structures are reasonably cost effective if they are located on permeable soils with the depth to groundwater and bedrock well below the bottom of the basin.

Pretreatment and infiltration basins and trenches should be designed for relatively frequent rainfall. Larger flows should bypass the infiltration basin by a separate pipe or overflow device. Studies of infiltration basin performance suggest that limiting the flow that basins receive and avoiding overload conditions will improve long term operation.

Pretreatment

The performance of infiltration structures depends on how much storm water is diverted to groundwater. Their ability to capture nutrients depends on the soil and the basin's detention volume. Infiltration structures should include provisions for pretreating the water to prevent premature clogging of the basin. The combination of pretreatment and infiltration removes the greatest amount of pollutants.

Significant disadvantages of infiltration structures are their potential for groundwater contamination and their tendency to lose effectiveness over time due to clogging. While metals and many nutrients are captured in the first foot or two of soil, some soluble pollutants travel much greater distances. Groundwater contamination problems can be minimized by pretreatment or diversion of some runoff water from the infiltration structure. Pretreatment can remove sediment, oil and grease, and is necessary to increase the life of the infiltration area by reducing surface clogging.

Recommended pretreatment options include presettling basins, sand filters, sediment sumps, biofiltration swales and vegetative filter strips. When contaminants cannot be removed by pretreatment, surface runoff should be diverted from the infiltration structure. Runoff sources that cause particular problems for infiltration basins include, but are not limited to:

- Sites with high pesticide or pathogen levels
- Construction site runoff due to high sediment loads
- Manufacturing and industrial sites because of high concentrations of soluble toxicants and soluble heavy metals
- Snowmelt runoff because of salts
- Combined sewer system overflows because of sewage contamination

Runoff from residential areas (rooftops and lawns) is considered the least polluted and, therefore, the safest runoff for discharge to infiltration structures and eventual return to groundwater. An economic advantage to infiltration of runoff from low to medium density residential areas is that it requires less pretreatment prior to infiltration, provided care has been taken in the use of fertilizers and pesticides on lawns.

Pollutant removal

Table 1 shows estimates of typical pollutant removal rates for basins and trenches based on field tests of similarly designed, rapid infiltration, land treatment systems built for wastewater applications.

Table 1. Typical pollutant removal rates for infiltration basins and trenches

Pollutant	Removal rate
Sediment	99%
Total P	65–75%
Total N	60–70%
Trace Metals	95–99%
BOD	90%
Bacteria	98%

(Schueler, 1987)

This information assumes pretreatment and infiltration of 90% of the design flow. Soluble and fine particulate pollutants are removed in the soil through sorption, precipitation, trapping, straining and bacterial degradation or transformation. Trace metals are usually captured with the sediment in the first one or two feet of soil. Phosphorus removal can be as high as 70–99% given optimum physical and chemical soil characteristics. Nitrification is essentially complete in the soil, and nitrate removal depends on the presence of a carbon source to encourage denitrification. With effective denitrification, nitrogen removal can be as high as 80% (US-EPA, 1981).

An infiltration basin or trench will not increase temperature or reduce dissolved oxygen concentrations in storm water because flows are not held for long periods of time, and the water cools as it travels through the soil. However, nitrate, chloride, gasoline and other heavier, less-volatile, very-soluble hydrocarbons may eventually migrate into the groundwater.

Recommended storm water quality monitoring to evaluate potential groundwater contamination

Urban runoff contaminants with the potential to adversely affect groundwater

- Nutrients (especially nitrates)
- Salts (especially chloride)
- Volatile organic compounds, or VOCs. If these are expected in the runoff (such as that from manufacturing, industrial or vehicle service areas) screen for VOCs with purgeable organic carbon analyses
- Pathogens (especially enteroviruses, along with other pathogens such as Pseudomonas aeruginosa, Shigella, and pathogenic protozoa)
- Bromide and total organic carbon (estimates disinfection byproduct generation potential if disinfection by either chlorination or ozone is being considered.)
- Pesticides, in both filterable and total sample components (especially lindane and chlordane)
- Other organics, in both filterable and total sample components (especially 1,3 dichlorobenzene, pyrene, fluoranthene, benzo(a)anthracene, bis(2-ethylhexyl) phthalate, pentachlorophenol, and phenanthrene)
- Heavy metals, in both filterable and total sample components (especially chromium, lead, nickel and zinc).

Urban runoff compounds with the potential to adversely affect infiltration operations

- Sodium, calcium and magnesium (allows calculation of the sodium adsorption ratio to predict clogging of clay soils).
- Suspended solids (to determine the need for sedimentation pretreatment to prevent clogging).

(Pitt, R., et al. 1994)

Site selection, proper design and construction, and a sustained maintenance program are critical to the life of infiltration structures. These structures may have fairly high failure rates and require frequent maintenance. A study of 12 infiltration basins in Maryland showed that all had failed within the first two years of operation (Galli, 1992). Reasons for failure were listed as:

- Poor site selection (especially separation distance to groundwater)
- Poor soil textures
- Clogging of the soil by contaminants in the runoff
- Compaction of the soil

None of the basins had built-in pretreatment systems. In addition, internal sediment loading from poorly stabilized side slopes was as much a problem as external sediment loading. Proper site selection, stabilization of the contributing area, and pretreatment to remove pollutants that can clog the infiltration structure bed will effectively minimize these problems.

Infiltration basins

An infiltration basin is an open impoundment created either by excavation or embankment with a flat, densely vegetated floor. It is situated on permeable soils and temporarily stores and allows a designated runoff volume to infiltrate the soil.

Constructing an infiltration basin is an effective management practice for converting surface runoff to groundwater recharge and for removing many nutrients and pollutants.

Planning guidelines

Feasibility study

Building an infiltration basin is an appropriate management practice when baseflow recharge or reduction of thermal impacts is a high priority for the watershed. Since soil properties are critical factors in designing infiltration basins, a preliminary screening of potential sites is necessary. The feasibility study should begin by examining any available local, county or U.S. Natural Resources Conservation Service (NRCS) soil surveys and maps. These reports will identify areas where the soil textures may meet the infiltration requirements, and may also provide information on the depth to groundwater and bedrock. If information is available, check for slopes in the area and to see if the area contains fill material. This information should be used only for screening, since some surveys are dated and land practices might have resulted in erosion or compaction of soils. Also, such surveys do not provide the detail needed to site infiltration practices.

Soils with shallow groundwater or fractured bedrock, sandy soils with low adsorption rates and high infiltration rates, areas with high loadings of soluble pollutants, and areas where the groundwater is a critical resource that must be protected from contamination usually are not suitable for infiltration structures (Davenport, 1991). In areas where groundwater quality impacts are especially critical, consideration should be given to a greater than five-foot separation distance from the bottom of the basin and high groundwater to minimize the effect of seepage from the pond.

A hydrogeologic investigation should be conducted before designing an infiltration basin to determine the:

- Depth to high groundwater
- Groundwater flow direction and rate of flow
- Vertical and horizontal gradients
- Presence and extent of perched groundwater
- Soil descriptions
- In-field infiltration rates
- Depth to bedrock
- Type of bedrock.

Delineation of the saturated and unsaturated soil zones is important because these zones use different pollutant removal mechanisms. The critical factor in protection of the groundwater is how well the unsaturated zone removes pollutants and prevents their migration through the soil.

Soil properties

Once a potential site is located, soil borings or test pits are required to confirm preliminary findings. For design purposes, the engineer must determine site-specific soil properties by laboratory and field tests at the proposed location. In-field investigation at the basin site should be completed to depths sufficient to document that the distance to high groundwater and bedrock is at least five feet from the bottom of the basin or greater if dictated by local ordinance. Investigators in the Maryland study of 12 infiltration basins suggested a separation distance of at least 15 feet (Galli, 1992). While a 15-foot minimum distance is probably not justified in many cases, it illustrates the importance of adequate separation to protect groundwater quality.

An in-field, double-ring infiltrometer test is the preferred method for gathering information on site suitability (ASTM, 1994). The test must be done at the depth of the proposed infiltration basin bottom, which may not be the current ground surface. The number of tests conducted depends on the site's size and uniformity. A minimum of three tests is recommended. To ensure that the basin is not undersized, design infiltration rates must be conservative. Over the years the infiltration rate may decrease, but pretreatment and a conservative design will help extend the basin's life.

Soil permeabilities must be at least 0.5 in/hr and at most 5.0 in/hr in the field. This restricts application to soils of Hydrologic Soil Group B, and some soils in groups A and C. Hydrologic soil groupings are available from the NRCS (USDA-SCS, 1975). Type C soils will provide very slow infiltration but maximum treatment due to the higher percent fines and greater adsorptive capacity.

Soils with more than 30% clay are not suitable because of their low infiltration rates; soils with 40% silt and clay are prone to frost heave and should not be used. High clay soils have a tendency to develop vertical fractures and channels, bypassing treatment of the storm water. Type A soils may provide rapid infiltration but minimal treatment, since sand acts like a sieve and does not bind pollutants.

In the interest of providing treatment, the soils should contain at least 5% fines. This increases the adsorptive capability of the soil. Soils of choice include loamy sand, sandy loam, loam and silt loam. The existence of an impermeable layer in the soil profile may interfere with optimum basin operation. In some cases this layer may be removed during construction, but often such areas must be avoided.

Other considerations

Infiltration basins are commonly used for drainage areas of 5–50 acres with land slopes of less than 20%. Steep slopes can cause water leakage in the lower levels and may reduce infiltration rates due to lateral movement. The basin itself should be located more than 50 feet from slopes greater than 20%. Basins must not be located on fill materials or on soils compacted by construction. Compaction reduces the infiltration rate and may make fill materials unstable. Slippage may occur along the interface of fill and in-situ soils which could be further aggravated by saturated conditions.

Design guidelines

Infiltration basins are usually irregularly shaped, elongated impoundments with vegetated or riprapped inflow and outflow areas. The typical depth of a basin is 3–12 ft, with the maximum depth dependent on the soil type. Basins should be designed to hold and allow infiltration of the water in a dead storage zone, to hold and infiltrate water from the design storm, and to safely pass through, or preferably bypass, flows up to the level produced by the 24-hour, 100-year storm.

From the standpoint of water quality, the optimum infiltration basin is an off-line impoundment in soils with an adequate infiltration rate. The grass cover and underlying soils must have sufficient organic matter and root systems to bind, decompose and trap pollutants. Finally, such a basin must be large enough to remain aerobic. In some cases, a facility may be built in combination with another treatment structure.

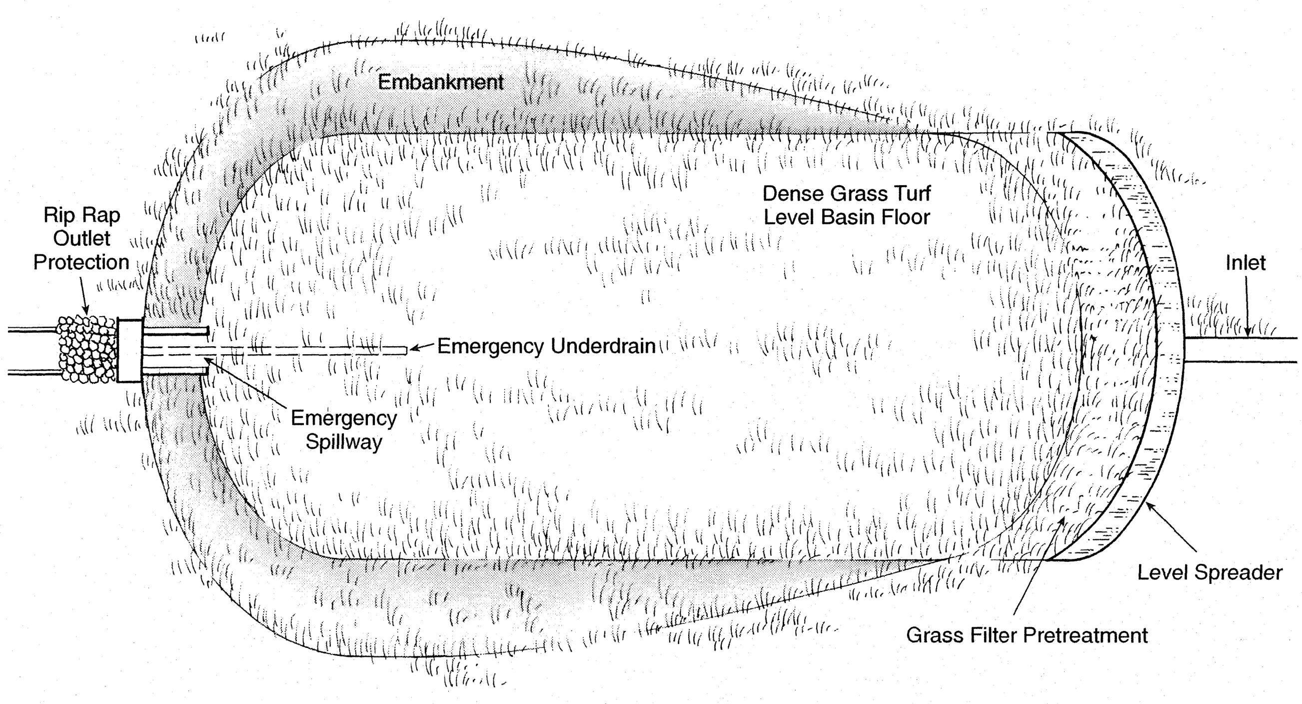
A common configuration for an infiltration facility is shown in figure 1. The detention basin can precede or be a part of the infiltration basin. Pretreatment to remove sediments that might clog the infiltration bed is critical to maintaining infiltration. Sediment is usually trapped either in a separate pretreatment structure or in a sediment bay of the infiltration basin. A riser in the combined infiltration/detention basin drains flows above the water quality volume.

Infiltration basins are not sediment control devices. The size and location of the infiltration basin must be adjusted to provide for removal of most particulates in a pretreatment unit.

Pretreatment is a requirement for all infiltration basins that receive any storm water containing particulate matter or pollutants that might clog the infiltration structure or leach to groundwater. Some modification or downsizing of the infiltration structure may be expected when a unit capable of full treatment is used for pretreatment only.

Access to the pretreatment facility is necessary for frequent cleaning and removal of sediment build-up. If oil and grease are contained in the runoff from the watershed, an oil and grit separator, oil and water separator, floating skimmer or filter should be a pretreat-

Figure 1. Schematic of an infiltration basin.



ment component. To encourage uniform use of the infiltration basin and to prevent channeling on the basin floor, a grass filter or level spreader should be used to create sheet flow across the basin floor.

Effective infiltration basin design will include the following features.

Site evaluation

A minimum of three soil borings should be conducted at each basin site with more required (at a rate of one per 5,000 square feet of infiltrating surface area) for larger basins or for basins with varying soil types. The soil tests must establish a minimum infiltration rate of 0.5 in/hr, a maximum of 5.0 in/hr, and a minimum separation distance to bedrock and seasonal high groundwater of 5 feet from the proposed bottom of the basin. Separation distances to seasonal high groundwater should be confirmed by looking at the static water elevation in the soil boring, changes in the soil moisture content, and soil mottling. The design infiltration rate should be based on in-field infiltration testing. With the inconsistency of soil testing and permeabilities, a safety factor of at least two is recommended for determining basin size. It is recommended that the engineer use half the measured infiltration rate as the design infiltration rate (WA-DOE, 1992). The more conservative the design rate, the longer the life of the infiltration basin. The Washington State storm water design manual recommends a minimum cation exchange capacity (CEC) of 5 milliequivalent/100 grams of dry soil to provide adequate treatment levels.

Since storm water can carry pollutants similar to pollutants found in wastewater or hazardous waste, minimum setback distances from private water supply wells must be 100 feet, and 1,200 feet for public wells. If it is desirable to locate a facility within 1,200 feet of a public water supply, a study of the groundwater flow in the area should be made to determine the site's pollution potential. In Wisconsin, all new municipal water supply wells (installed after April 1992) must have a Wellhead Protection Plan that governs separation distances to the well. In some cases this distance may be greater than 1,200 feet. Basins must be located at least 10 feet downslope and 100 feet upslope from building foundations to prevent the foundations from settling and basements from flooding. The engineer should consider an even more conservative setback distance if large quantities of storm water are reaching the subsoil.

Infiltration structures must not be located in the floodplain and must meet all other applicable state and federal requirements. Embankments may be subject to dam construction regulations.

Watershed size

Infiltration basins designed solely for water quality control are appropriate for watershed areas of 5 to 25 acres. For combination basins (detention and infiltration), up to 50 acres is typical, although larger areas may be considered. If more than half of a given watershed is impervious, an infiltration basin might not be an appropriate application, because the amount of flow will be large and the space required for infiltration might not be available.

Infiltration time

The water quality infiltration volume must be equal to the runoff volume from the design-level storm plus the rainfall on the structure. To ensure adequate treatment of the stormwater in the soil for groundwater protection, infiltration should be completed in not less than 6 hours or more than 48 to 72 hours, depending on soil and vegetative conditions. This will help ensure adequate treatment of the storm water for groundwater protection, protect vegetation and avoid the possibility of anaerobic soil conditions. Effective operation includes both treatment and movement of the water out of the basin in time for the next storm. A load and rest operation will encourage aerobic soil conditions. Infiltration times as great as 72 hours may be used for infiltration basins on some hydrologic B soils and moisture tolerant vegetation.

Basin shape

All basins must be flat on the bottom with stable side slopes. Consider side slopes of 4:1 or flatter for ease of maintenance and safety. The basin shape can be any configuration that blends with the surrounding landscape. Groundwater mounding, or a raising of the water table elevation just under the basin floor, is common in infiltration systems. Groundwater mounding can restrict the amount of downward flow, reducing the infiltration rate. Less groundwater mounding will occur under a basin with a long, narrow configuration.

Vegetation

Plant a water-tolerant, fast-germinating, hardy grass on the bottom and side slopes. Mow to maintain a dense turf. Mow when the surface is dry to avoid rutting and compaction. Generally, fertilizers should not be applied. If fertilization appears necessary, conduct a soil test and apply fertilizer to match the nutrient needs indicated by the test.

Basin inlets

Erosion protection is required at the inlet. Riprap aprons or other energy dissipators help to reduce velocities and spread flows. A 20-foot filter strip with a level spreader will also provide sheet flow. The inlet should discharge at the basin floor.

Winter operation

When the soil freezes, infiltration may cease. While infiltration may occur under some frozen conditions, the basin cannot be depended upon to treat rain or snowmelt during the winter since the system will often be frozen.

Enhancing pollutant removal

An infiltration basin is designed in part to treat runoff to improve water quality. A number of design features will enhance pollutant removal rates.

- Large, shallow basins are more effective than small, deep configurations.
- Deep tilling after construction helps prevent compaction.
- Riprap at the inlet and outlet channels stabilizes velocities and spreads flows.
- During construction, flows from the watershed should be diverted around the proposed treatment site.
- The basin floor should be as flat as possible to ensure uniform ponding over the surface.
- All upland soils and the slopes on the sidewalls must be stable before the basin is placed in operation. Slopes of 4:1 or flatter should be considered for maintenance and safety considerations.
- The grass should not be cut below 3 inches or it will not survive flooding.
- Storm water should not be introduced into the basin until a dense, water tolerant grass sod is established in the basin.
- Hydrologic soil group B soils provide the optimum infiltration rates and treatment capabilities.
- A load and rest operation is important in maintaining the aerobic condition of the soil.
- Pretreatment will make the basin last longer and be more effective.
- While the basin can provide both quantity and quality control in one practice, separate, interconnected practices are more effective.
- The design should include an emergency drain to facilitate maintenance.
- Compactions during and after construction must be avoided. The basin should not be used for parking or as a recreational facility.

Basin buffer

A vegetative screen around the basin to restrict views from nearby properties may improve the aesthetics of the site and public acceptance of the facility. Mowing the basin regularly will prevent woody vegetation growth that might migrate in from the buffer area to the infiltration basin. Mechanical rather than chemical removal is recommended for undesirable plant invasion at the site. Removing the clippings will remove some nutrients from the basin; however, nutrients from clippings usually are quite small compared to the total load.

Access

A public right-of-way around the basin is necessary for maintenance access. The access route should not be constructed over the emergency spillway. Access is a topic that must be considered while the facility is being sited.

Safety

Fencing around the basin can serve as a safety feature if the intent is to deny public access to the basin. If the area around the facility has a recreational use, considerations should be given to construction of a safety shelf for times when the basin is flooded. Steep slopes should be avoided. Signs should warn against deep water or health risks. Provide an emergency spillway to safely bypass or move high flows through the basin to prevent structural failure. A spill or accident that results in harmful chemicals being flushed into an infiltration basin is a serious problem and could affect the basin's ability to treat and/or infiltrate storm water. If the basin serves an area where a spill could occur, it is critical to control the spill at its source to prevent it from draining to the storm water treatment facility.

Storage

Storage volume for runoff from design storms and for precipitation directly on the basin should be calculated. Storage depth will be limited by the infiltration characteristics of the soil as described in the section on design calculations.

Design calculations

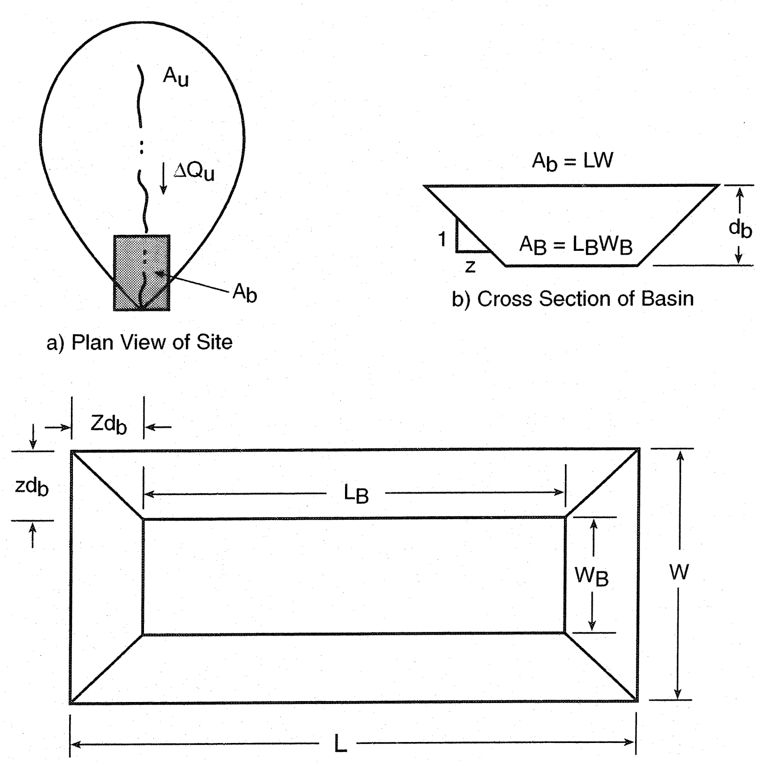
The size of the basin depends on the infiltration rate of the soil and on the volume of runoff from the tributary area. A rough estimate for determining the basin area is that the infiltration surface area should be greater than half of the contributing impervious surface (Stahre and Urbonas, 1990). To determine the design dimensions of the basin, a hydrologic analysis of the contributing watershed must be conducted to predict the runoff from the design storm using small storm hydrology. The storage volume can then be calculated given the infiltration rate for the basin area and the desired infiltration time.

Some designers take into account the infiltration rate through the sides of the basin at ⅓ the rate through the bottom. In most cases, the volume infiltrated through the sides is a relatively small portion of the total water infiltrated and can be neglected.

The following design calculations assume infiltration only through the bottom surface area. This provides an additional design safety factor. For illustration purposes, a trapezoidal infiltration basin is assumed. Three design relationships must be considered:

1. Storage volume
2. Maximum basin depth
3. Basin volume.

Figure 2. Schematic of basin nonmenclature



a) Plan View of Site

b) Cross Section of Basin

c) Top View of Basin

Storage volume

The average end-area equation can be used to estimate the storage volume of the infiltration basin. For a rectangular basin, as illustrated in figure 2, this equation can be written as:

$V_W = ((A_B + A_b)/2)(d) = (LW + L_BW_B)/2)(d)$

where V_W = the basin volume

A_b = the water surface area at the design depth

A_B = the bottom surface area

d = design depth

L = the top basin length

W = the top basin width

$L_B = L - 2zd$ = the bottom length

$W_B = W - 2zd$ = the bottom width

z = horizontal component of the side slopes.

Maximum basin depth

The maximum depth (d_m) can be determined by multiplying the design infiltration rate (f) times the maximum allowed ponding time (T_p).

$d_m = fT_p$

Basin volume

The required capacity may be determined as the design runoff from the upland area plus direct precipitation on the basin surface minus the infiltration from the basin during the runoff event. The volume equation can then be written as:

$V_W = QA_u + PA_b - fTA_B$

where A_u = area of the upland watershed

Q = the upland runoff depth

P = the design precipitation

T = the effective runoff time.

T is a small number (commonly 1 to 2 hours based on engineering judgement) since it reflects only the time when the inflow exceeds the outflow (in this case outflow by infiltration into the soil). In fact, fTA_b may be so small in relation to the amount of runoff and rainfall that it can be eliminated from the equation without significant error.

Additional design considerations

Basin design based on the three equations on page 7 must also consider site conditions and owner preferences. For example, if area is limited, it is desirable to maximize the basin's depth. If sufficient area is available a smaller design depth may be desirable to reduce ponding time, improve safety or reduce cost. A long, narrow basin generally improves infiltration, and may influence the selection of a length to width ratio. The side slope steepness will depend on maintenance practices and safety issues.

The maximum depth should be calculated and serve as the upper limit for the design depth. The required storage volume must also be calculated from the design storm and the watershed characteristics. The side slope steepness must be selected. The length or width of the basin is set and the equation solved for the remaining dimension. Double-check that L and W are greater than $2Zd_b$, and that the bottom elevation of the basin is at least 5 feet above seasonal high groundwater. Adjust L, W and d until the desired basis configuration is achieved.

Example

To see how these calculations and considerations would be applied in a real-world situation, assume the following conditions for the design of a rectangular infiltration basin:

Design rainfall = 1 inch

Design runoff depth from contributing area = 0.5 inch

Runoff contributing area = 3 acres

Runoff time = 2 hours

Design infiltration rate = 0.75 inches per hour

Maximum infiltration time = 48 hours

Owner prefers that basin be not more than 2 feet deep, that the length not exceed 100 feet, and the side slopes to be 4:1 for safety and maintenance.

What should be the width of the basin?

Step 1. Check to make sure the 2-ft depth is less than the maximum depth for the basin.

$d_m = fT_p = 0.75$ in/hr x 48 hrs = 36 inches = 3 feet

Therefore, 2-ft design depth is within acceptable limits.

Step 2. Calculate storage volume required.

$V_W = QA_u + PA_b - fTA_b$ = 0.5 in x 3 acres + 1 in x A_b - 0.75 in/hr x 2 hr x A_b

In this example it is assumed that the area of the basin receiving rainfall and the infiltration area are the same. For shallow depths this approximation will not cause design problems. For deep basins a distinction should be made between the two areas.

Step 3. Determine the required dimensions for the infiltration area.

$V_W = ((LW + L_BW_B)/2) \times d$

$QA_u + PA_b - fTAb = ((LW + L_BW_B)/2) \times d$

For d = 2 ft, L = 100 ft, and z = 4 the above expression may be written as:

0.5 inch x 3 acres x 43,560 square feet/acre x 0.083 feet/inch + 1 inch x W x 100 feet x 0.083 feet/inch - 0.75 inch/hour x 2 hour x W x 100 feet x 0.083 feet/inch = ((100 x W) + (84 x (W-16))/2) x 2   W = 37 feet

Construction guidelines

Infiltration basins usually fail for one or more of the following reasons:

- Premature clogging
- A design infiltration rate greater than the actual infiltration rates
- Because the basin site was used for construction site erosion control
- Soil was compacted during construction
- The upland soils or basin walls were not stabilized with vegetation, and sediment was delivered to the basin.

Note that all these failures result from improper planning, design or construction.

If the infiltration basin is to operate effectively, special care must be taken before construction begins. The development plan sheets should list the proper construction sequence so that the basin site is protected during construction and not placed in operation until upland areas are stabilized. All heavy equipment, sediment and runoff must be diverted away from the basin site during construction in the watershed. To avoid soil compaction, the site intended for the basin should not be used while construction proceeds in the watershed. If a temporary basin for construction site erosion control is to be used, it should be located outside the perimeter of the final infiltration basin. If the basin site must be used, all accumulated sediment plus two additional feet should be excavated to ensure that the surface is not clogged.

Excavate the basin during dry periods, using only light earth-moving equipment or over-sized tires. If feasible, excavate from the sides so all equipment will be kept off the basin's floor. Avoid using bulldozers and end loaders. The site should be deep-tilled and leveled after excavation. Engineering standards, such as NRCS *Technical Guide Practice 378* for embankment construction, must be followed. (USDA-SCS, 1987)

Seed vegetation shortly after construction (USDA-SCS *Technical Guide Practice 342*) for a low-maintenance, fast-germinating, stoloniferous grass. Non-grass species such as sedges and forbes may also be acceptable. Highly invasive plants such as reed canary grass or creeping red fescues are not recommended. Plant species native to Wisconsin are biologically and aesthetically more valuable than non-native species and may provide a longer-lived, stable system for infiltration. The following native species are recommended (Trochlell, 1994):

- Canada bluejoint grass (*Calamagrostis canadensis*)
- Prairie cordgrass (*Spartina pectinata*)
- Woolgrass (*Scirpus cyprinus*)
- Rice cutgrass (*Leersia oryzoides*)

Native species should be purchased from reputable plant nurseries that have collected the seeds from the local region (within 100 miles) when possible.

During early growth, check the vegetation and reseed or irrigate as necessary. If a dense mat does not develop, consider a different seed mixture. A dense grass mat has two primary benefits for the infiltration basin: 1) the roots will help maintain infiltration capacity; and 2) the grass will hold the sediment and decrease resuspension during high inflow velocities.

Native plantings should not be fertilized because fertilizers tend to encourage weeds. Also, it might take up to 2 years to establish native grasses; during this time the plants might appear sparse while their root systems develop. Planting a top-cover of annual rye or oats is a good way to give native grasses time to grow while maintaining ground cover. Fertilization, if needed for non-native grasses, must be carefully controlled to minimize phosphorus loading to the receiving stream or lake or nitrate leaching to groundwater.

Maintenance

An infiltration basin is a high maintenance facility. A storm water management plan must include maintenance, inspection, access and enforcement of the basin's operating requirements or the system will fail (Lindsey et al., 1992). Identify the party responsible for maintenance early in the planning process, and provide funding for routine and non-routine maintenance. An operation and maintenance manual should be written before the basin is put into operation. Following construction, inspect the basin monthly, as well as after every major storm, to see if the basin is draining within the design time limits. If it is not, evaluate and repair the facility in accordance with the installation performance bond or construction agreement with the contractor.

Inspect annually or seminannally for settling, cracking, erosion, leakage, tree growth on the embankment, the condition of the inlet and outlet channels, sediment accumulation in the basin, and the health and density of the grass turf. Always check a facility after large storms to correct any damage high flows may have caused. Eroded areas should be revegetated immediately.

The basin should be mowed twice a year to prevent woody growth, stimulate grass growth and enhance nutrient removal. Do not mow when the ground is wet to avoid compacting the soil and matting the grass. Also remove any trash or debris at this time. If the surrounding site has recreational value, more frequent mowing will be necessary.

If the soils are marginal for infiltration and the basin is prone to ponding, periodic tilling and reseeding might be needed. If this is the case, till and revegetate in the late summer.

Over time, an infiltration basin is likely to accumulate sediment and the infiltration rate might decrease. Deep tilling, regrading and replanting will help restore the original infiltration rate. When the basin is thoroughly dry, remove the top cracked layer of sediment, and till and grade the remaining soil. Some basins have a 6- to 12-inch layer of sand on the bottom or a filter fabric to facilitate sediment removal.

In a vegetated basin, sedimentation must not occur faster than the grass can grow through it. If it does, the pretreatment system should be re-evaluated. Maintenance of the pretreatment facility, including sediment removal, oil and grease skimming and mowing of the grass filter strip must occur on a regular schedule to prevent these materials from washing into the infiltration basin. An emergency drain built in the basin will allow for easier maintenance. In general, the lifetime of a pretreatment or inlet/bypass structure might be shorter than the lifetime of the infiltration basin itself, and will require occasional structural or equipment repair or replacement.

Infiltration trenches

An infiltration trench is an excavation, 2–10 feet deep, often lined with a sand base, a protective layer of filter fabric on the sides, and filled with coarse stone aggregate (figure 3). The empty spaces between the stone provide temporary storage of runoff, with the runoff making its final infiltration through the undisturbed subsoils at the bottom of the trench. The top layer of the trench may be a stone, gabion, sand or topsoil with a vegetative cover with or without an inlet. Sometimes trenches are located beneath grass swales. Infiltration trenches are appropriate in small drainage areas such as residential lots, commercial areas, parking lots and open space.

Place infiltration trenches on permeable soils, with a 5-foot separation distance from the bottom of the aggregate to seasonal high groundwater and/or bedrock. If they are sited correctly, infiltration trenches can recharge groundwater, control runoff volume and augment low flow for headwater streams. Depending on their size, these trenches are able to divert up to 90% of the annual runoff volume into the soil.

Trenches are most effective when used for storm water runoff from small to moderately sized storms. Trenches can help prevent localized streambank erosion on small streams by reducing the runoff rate, but they are generally too small to have a significant impact on larger streams. An additional advantage of infiltration trenches is that they fit easily into non-utilized areas, perimeters and margins of a developing site or in-fill development. The disadvantages of trenches are similar to infiltration basins—they clog easily, can be a threat to groundwater and require regular maintenance.

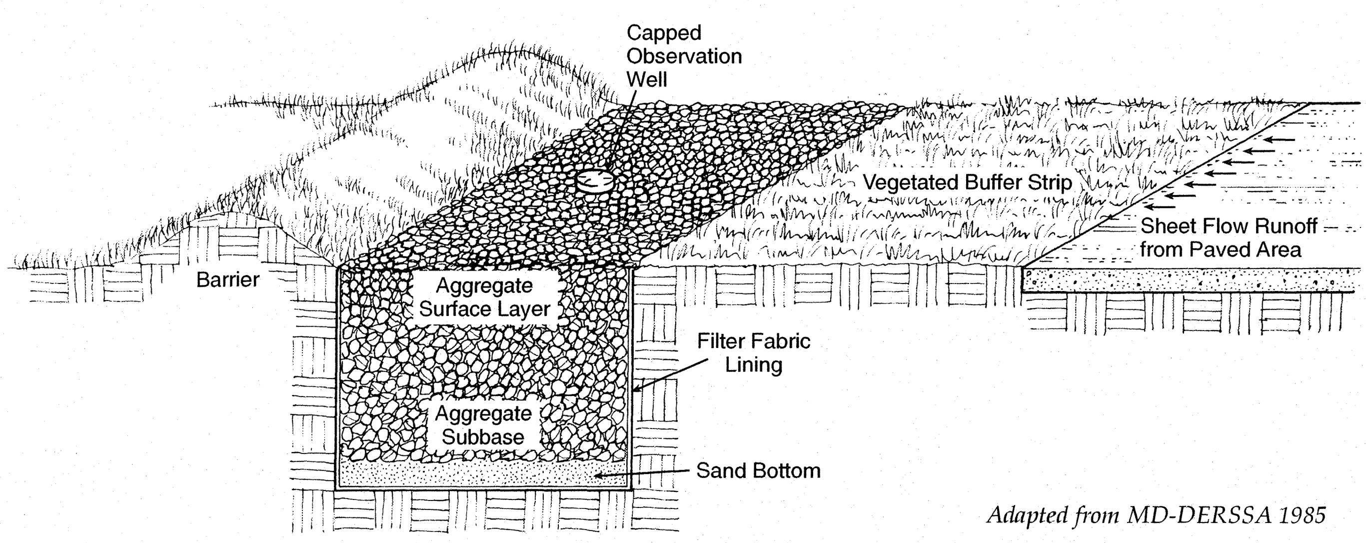
Infiltration trenches return runoff to the groundwater. They can be sized to provide volume and/or water quality control by storing and infiltrating all flows equal to or less than the design water quality volume. Higher flows will pass through or be diverted from the system via an overflow channel.

Planning guidelines

In determining the suitability of a given site for an infiltration trench, several factors must be considered, including separation distances, the size of the tributary area and the physical constraints of the site. Because site suitability is a critical issue in locating trenches, the soils in the area should be screened for adequate permeability, slope, depth to groundwater and depth to bedrock. Local soils maps and survey information are available from NRCS. Actual infiltration rates must be determined through field tests.

Trenches should not be located where the watershed slopes are 20% or greater. Slopes less than 5% are preferred. A trench should be located at least 100 feet from a private water supply well and 1,200 feet from a public well. Some municipalities might have established wellhead protection areas using a calculated fixed radius greater than 1,200 feet. No infiltration structure can be constructed within a wellhead protection area. Contact local officials for other restrictions on locating near public wells.

Figure 3. Typical trench configuration



Adapted from MD-DERSSA 1985

Trenches should be at least 10 feet down-slope and 100 feet up-slope from a building foundation to reduce the potential for wet basements and saturated soils around structures. Designers should consider a more conservative building separation if the amount of water coming to the trench is substantial. Trenches generally serve developments smaller than 5 acres but could be considered for 5- to 15-acre sites. In the case of large tributary areas, the larger area may be divided into subareas with individual trenches.

Design guidelines

Soils investigation

A hydrogeologic investigation should be conducted prior to design of the infiltration trench to determine the following:

- Depth to high groundwater
- Groundwater flow rate and direction
- Vertical and horizontal gradients
- Presence and extent of perched groundwater
- Soil descriptions
- In-field infiltration rates
- Depth to bedrock
- Type of bedrock

Hydrologic Soil Groups A, B and some C soils can be considered for an infiltration trench if the measured infiltration rate is at least 0.5 in/hr and less than 5 in/hr. Hydrologic soil groupings are available from the Natural Resources Conservation Service (USDA-SCS, 1975). This includes some sands, loamy sand, sandy loam, loam and silty loam. Sand with at least 5% silt or clay is necessary to provide treatment in the soil. A maximum infiltration rate of 5.0 in/hr is also recommended to protect the groundwater from pollutants which may not be filtered by soils with rapid permeability. Soils with more than 30% clay or 40% combined silt and clay may not be suitable, due to frost heave. The bottom of the trench must be below the frost line for successful operation in the winter.

Clay lenses or other restrictive layers below the bottom of the trench will reduce infiltration rates unless excavated. Trenches must not be located on fill material due to its unstable condition and the potential for movement at the interface between the fill and in-situ soils.

Design storm

Local regulations should be consulted to determine the design storm return period and duration. Generally, a storm that occurs relatively frequently is used as the model for trench design. In Wisconsin, capacity to handle the runoff from a 1.5-inch storm is recommended (WI-DNR, 1997). Additional storage may be needed if peak flow control is to be included in the design. A trench can be built in combination with another facility to meet water quality and quantity requirements.

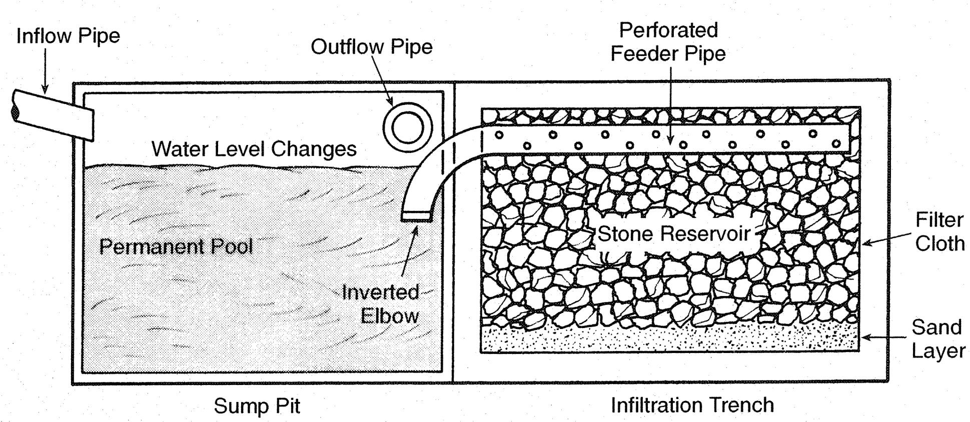
Alternatively, an outlet elevation can be set to store and allow infiltration of flows less than or equal to the water quality volume, while using the outlet for slow release of flows above that volume.

Pretreatment

To prevent clogging, sediment, oil, grease, floatable organic materials and solids capable of settling must be removed before the runoff enters the infiltration trench. If the trench has a surface inlet, the system must be designed to capture sediment either through a vegetated filter strip, grass swale or mechanical sediment trap such as a sump pit (figure 4). A sand filter system or oil/grit separator should be considered for oil and grease removal.

In a Maryland study, trenches with sump pit pretreatment lasted longer than trenches with grass filter strips for pretreatment (Galli, 1992). The sump

Figure 4. A sump pit



Adapted from Galli, 1992

pit, shown in Figure 4, captures coarse-grained, inorganic sediment, some fines, and large organic matter. Oil and grease can be trapped if the intake elbow to the trench is located about one foot below the permanent pool elevation of the sump pit. Scouring and resuspension of solids will occur if the sump pit is not cleaned frequently.

Depth to groundwater and bedrock

Soil borings or test pits are needed to establish that the depth to seasonally high groundwater and bedrock is at least 5 feet below the proposed bottom of the trench. The bottom of the trench is defined as the surface at the top of the native soil where infiltration will occur. The 5-foot separation distance exists to allow treatment of the storm water as it travels through the soil. This reduces the potential for groundwater contamination, and prevents long term soil saturation due to groundwater mounding at the bottom of the trench.

Storage volume

The design storage volume depends on the runoff from the design storm, the infiltration rate for the soil, and the porosity of the rock storage. A stone aggregate of clean, washed gravel, 1.5 to 3.0 inches in diameter, has a porosity of 30–40%. Since infiltration tests are the least precise measure used in the design calculations, the infiltration trench should be oversized to account for the uncertainty. Use half the measured infiltration rate to provide a safety factor of two for sizing (WA-DOE, 1992).

Configuration

Infiltration trenches can be constructed in a variety of configurations, with a rectangular cross-section being the most common. The primary variation is the method by which the runoff is introduced into the trench. Infiltration trenches can be built as surface trenches into which water either infiltrates through a layer of topsoil about one foot thick or they can be built directly into the rock fill. They may also have an inlet grate for overland flow of runoff into the trench. Finally, there can be underground inlets that allow runoff to reach the trench through a sub-surface pipe. Underground systems are not visible at the surface other than for the observation wells. Underground systems, with storm water entering through a piping system must not be designed as injection wells as defined by EPA regulations.

The bottom slope of a trench should be flat across its length and width to evenly distribute flows and encourage uniform infiltration through the bottom. A series of trenches rather than one long trench will provide a better flow pattern. This configuration also reduces the rate of clogging, since the first trench will receive and trap the heaviest sediment loads. Easy maintenance access must also be built into the design.

> At one time it was common practice to install drywells or french drains for disposal of storm water. While these practices have some characteristics in common with infiltration trenches, they must be avoided. There is a concern that some trenches using perforated piping to direct storm water underground meet the U.S. Environmental Protection Agency (EPA) definition of a Class V injection well. Injection wells for disposal of pollutants are prohibited under NR 812.05 Wis. Adm Code. A trench could be considered an injection well if it is deeper than it is wide.

Drain times and trench depth

The trench should completely drain in 48 to 72 hours. The depth of water in the trench that will allow drainage within 72 hours is dictated by the soil's infiltration rate. Trenches are usually less than 10 feet deep, and depths less than 8 feet allow for easier maintenance. Trench dimensions can be varied to accommodate depth limitations.

Filter fabric

The infiltration trench should be lined on the sides and top by an appropriate geotextile fabric. The top layer of fabric is located 1 foot below the top of the trench and serves to prevent surface sediment from passing into the stone aggregate. Since this top layer serves as a sediment barrier, it will need to be replaced more frequently and should be readily separable from the side sections.

Filter fabric can be placed on the bottom of the trench, but it is better to use a 6-inch layer of clean, washed sand. Clogging often occurs at the filter fabric layer, and sand restricts downward flow less than fabric. The sand also encourages drainage and prevents compaction of the native soil while the stone aggregate is added.

Aggregate material

The stone aggregate in the trench should be washed, bank-run gravel. This material is least likely to cause clogging by dust from the stone, which can fill the void spaces or settle to the bottom. If a crushed rock is used, it must be thoroughly washed to minimize dust problems.

Overflow

A diversion path rather than an emergency spillway should be used to pass excess flows over the trench to a waterway. The path must be constructed to prevent erosion from concentrated, uncontrolled flows.

Observation wells

Trenches must have observation wells to determine how quickly the trench drains after a storm and to observe the sediment build-up in the bottom. The well should be constructed as indicated in figure 5. The observation well should be a perforated PVC pipe, 4- to 6- inches in diameter, extending to the bottom of the trench where it is connected to a foot plate. Cap and lock it to prevent vandalism or tampering.

Vegetation

If the trench is covered with native top soils and planted in grass it will be similar to any other greenway in a developed area. If not covered, the stone aggregate will be visible. A vegetated buffer strip 20–25 feet wide on either side of the trench will help protect it from sediment build-up. The buffer should be stabilized prior to placing the trench in operation. The trench and buffer vegetation should blend in with the surrounding area; native grasses are preferable if compatible with the area.

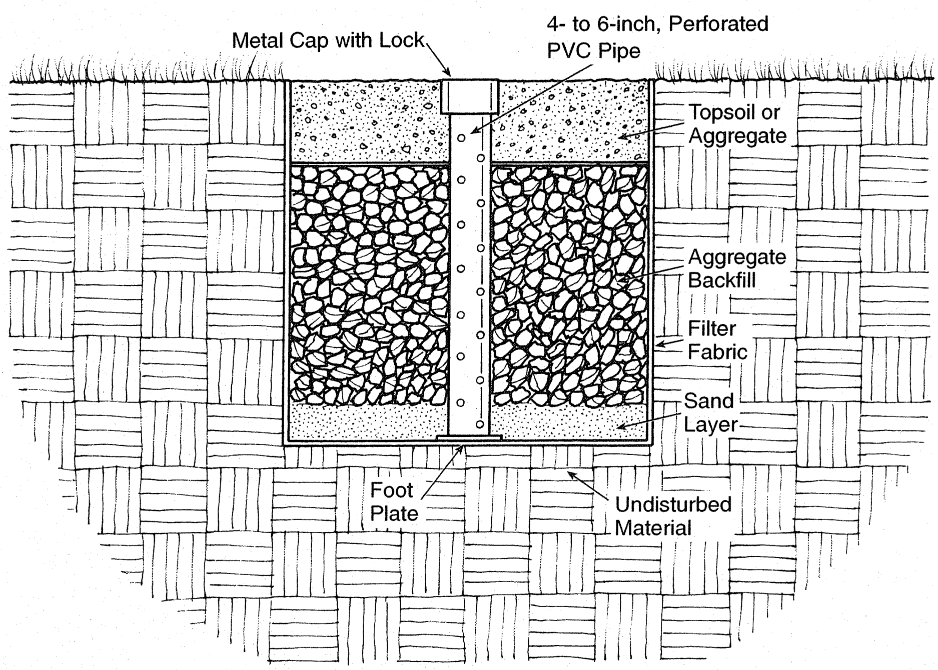
Winter operation

Infiltration trenches can be operated in the winter if the bottom of the trench lies below the frost line. Freezing is not as likely if a subsurface pipe brings the storm water into the stone aggregate. If the trench has a surface inlet grate, it must be kept free of ice and snow to operate effectively. Trenches covered with top soil may not operate efficiently during the winter because frozen soils tend to reduce infiltration.

Safety

In general, trenches are not likely to pose a physical threat to the public and do not need to be fenced. The primary public safety concern is ponding from an improperly draining trench, which could create a hazard, habitat for mosquitoes or some other nuisance condition. Inlet areas and observation wells are accessible and need to be locked to protect against vandalism.

Figure 5. Observation well construction



Adapted from MD-DERSSA, 1985

Design calculations

Depth

The infiltration rate and the porosity of the rock determine the maximum allowable depth (d_{max})(in):

$d_{max} = fT_s/n$

where f = design infiltration rate (in/hr)

T_s = maximum storage time (hours)

n = porosity

For a given soil and stone aggregate, the maximum depth of the trench can be determined. This depth may also be affected by groundwater or bedrock elevation at the site, which might require a shallower trench than the d_{max} calculated from this equation.

Volume

The volume of the trench is based on the water quality volume, plus the volume of rain that falls on the trench, minus the infiltration volume from the bottom of the trench during the runoff period.

$V_w = QA_u + PA_t - fTA_t$

Where A_u = the upland area

A_t = the trench area in the horizontal plane

Q = the water quality runoff depth

P = rainfall

T = the time the inflow is greater than the outflow and the trench fills (generally less than 2 hours)

f = the design infiltration rate

V_w = required trench capacity

The trench storage volume can also be written as the ratio of the volume of water that must be stored over the porosity (V_w/n). It is also the depth (ft) times the surface area (ft^2) or ($d_t\ A_t$). Combining these two equations leaves

$V_t = V_w/n = d_t\ A_t$ or

$V_w = d_t\ A_t\ n$

Equating the two previous equations:

$d_t\ A_t\ n = Q\ A_u + P\ A_t - fTA_t$

The surface area will then equal:

$A_t = Q\ A_u / (n_r\ d_t - P + fT)$

The factor d_t can be based on the maximum allowable depth, or a depth chosen to match site restrictions. Trenches are often used in small, restrictive sites so the length (L_t) or width (W_t) might already be decided. The trench configuration then depends on the remaining dimensions.

$L_t = (Q\ A_u)/\ ((n_r\ d_t - P + fT\ )\ W_t\ )$

Additional storage will be needed if the infiltration trench is to be used for peak shaving. Use TR-55 or another acceptable method to estimate this volume.

Enhancing pollutant removal

Following the basic guidelines will help ensure design of a successful infiltration trench. Close adherence to a few key points provides a greater margin of safety and enhances pollutant removal.

- **Surface area.** Broader, shallower trenches reduce the risk of clogging by spreading the flow over a larger area for infiltration, and increase the separation distance to groundwater.
- **Subsoils.** The objective of the trench is both to treat pollutants in the storm water and to move water through the underlying soils. A balance of pollutant-removing qualities, along with the infiltration potential of the soil, is necessary for successful operation. The clay and organic content of the soil determines the amount of sorption and bacterial degradation of pollutants. The texture of the soil largely determines movement. Fine textured soils, such as clays, optimize sorption while sandy soils optimize movement. Intermediate textured soils provide the best combination of treatment and movement.
- **Drain time.** A 48- to 72-hour drain time is appropriate for design. For marginal (finer textured) soils, a 48-hour drain time will build in a sufficient safety factor. Marginal soils tend to clog faster than sandier soils, so a more conservative design will prolong the facility's life. For adequate pollutant removal, the minimum recommended drain time is 6 hours.
- **Maintenance.** If the trench drains in less than 6 hours or more than 72 hours after a significant storm, remedial measures will be needed. These measures could include reworking the trench, rototilling or removing and replacing a clogged filter fabric. Close observation of the trench during start-up and regular inspection thereafter is necessary to determine how well it is operating.

Construction guidelines

Premature clogging is often a result of poor construction techniques or improper control of sediment during construction. The following guidelines will help minimize the problem.

- Before any construction begins, divert storm water runoff and construction traffic away from the site of the trench.
- Trench construction should not begin until the upland site is stabilized or runoff diverted. The trench site should not be used as part of the construction site erosion control plan.

- Excavate the trench using a backhoe or trencher with oversized tires to prevent compaction, following the U.S. Occupational Safety and Health Administration (OSHA) Trench Safety Code for acceptable construction practice. Do not use bulldozers or front-end loaders. Each trench section should be dug, filled with rock and covered before a new section is dug. Start only a portion of the trench that can be completed in one work day. Place excavated materials at least 10 feet away from the edge of the excavation to prevent backsliding or cave-ins.
- After the trench is dug, roughen or scarify the bottom and sides to restore infiltration capacity that may have been compromised by rainfall or smearing of the soil surface during digging.
- Cover the trench bottom with 6 inches of clean sand. Place a geotextile filter fabric on the sides and one foot below the top of the trench, overlapping it at the seams to prevent soil fines from entering the stone aggregate. The fabric should be flush with the walls. If voids have occurred during excavation, fill these spaces with soil. Trim tree roots flush with the sides to prevent tearing or puncture while the fabric is placed. Select a suitable filter fabric, since they vary significantly in permeability and strength. Filter fabrics must be able to retain the soil at the site while allowing water flow without clogging. Non-woven geotextile fabrics retain more soil fines, are less prone to clogging and have very good permeability characteristics as compared to woven geotextiles. The filter fabric should meet the requirement in Natural Resources Conservation Service Material Specification 592 Geotextile Table 1 or 2, Class 1, with an equivalent opening size of 30 for non-woven and 50 for woven fabric. Filter fabric is susceptible to ultraviolet degradation, so take care to minimize exposure during construction.
- Install an observation well to locate the site and provide access for collection of operational data.
- Clean, washed, 1.5- to 3.0-inch stone aggregate should be placed in the trench in lifts and lightly compacted with a plate compactor. Using unwashed stone will result in premature clogging from the stone's heavy sediment load. If the stone aggregate is contaminated by sediment during construction, remove and replace it with clean aggregate.
- Place filter fabric horizontally over the aggregate approximately 1 foot below the surface, and then cover it with permeable top soils or with larger aggregate. The top filter cloth will capture sediment from surface runoff and reduce the chance of clogging at the infiltrating surface layer.
- A 20- to 25- foot vegetative buffer around the trench will intercept surface runoff, protect the structure and prolong its life.
- Vehicle traffic must be kept off the trench before, during and after construction to prevent compaction.
- Sediment control after construction is critical. Sodding the upland areas and the vegetative buffer will speed up the stabilization of the area. If upland areas are seeded, the area must be inspected regularly until it is well established. (Refer to NRCS Technical Guide #342 *Practice Standards and Specifications for Critical Area Planting.)*

Maintenance

Trenches are prone to clogging by sediment, oil, grease and debris. Keeping the pretreatment facility in good condition will reduce maintenance and improve the trench's operation. Before construction, determine responsibility for maintenance of the system and set aside funds for both routine and nonroutine work. A maintenance manual should be developed and reviewed by both the party responsible for maintenance and the owner of the infiltration structure.

Monitor the trench frequently in the first year to determine how well the system is performing. If there are problems, continue monitoring on a more frequent basis. In the absence of problems, an annual observation with drain times recorded will be sufficient.

Sediment will naturally build up in the pretreatment portion of the trench. A sump pit used as pretreatment will need frequent cleaning. Other pretreatment facilies must be monitored for sediment build-up and cleaned as appropriate. Sediment can also build up on the top foot of the trench itself. Estimate the level of sediment clogging by digging a small hole down to the filter fabric.

Maintain the buffer and surface vegetation by reseeding bare spots and mowing as often as dictated by the aesthetic needs of the area. The grass should not be cut shorter than 3 inches to maintain filter performance. Mowing will also prevent undesirable woody growth on the surface of the trench.

Even well-designed, constructed and maintained trenches will lose effectiveness over time. The maintenance plan should include non-routine maintenance such as rehabilitation of a trench after it clogs.

Surface trenches often clog at the top. This can be corrected by stripping off the top layer, replacing the clogged filter fabric, and replacing the top foot of aggregate or soil. Underground-loading trenches typically clog at the bottom filter fabric or sand layer because storm water flows are piped directly to the aggregate layer. Correcting a clogged, underground-loading filter can be very costly, because it requires removal of all aggregate, tilling the bottom and replacing the top layers.

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The Wisconsin Storm Water Manual: Infiltration Basins and Trenches (G3691-3)

G3691-4

The Wisconsin Storm Water MANUAL

Wet Detention Basins

Numerous techniques are available to determine design storms and to predict peak flow rates and volumes. (For an example of an alternative method see the *Wet Detention Basin Standard* No. 1001 (WLWCA, 1999).) Design storms and techniques used to predict peak rates and flow are for illustrative purposes and are described in detail in the hydrology section of this manual.

Multiple detention basins within a watershed may greatly change the natural flow conditions in the downstream reaches of receiving waters. Construction of storm water facilities should be part of an overall watershed management plan. The designer should coordinate construction of detention basins or any other storm water facility with local, municipal, county and regional planning representatives to minimize the risk of flooding both upstream and downstream from the facility.

Detention basins are excavated areas or enhanced natural depressions designed to detain storm water runoff. These structures detain or impede flows by storing runoff and releasing the stored volume at a reduced rate. Such structures have historically been employed to reduce peak discharges and provide greater protection to areas that are susceptible to flooding.

With increased public interest in improving water quality, detention basins have gained importance for their ability to remove pollutants from storm water runoff. The objective of this publication is to assist engineers and designers in planning and designing water quality detention basins by presenting sizing and construction design criteria to meet water quality goals.

Flood control and/or peak shaving components are often incorporated into the detention basin. The U.S.D.A. Natural Resources Conservation Service (NRCS), the Army Corps of Engineers, the Department of Natural Resources, and others have design standards for peak flow control; therefore, only the water quality aspects of design will be discussed here.

Recommended design objectives

To obtain water quality improvements for urban water quality basins, the Wisconsin Department of Natural Resources (DNR) recommends that basin design meet the following criteria (WI-DNR, 1997):

- Storm water management practices should remove 80% total suspended solids (TSS) from runoff generated from the developed tributary drainage area on an annual basis.
- Storm water management practices must limit the peak discharge from the post-developed site to the peak discharge of the pre-developed site for the 2-year, 24-hour rainfall event. This requirement is intended to limit streambank erosion downstream from the facility under bankfull flow conditions. In cases where the facility's discharge will have no adverse impact on the downstream conveyance system, this requirement can be waived.

Other criteria may be included where specific pollutants such as metals or pesticides are of concern. Compliance with the above criteria will ensure that a significant amount of the pollutants contained in storm water runoff will be removed.

Types of detention basins

Detention basins may be categorized as either dry or wet detention. Dry detention basins offer maximum storage potential and reduce the risk of flooding and streambank erosion by attenuating peak flows. However, they have limited ability to permanently remove pollutants because the deposited materials are often re-suspended by succeeding storms. For this reason, dry detention is not recommended as a water quality improvement practice.

Material discussed here concentrates on wet detention. Because no standard definitions of the various storm water storage facilities have been established, the types of wet detention basins used in this manual are defined as follows:

A *wet detention basin* is an impoundment containing a permanent pool of water. It also has additional storage capacity above the pool's surface to provide temporary storage for runoff peak reduction. Water quality treatment is usually accomplished through physical and biological processes in the permanent pool. Wet detention basins may be used as a single pollutant removal facility or as a pretreatment device in combination with other storm water management practices.

An *extended wet detention* basin is a detention facility designed to store runoff for an extended period of time. The extended detention time of these basins allows more time for physical settling of pollutants. Extended detention systems typically have a shallow marsh in combination with a dry area or have a permanent pool in combination with a dry area. Extended wet detention or a wet detention basin in combination with another practice will be somewhat more effective in removing silts, clays, phosphorous and some of the other pollutants from storm water due to the increased detention time.

The design of an extended wet detention basin will incorporate many of the same aspects as the wet detention. The basic design differences between extended wet detention and wet detention is that extended wet detention requires a smaller discharge, longer detention time, a larger storage area and vegetation more tolerant of varying water levels.

Detention basin benefits

Wet detention basins are generally effective storm water quality management structures if designed and maintained correctly. Basins can be used on individual sites or as regional storm water facilities. Use as a regional facility introduces economy of scale, providing advantages over site-by-site installations. Compared to site-by-site facilities over a total drainage area, regional facilities have smaller land area requirements, are less costly to construct than multiple basins, and require less maintenance.

Basins must be designed in a manner that does not increase the chance of flooding downstream; flow routing through the multiple basins may add to the design complexity.

Detention basins can also be used in conjunction with other water quality facilities to enhance pollutant removal capabilities. By reducing discharge and removing sediment in upstream basins, detention basins allow water quality practices downstream to operate more efficiently. For example, artificial wetland storm water management systems and infiltration structures will not operate efficiently if flash flows and sediment from urban areas enter them directly. Installing a detention basin to provide pre-treatment for these practices can reduce flow rates and sediment loads to levels that prevent premature failure, and often provide pollutant removal efficiencies at levels higher than both practices could achieve operating independently.

The design rate of discharge from a detention basin used in conjunction with a downstream practice, such as an artificial wetland storm water management system or infiltration structure, will depend on the inflow requirements and the volumetric capacity of the downstream practice.

Compared to other water quality practices, wet detention generally requires less land area and achieves comparable levels of pollutant removal. Because of their storage capability, detention basins are able to handle much larger volumes of flow than other practices such as grassed swales or infiltration structures. In addition, detention basins are less susceptible to failure and require less maintenance than infiltration practices.

The major pollutants contained in storm water include sediment, lead, arsenic, copper, mercury, atrazine, polycyclic aromatic hydrocarbons (PAH), phosphorous, zinc, bacteria and dissolved nutrients (US-EPA, 1983). Estimated removal rates for wet detention basins are shown in table 1.

Table 1. Percent reduction of pollutants for wet detention basins

Pollutant	**Removal rate** (%)
Suspended solids	70–95
Total phosphorous	40–70
Nitrogen	60–90
COD	20–55
Lead	70–90
Iron	43–92
Zinc	40–80
Oxygen demand	50–90
Copper	60–80

Adapted from Pitt, 1991; Schueler, 1987; Stahre and Urbanos, 1990 and MD-DERSSA, 1991

In addition to improving water quality, properly designed wet detention basins may provide other benefits. If additional storage is provided, the peak storm water discharge from larger design storms may be reduced.

A basin may improve the aesthetics of an area through proper siting and use of an irregular shape for the basin edge. In some cases increased recreational opportunities may be created by integrating the detention basin into the surrounding land use.

If located and maintained appropriately, wet detention facilities are an attractive amenity, and in some cases will actually increase the surrounding land values (Schueler, 1987). Accomplishing these benefits usually requires that basin design be a primary element in the development plans.

Detention basin drawbacks

While detention basins effectively remove a number of pollutants, they do not consistently or significantly remove soluble substances such as certain pesticides, zinc and petroleum products. Detention basins also allow sunlight to increase water temperatures, which may have a detrimental effect on aquatic life in the receiving water body.

If thermal impacts to the receiving water body are a concern, some other method of pollutant removal should be used in conjunction with detention. For example, an infiltration basin placed downstream from a detention basin would reduce water temperature and help minimize thermal impacts on the receiving body of water.

Provisions must be made to dredge, test, and properly dispose of sediment on a regular basis. The responsibility for maintenance and long-term accountability for maintenance are often difficult to establish.

A maintenance schedule, statement of procedures and a cost estimate should be a part of the detention basin design. A maintenance agreement should also be developed before constructing the basin to establish the parties responsible for maintenance and repair.

Safety is also a concern with detention basins. Precautions should be taken to discourage swimming and entry to the pool area. Features such as safety shelves will decrease the risk of injury and drowning, but will not eliminate these risks.

Detention basins and water quality

Detention basins are designed to interrupt and detain the normal flow of storm water runoff. Unlike flood control facilities, detention ponds for water quality control are designed for the more frequent or smaller storm. Ideally, in cases where downstream flood control is required or where bank erosion would be intensified through development, the detention facility would be sized for both water quality and peak flow control.

Sediment removal

The primary pollutant removal mechanism used in detention basins is particle settling, supplemented by biological and chemical activity. Settling in detention basins generally takes place at two distinct times and under different hydraulic conditions.

The first type of settling is called dynamic settling and occurs during flow through the pond. The second type, quiescent settling, occurs during the period between rainfall events.

The analysis of settling is often conducted using the assumptions of a "plug flow" system. In a plug flow system, the water that has been held in the pond from the previous rainfall event is displaced by inflow from the current event. Given enough time in a semi-quiescent water body, suspended solids settle to the bottom of the basin through the action of gravity.

In cases of large inflow volumes, however, flow in the pond occurs predominately near the surface, with much slower velocities existing near the bottom of the facility. In this situation, distribution of flow over a large surface area to slow the inflow velocity is a critical factor in removal of suspended solids. Particles settling below the outlet will be captured in the pond; those that do not settle below the outlet will be transported downstream. The relationship between the surface area and the particle removal for an ideal settling basin has been described by numerous authors including Pitt (1994).

The critical particle settling velocity is defined as:

$$v_c = Q_{out}/A_{surface}$$

where:

Q_{out} = pond outflow rate (cubic feet per second),

$A_{surface}$ = pond surface area (square feet: pond length times pond width), and

V_c = upflow velocity, or critical particle settling velocity (feet per second).

For an ideal detention pond, particles with settling velocities greater than this critical settling velocity will be completely removed. Increasing the surface area or decreasing the pond outflow rate will increase pond settling efficiency. Increasing pond depth reduces the possibility of bottom scour and resuspension of sediments, decreases the amount of attached aquatic plants and decreases the chance for winterkill of fish. Deeper ponds may also be needed to provide sacrificial storage for sediment between dredging operations (Pitt, 1994.)

Therefore, surface area is the critical factor when designing detention basins for settling efficiency, and depth is primarily important only from the aspect of protecting bottom sediments from surface turbulence and providing sediment storage.

For the purposes of this manual, the fall velocity of a 5 micron particle, or a settling velocity of 0.00013 feet/second, is used to accomplish an annual 80% removal rate. This 5 micron particle size is based on findings of sampling data taken from runoff in Madison and Milwaukee streets. The research determined that particles equal to and larger than the 5 micron particle comprised approximately 80% of the particle size distribution coming from the streets sampled (WI-DNR, 1997).

While the specific weight of soil particles varies, affecting the settling velocity, the particles typically found on urban surfaces have a specific gravity of approximately 2.75 (Pitt, 1994). This value is generally larger than the specific gravity for native soil particles. Due to variations in particle size distribution and density among sites, however, designers are encouraged to determine the particle size distribution and specific gravity for the site before design.

Limiting peak flows

As an area is urbanized, the amount of impervious surface area in the drainage area is usually significantly increased. Storm sewers are installed to quickly convey runoff from developed sites. Landscaping and surface grading remove natural surface depressions that provided storage areas for runoff.

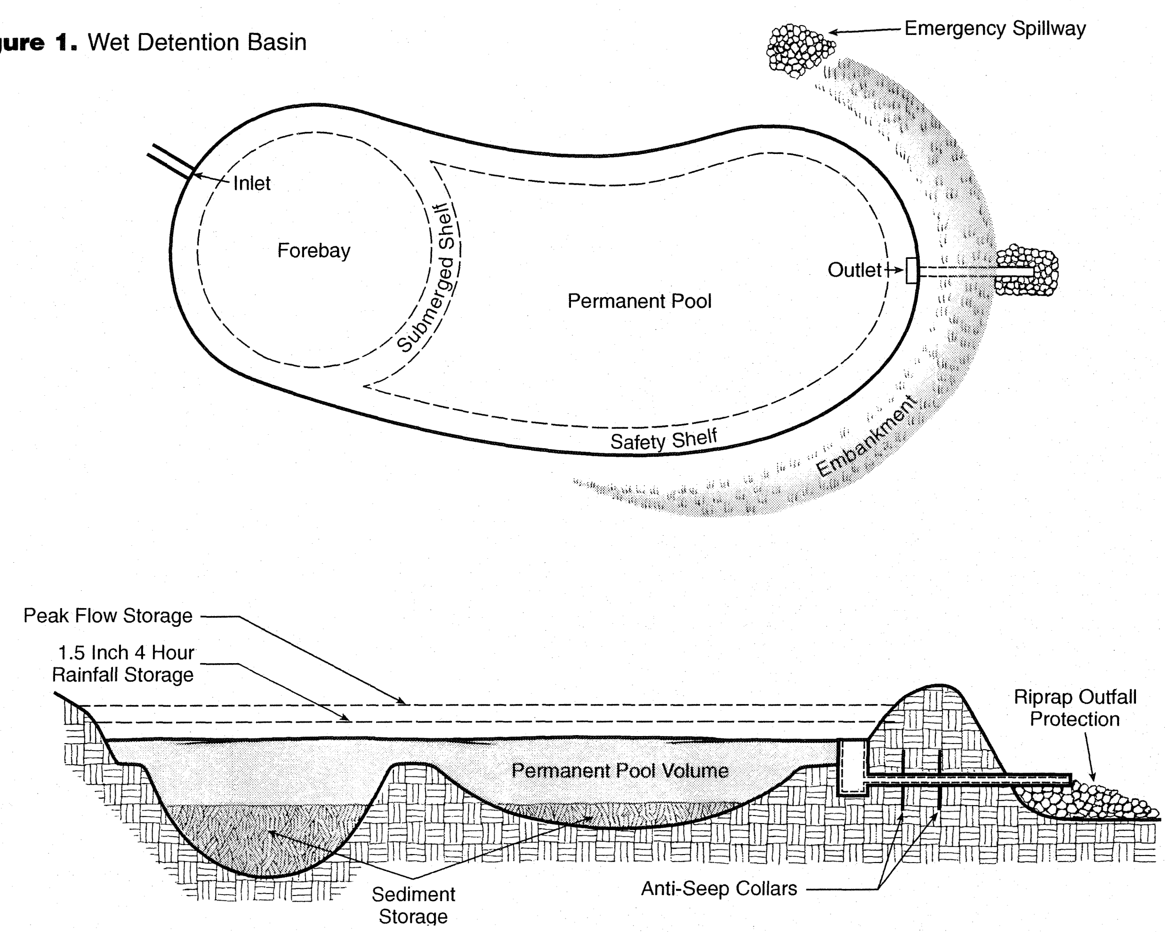
These combined changes have a number of detrimental effects on receiving streams. These effects include:

- Increased runoff peak discharges
- Increased runoff volumes
- Increased flow velocity during storms
- Decreased time of concentration
- Increased frequency and severity of flooding
- Reduction in base flow between storms
- Increased streambank erosion
- Increased water turbidity due to bank erosion and upland erosion
- Increased introduction of toxic chemicals to water bodies

As a result, streams widen and become shallower, sediment fills natural pools and coats the streambed, and the diversity of aquatic species is reduced.

By restricting peak flows from a developed site to the peak flow that existed before development, damage to downstream areas can be greatly reduced. Detention basins are an excellent way to diminish the destructive effects listed above. By designing detention basins to restrict flows and temporarily store the increased flows produced by urbanization, downstream flow conditions can more closely approximate the flow regime that existed before urbanization in the watershed.

Figure 1. Wet Detention Basin



Research conducted by Leopold (1968), Wolman and Schick (1967) and others has shown that stream channels and courses are greatly affected by smaller, more frequent rainfalls. In Wisconsin, storm events between the 1-year and 2-year return period generally cause bankfull flow condition.

This flow quantity controls and forms the natural stream channel. Therefore, restricting peak flows from the more common rainfall events can reduce the damaging effects resulting from the increased runoff produced by urbanization.

While determination of the appropriate return period is subject to debate, the 2-year storm event will cover the wide range of stream flow characteristics and, when used with the water quality design guidelines, will adequately protect streams from the negative effects of high frequency storms.

As can be seen from the previous description, peak flow limitations are applied to streams or other water bodies that will be negatively affected by increased flows. Water bodies in which the increase in flow produced by urbanization will have little effect on the receiving water body do not have to conform to these guidelines. For example, a relatively small basin discharging into a large lake such as Lake Winnebago or Lake Michigan may have no significant effect on water quality.

Basin design

To achieve its water quality goals, basin design must incorporate a number of design criteria. These include permanent pool volume, active storage volume, surface size and shape, pond depth, site topography, inlet and outlet structure design, slope stability, and safety.

Guidelines provided in this section will assist engineers and designers in constructing water quality detention basins to meet water quality goals of removing 80% of suspended solids and maintaining the pre-development, 2-year, 24-hour peak flow rate. An example of a wet detention basin with its basic features is illustrated in figure 1.

Planning guidelines

A number of items must be addressed before the design phase. These include identifying unique or sensitive natural areas, expected future land uses, availability of land, permits required and the effects of the proposed structure on downstream flow. A comparison of detention basins to other pollution removal practices should also be conducted.

To thoroughly assess what may be required in managing storm water pollution, a watershed or sub-watershed study should be conducted first. In some watersheds these studies may have been completed. Consult local officials, the regional planning agency, and the DNR to determine the status of watershed plans and other watershed information.

Regional reports may not be detailed enough for a specific site and the surrounding area. In every case the developer has the obligation to contact government organizations to determine how the proposed structure will affect the watershed and whether the structure is consistent with watershed plans. In addition, the contractor or designer is always responsible for ensuring that the risk of flooding is minimized by assessing flooding potential both upstream and downstream from the proposed site.

The design of a detention basin is often an iterative and complex process. The following overview provides a guide to the order and overall process.

1. Check with local officials, regional planning agencies and state and federal agencies to determine zoning restrictions, watershed and/or surface water requirements and permits that may apply to the development site or watershed.
2. Conduct a site evaluation. The initial field inspection should include identification and location of any springs in the immediate vicinity of the proposed basin site. The flow from these springs should be considered and rerouted if necessary to prevent instability of the detention structure.

   All utility lines should be located outside the basin site. If necessary plans should be made to move lines outside the basin site. All sanitary pipes should be located outside the basin site and located to minimize the chance of pond contamination should a pipe fail. Check local ordinances for criteria. Manholes in the area where the wet surface will overtop the manhole for the 2-year, 24-hour event should be relocated outside the wet surface area. Determine public and private well locations that may be affected by the detention basin.

   Take soil samples from the potential detention sites to help determine:

   —In situ soil permeability, to ascertain if the soil is capable of inhibiting seepage. This information will help establish the necessary degree of compaction or whether a liner will be required to prevent large surface water fluctuations.

   —The soil's ability to support loads and maintain its shape.

   —Depth to groundwater or fractured bedrock. If separation distance is less than 4 feet, special precautions will be necessary to prevent movement of pollutants to groundwater. A basin liner will help minimize pollution potential due to seepage from the basin.

3. Make sure the watershed and conveyance channels are stabilized to minimize sedimentation in the detention basin.
4. Analyze watershed data to determine the viability of a detention basin.
5. Estimate the wet pond surface area based on future land uses in the drainage area.
6. Using the site survey, calculate the storage volume associated with several elevations or stage levels.
7. Determine if the area can accommodate:

   —A permanent pond volume equal to the runoff volume from the drainage area in the fully developed condition for the water quality design storm (1.5-inch rain).

   —An active storage volume, large enough to remove the 5 micron particle from the runoff volume for the drainage area in the fully developed condition for the 1.5-inch, 4-hour rain. This is approximately equal to one-half the permanent pond volume.

   —An active storage volume large enough to meet outflow requirements for the fully developed area runoff from a 2-year, 24-hour storm.
8. Calculate the expected future runoff volume for the 1.5-inch storm and the 2-year, 24-hour storm.
9. Create the hydrograph for the 1.5-inch, 4-hour rainfall and the 2-year, 24-hour storm. These are the inflow design hydrographs for the detention basin. If it is desirable to control flooding from larger magnitude storms, also create the appropriate inflow hydrographs.
10. Determine if the selected sites will accommodate the estimated volumes. If yes, go to step 10. If not, site conditions must be modified and steps 2 through 5 repeated.
11. Design an outlet that will restrict pond discharges from the 1.5-inch, 4-hour storm to 0.00013 cubic feet per second per square foot of pond surface area. If sediment characteristics vary from those used in this manual the flow rate must be revised.
12. Route the 1.5 inch rainfall hydrograph through the basin as a check of the water quality active storage and outlet basin design. If discharge and storage requirements are satisfied, continue to step 13; if not, redesign outlet and/or modify basin to satisfy storage requirements and repeat steps 9 through 11.
13. Determine the peak runoff and the runoff volume from the 2-year, 24-hour storm with the drainage area in its pre-development condition.
14. Develop the runoff hydrograph and the runoff volume from the 2-year, 24-hour storm with the drainage area in its fully-developed condition.
15. Compare the peak flow from the hydrograph developed in step 14 with the peak flow for the outlet designed in steps 9 through 11. Make sure the design of the outlet limits the fully developed peak flow to the pre-developed peak flow. For a first estimate of the storage volume needed to limit the fully developed to the pre-developed peak flow use the difference between the fully developed volume and the pre-developed volume.
16. Route the 2-year, 24-hour rain event hydrograph through the basin as a check of the 2 year storage and outlet design. Determine if the peak discharge and storage requirements are satisfied. If so, continue to step 16; if not, redesign outlet and/or modify basin to satisfy storage requirements and repeat any steps that may apply from steps 9 through 15.
17. Route other peak or flow control storms and check design features.
18. Assess basin flow characteristics as they affect the watershed. This may involve checking with regional and local government staff or with the DNR.
19. Design details of the basin including safety, maintenance and operational features.

Basin sizing calculations

In developing the design requirements for a properly sized basin, DNR staff conducted a study to determine the design storm volume to achieve an 80% removal of total suspended solids (TSS) on an annual basis.

The study determined that a basin with a permanent pond sized to contain the runoff from a 1.5-inch rainfall would perform at the required level of pollutant removal. Other design criteria may be found in the literature; however the design requirements described in this manual use the runoff from the 1.5-inch rainfall for the permanent pond specification. Because the design guidelines given here apply for a wide variety of conditions, the design is necessarily conservative and basins designed for specific conditions may be smaller.

Where there are land area limitations, or where land purchases would be costly, the designer may want to employ a more specific design. A number of computer models may be used to design detention basins. Check with local officials to determine which models are acceptable.

The following sections describe the basic elements of the basin structure and the procedure used to size various basin storage volumes and the outlet structure to meet water quality needs.

While the function of the inlet structure is also described here, specific information on sizing the inlet structure must be obtained from sources such as the NRCS and the Army Corps of

Engineers. The reader is also advised to look to other sources for flood control design elements and for the structural design of the basin.

The design sequence begins with the inlet structure and expected inflow from the drainage area, followed by sizing of the permanent wet pond and the sizing procedure for the water quality active storage volume and the outlet structure. The water quality active storage volume is determined to a large degree by the outlet structure. For this reason the outlet structure and the water quality active storage volumes are determined by flow routing using the runoff hydrograph for the drainage area as the inflow to the basin.

The design method described here is derived from five main sources (Pitt, 1994; WA-DOE, 1992; Walker, 1987; Schueler, 1987; and Barfield et al., 1983).

Inlet structure design

The detention basin inlet is a structure that takes concentrated flow and distributes it so that the energy can be more easily dispersed in the permanent wet pond. When properly designed, the inlet transforms the concentrated incoming flow to a dispersed, surface flow that does not disturb the settled bottom sediments.

To prevent erosion near the inlet, adjacent areas may need to be protected by vegetation, riprap or some other means. Inlet structure sizing is determined by the peak rate of flow generated from the drainage area served by the detention basin.

The inlet should be constructed in a manner consistent with methods described for conveyance structures in NRCS Technical Guides (USDA-SCS 1985,1987,1993). The minimum design storm for the inlet structure should be a 10-year frequency storm unless the maximum expected conditions warrant use of a larger storm model.

Permanent pond surface area calculation

In order to settle the 5 micron particle for a 1.5-inch rainfall, the permanent pond must provide an adequate surface area for the expected incoming flows. Observations noted by Pitt (1994) as modified by the Wet Detention Basis Standard No. 1001 (WLWCA, 1999) may be used to approximate the required surface area. These observations indicate that the surface area recommendations provided in table 2 should be followed when sizing the permanent pool.

In most cases the drainage area consists of mixed land uses, and the pond surface is determined by multiplying the acreage of each land use area by the recommended percent from table 2 and summing the components.

As an example, assume that a 100-acre drainage area has the following land-use characteristics and estimated pond surface area as shown in table 2:

100-acre drainage area

Residential	52 acres @ 0.8%
Manufacturing	13 acres @ 2.1%
Institutional	25 acres @ 1.8%
Open space	10 acres @ 0.6%

The estimated permanent pond surface area for 80% control would be:

(52 acres x 0.008) + (13 acres x 0.021) + (25 acres x 0.018) + (10 acres x 0.006) = 1.2 acres

Permanent volume calculation

The permanent pond is designed to dissipate the energy of the incoming flow, allowing suspended solids to settle as flow velocity decreases. The permanent pond also provides a protective water column over sediments that previously settled. In many cases the permanent pond is excavated below the existing surface to ensure that the pond water volume will not present a flood risk downstream in cases of impoundment failure.

Table 2. Estimating pond surface area as a percent of tributary drainage area

Land use/ Description/ Management	Total impervious (%)	Miniumum surface area of the permanent pool (% of watershed area)
Residential		
• <2.0 units/acre (>1/2 acre lots)	8–28	0.7
• 2.0–6.0 units/acre	28–41	0.8
• >6.0 units/acre (high density)	41–68	1.0
Office park/Institutional/Warehouse		
(Non-retail related business, multi-storied buildings, usually more lawn/landscaping	<60	1.6
not heavily traveled, no outdoor	60–80	1.8
storage/manufacturing	>80	2.0
Shopping/Manufacuring/Storage	<60	1.8
(Large heavily used outdoor parking areas,	60–80	2.1
material storage or manufacturing operations)	>80	2.4
Parks/Open space/Woodland/Cemeteries	0–12	0.6
Highways/Freeways (Includes right-of-way area)		
• Typically grass banks/conveyance	<60	1.4
• Mixture of grass and curb/gutter	60–90	2.1
• Typically curb/gutter conveyance	>90	2.8

The permanent pond volumetric calculation described is a simplified method to size the permanent pool for water quality improvement. This method results in a permanent pool volume that may be slightly conservative. A reduced permanent pool volume may be obtained by using a water quality detention basin model. The basin should be sized for an 80% TSS removal.

The permanent pool consists of three volumes—a sediment storage volume, a forebay volume, and a main pool volume. The total permanent pool volume should be equal to the total runoff volume generated from a 1.5-inch rainfall on the drainage area in its fully developed condition plus the sediment storage volume.

To capture the majority of sediment entering the pond, a sediment forebay area is located adjacent to the inlet. By trapping and holding the majority of sediment in the forebay area, sediment may be more easily removed, thereby lengthening the useful life of the main pond. Containing sediments in the forebay requires that a submerged shelf be constructed to separate the forebay from the main pool. The surface area of the forebay should be approximately 12% of the permanent pond surface area.

Confining the sediments in the forebay requires that it include a sacrificial storage volume for sediment in addition to the storm water volume. The sediment storage volume should have a minimum wet pond depth of 3 feet above the projected maximum sediment storage volume to prevent resuspension of settled sediments.

Experience indicates that urban detention basins have an annual sediment deposition approximately equal to 1% of the permanent pool volume (Schueler, 1987). This volume multiplied by the desired maintenance design life provides an estimate of the design sacrificial sediment storage volume in the forebay area.

The sediment removal frequency should normally be 5 to 10 years. So, a pond design life of 10 years requires a sediment storage volume of 10% of the permanent pool volume. It should be noted that in order to lengthen the life of the main permanent pond area, the pond should initially be constructed to allow at least 1 foot of sacrificial storage.

To summarize, the permanent pond volume:

- Contains the runoff volume from 1.5 inches of rain falling on the drainage area in the developed condition
- Possesses a sediment forebay with a surface area equal to about 12% of the permanent pond surface area
- Has a sediment forebay capable of storing sediment between cleanout periods equaling 1% of the permanent pool volume multiplied by the time between cleanouts.

At this point in the design sequence the designer should make the first of many checks to be sure that the site selected will accommodate the pond volumes and flows.

Active storage volume for water quality control

While there may be several layers of active storage above the permanent pond in detention basins, this manual is concerned only with the water quality storage volume, which involves two storage volumes. One limits the flow rate to a discharge of 0.00013 cubic feet per second per square foot of pond surface area for a 1.5-inch, 4-hour rainfall to ensure adequate settling. The second storage volume limits the 2-year, 24-hour storm developed area peak flow to the pre-development 2-year, 24-hour peak flow.

Additional capacity may be required if control of larger storms is desired. The designer should assess the basin over the entire range of storms for which the structure is designed.

The 1.5-inch, 4-hour active storage volume is the first storage volume above the permanent pond of the detention basin. In conjunction with the permanent storage pond, this storage volume is responsible for removing suspended solids. The peak shaving volume for the 2-year, 24-hour storm is above the 1.5-inch, 4-hour pond stage and is used to limit streambank erosion downstream from the basin. Both of these volumes are determined by calculating the relationship between the flow hydrograph and the outlet structure's release characteristics.

Conveyance structures

- Inlets and outlets should allow for authorized access for maintenance and general repair, restrict entry by unauthorized persons, and use materials and designs that inhibit vandalism. The design should incorporate erosion protection and provide a sufficient foundation to reduce settling and frost heave.
- Code requirements for minimum pipe size, slope and cover should be determined if the basin is a component of a public storm drainage system. To enhance self-cleaning characteristics, pipe should not be laid on less than a 1% slope. However, if it is documented that 1% is not obtainable, then actual slope may be as low as 0.5%. When pipe is laid on an area with a slope greater than 20%, the pipe should be anchored and particular attention given to pipe joint areas. Check local ordinances to determine if the design is in compliance.

Calculating basin storage volume

The flow-routing method is used to calculate both the basin storage volume for the 1.5-inch rainfall and the basin storage volume for the 2-year, 24-hour peak shaving. The two methods are slightly different and the steps for both are listed below. Examples of routing procedures are presented in the Hydrology section of this manual.

Calculation for the 1.5-inch, 4-hour rainfall active storage volume

1. Develop the hydrograph for the 1.5-inch, 4-hour storm with the drainage area in its fully developed condition.
2. Using basin site surveys, calculate the stage-storage volumes and stage-surface area relationships at several stage elevations to a height above the expected maximum storage volume. To establish accurate storage volumes, break points where the land surface changes slope should be identified and used as a storage stage height.
3. Using the permanent pond area and the stage-storage relationship from step 2, choose an outlet structure that will approximate the allowable discharge rate of 0.00013 cubic feet per second per square foot of pond surface area. Recognize that the surface area of the pond increases as the pond elevation rises. Usually a good first estimate of the needed volume of storage for the 1.5 inch rainfall is one-half the volume of the permanent pond volume. Using the designed outlet, develop a stage-discharge relationship.
4. Route the runoff hydrograph developed in step 1 through the pond to check discharge limits and storage.

A pond outflow hydrograph and a pond stage-storage curve are used to determine if water quality goals for the pond will be met, if a larger storage volume will be required, or if the outlet structure will require alteration. This is an iterative process for determining if the outlet and active storage volume will meet the outflow-surface area ratio requirement. For example, if a chosen outlet and the consequent pond have an outlet release rate/surface area ratio that is greater than the required 0.00013 cubic feet per second outflow per square foot of pond surface area, the outlet opening must be reduced or the pond active surface area must increased. The resulting design must then be assessed again to determine if the outlet release-surface area ratio requirement is met.

Calculation for active storage for streambank protection

In reducing the peak flow for streambank protection, pre-development flow from the 2-year, 24-hour storm should be maintained. To design a structure that will limit peak flows to the predeveloped condition, use the following procedure:

1. Calculate the pre-development inflow peak for the 2-year, 24-hour storm using the tabular method in TR-55 (USDA-SCS 1986). This will determine the design peak outflow for the post development condition.
2. Calculate the post-development inflow hydrograph for the 2-year, 24-hour storm using the tabular method in TR-55 (USDA-SCS 1986).
3. Choose an outlet that will limit the peak flow from the detention pond with the drainage area in its developed condition to the peak flow calculated in the pre-development hydrograph. Ideally the flow from the detention basin should replicate the pre-developed inflow hydrograph as closely as possible, including the base stream flow.
4. Using the permanent pond area and the stage-storage relationship from step 2, choose an outlet structure that will approximate the allowable discharge rate of 0.00013 cubic feet per second per square foot of pond surface area. Recognize that the surface area of the pond increases as the pond elevation rises. Usually a good first estimate of the needed volume of storage for the 1.5-inch rainfall is one-half the volume of the permanent pond. Using the designed outlet, develop a stage-discharge relationship.
5. Route the runoff hydrograph developed in step 1 through the pond to check discharge limits and storage.

- Anti-seep collars should be provided for pipe inlets and outlets where pipes pass through berms. The collars should have watertight connections to the pipes. Maximum spacing should be approximately 14 times the minimum projection of the collar measured perpendicular to the pipe. Collar material should be compatible with pipe materials. The anti-seep collars should increase by at least 15% the seepage path along the pipe.
- Pipe outlets, both the principle outlet and the de-watering outlet, should ensure stability. For outlets 10 feet or less in height, a square concrete base 18 inches thick and a width twice the diameter of the pipe width may be used to anchor the outlet. The pipe should be placed at the center and embedded 6 inches into the concrete. Other approved methods may be used.
- Outlet structures should provide a skimmer type shield around perforated risers. A device or configuration that reduces the risk of outlet blockage should be provided on all outlet structures.
- To prevent structural damage to the basin facility, an emergency outlet capable of passing the flow equal to the flow capacity of the downstream conveyance system should be installed. To reduce the risk of erosion and structural failure, spillways should be placed on undisturbed ground. For most communities, the storm sewer system is designed for the 10-year, 24-hour event, and most communities have provisions to pass the 25 year storm event. Check with local officials to determine the proper design storm for the emergency outlet.
- To prevent clogging, trash racks should be installed to filter material that may be caught in the conveyance system downstream from the outlet. The spacing in the opening of the trash rack should be smaller than the outlet diameter. A general rule is to provide a trash rack that has a net open area no less than four times the open area of the outlet.
- Easements should be provided for all structures in need of regular maintenance. This is especially important for manhole facilities.

Additional design criteria

In addition to the basic basin design calculations, the designer should keep several other considerations in mind to ensure that the basin removes pollutants at the design levels, and that safety and aesthetic issues are adequately addressed.

- A detention basin should blend with the surrounding landscape and community as much as possible. While the placement of the facility may be limited due to hydraulic constraints, properly designed basins can be aesthetically pleasing and help serve community recreational needs.
- The surface shape of the pond should have a length to width ratio of 3 to 1 or greater. If this arrangement is not possible, baffles or gabions should be installed to lengthen the flow path. Shapes should conform to the natural contour of the site to the greatest extent practical. Shapes that avoid dead or stagnant zones are encouraged because stagnant zones reduce sediment trap efficiency, often become overgrown with vegetation, and can increase mosquitos.
- The inlet and outlet structure should be at opposite ends of the pond to discourage short- circuiting of incoming water. This arrangement tends to maximize detention time, which allows for greater treatment of polluted water. If the inlet and outlet cannot be placed at opposite ends, baffles or gabions should be installed to lengthen the flow path.
- For elongated ponds in the direction of strong prevailing winds, consideration should be given to reinforcing banks, extending safety shelves, or other measures to prevent embankment deterioration due to wave action.
- Special design precautions may be needed to protect outlet structures from ice damage.
- Earthen embankments should meet all local and state regulations. Precautions must be taken to prevent flood damage in cases of embankment or dam failure. Dam regulations should be consulted to determine if the detention basin is subject to such regulations. For information contact the NRCS or DNR.
- Vegetation or some other type of slope protection may be needed to prevent soil erosion along banks in the zone where pool surface water fluctuates.
- The slope contours of detention basins outside of the live storage pool to the wet pond edge may vary from 4 feet horizontally to 1 foot vertically (4:1) to 10 feet horizontally to 1 foot vertically (10:1). Slopes steeper than 4:1 can cause safety problems due to slippery footing and hazardous operating conditions during maintenance. Slopes flatter than 10:1 may present drainage problems and provide mosquito habitat.

In the wet pond, the slope near the water's edge should be somewhat steep (4:1) to reduce mosquito problems and to provide relatively fast pond drawdown after most common storms. This slope should not exceed 18 vertical inches to reduce the risk of drowning. An area around the perimeter of the facility should have a safety shelf 10 feet wide. This shelved section should be relatively flat, 10:1 or flatter, to allow persons who may fall in a chance to regain their footing and pull themselves out. The sloped zone toward the center of the pond beyond the shelf region should be sloped as steeply as soil stability will allow to provide for the maximum volume of wet detention storage.

- Wet ponds should have a minimum three foot depth in the center section of the pool. This three foot requirement is in addition to any sediment storage that may be required.
- Planned vegetation is often overlooked in detention basin design. Vegetation can be a critical factor in determining the basin's water quality efficiency. For example, waterfowl manure can create significant biological oxygen demand (BOD) demands on the facility. Vegetation that completely surrounds the pond can be incorporated into the design to discourage waterfowl habitation. Vegetation can also provide an additional safety factor by restricting access to unsafe deep water, and may be used to enhance the appearance of the detention basin.

10 Varying the water depth can be an effective means of achieving a variety of vegetation. Safety shelves that surround the edges of detention ponds often have water depths less than 6 inches and may support emergent plants. Submerged plants require water depths between 1 and 2 feet. Open water is obtained with a design depth of four feet or greater.

- To ensure that a detention basin is properly sized, existing and planned future land uses must be known. The detention basin designed to take care of today's storm water needs may be undersized after 10 or 20 years of development in the drainage area. Likewise, a basin designed only to handle an area's future land use without regard to its existing condition may have an insufficient inflow and low water levels with stagnant water.
- A utility easement should be provided to allow maintenance access for sediment removal from the basin and forebay area.
- A dewatering outlet with a shutoff valve should be installed in the basin to allow the permanent pond and sediment forebay to be drained for structural maintenance.

Maintenance

Maintenance plans must be developed as a part of the planning process and agreements must be in place for inspection and maintenance. Maintenance programs should include provisions for routine inspections and housekeeping maintenance, special inspection and repair, nuisance control and sediment removal.

Routine inspection and housekeeping maintenance should be performed regularly and frequently. Activities should include removal and disposal of litter from the landscaped areas and any materials floating on the surface, removal of any materials clogging inlets or outlets and maintenance of vegetated areas through reseeding damaged areas, mowing and removal of tree seedlings.

Special inspection and repairs should be conducted annually and after each significant runoff event. Inspect and repair any eroded or slumping areas on or around the embankment, emergency spillway, inlet and outlet. Inspect for excessive deposition and identify and correct the source area. Inspect all inlets and outlets for needed repairs or clogging and repair if necessary.

Control of nuisance aquatic plants and mosquitos is critical to public acceptance of detention basins. These activities should only be conducted if a nuisance occurs or threatens. Mechanical controls should be used where feasible. Chemical control should be used sparingly and only if necessary.

Any maintenance plan must include provisions for sediment removal. Survey bottom elevations to determine the permanent pool depth and sediment depths in the basin. Remove and safely dispose of sediment as needed to maintain minimum acceptable depth for sediment storage. If the basin has a forebay, frequent cleaning may be necessary to prolong the life of the basin.

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The Wisconsin Storm Water Manual: Wet Detention Basins (G3691-4)

G3691-5

The Wisconsin Storm Water MANUAL

Artificial Wetland Storm Water Management Systems

Artificial wetland storm water management systems (AWSMS) consist of watershed conservation measures, constructed wetlands and some combination of sediment basins, grass filters, deep ponds and polishing areas designed primarily to remove contaminants from storm water. When practical, natural landscape features that provide water quality improvement functions may be incorporated into the system. The selection, combination and order of the AWSMS components overcome limitations often encountered by single component practices.

Advantages of AWSMS include (Schueler, 1992):

- Reliable pollutant removal
- Dampening of flood flows and peaks
- Creation of wildlife habitat and aesthetic potential

However, AWSMS may not be appropriate for every situation. Disadvantages include:

- A relatively high land requirement
- Significant management demands during and after establishment
- Time required for vegetation to mature and achieve optimum performance
- Potential for adverse impacts such as increased water temperature within sensitive watersheds

AWSMS are constructed systems that mimic the complicated, interdependent contaminant removal mechanisms of natural wetlands. It is important to remember that these artificial systems are designed primarily to treat storm water runoff. Although they may be enhanced to provide some of the other functional values of natural wetlands, these considerations are secondary to the system's pollutant removal potential and should not be included if they compromise the pollution control function. These artificial systems are not intended to mitigate the historic loss of natural wetland habitat. As such, artificial storm water treatment systems are not to be considered either restored or mitigation wetlands.

Conversely, natural wetlands, as well as mitigation and restored wetlands created to replace the full range of wetland functions, should not be used to treat storm water runoff. Although natural wetlands provide water quality benefits, discharging storm water directly to natural wetlands can have adverse impacts.

According to the U.S. Environmental Protection Agency (US-EPA, 1993), water level fluctuations affect wetlands and wetland functions adversely. When hydrologic changes or pollutants exceed the assimilative capacity of natural wetlands, wetlands become stressed and may be degraded or destroyed.

The Wisconsin Department of Natural Resources (WDNR) strongly discourages the use of natural wetlands for storm water treatment. In fact, new storm water discharges to wetlands under WDNR regulatory authority (Wisc. Admin. Code NR 103) are prohibited if an alternative to the discharge is available. Even where no reasonable alternatives exist, such discharges are not allowed if they will lead to significant degradation of wetland function.

Artificial wetland storm water treatment systems have been successfully constructed and operated in Midwestern states, including Wisconsin and Minnesota. Two Wisconsin projects include the Lake View project and the Delavan Lake project.

The Lake View Industrial Park project near Kenosha created a 600-acre wetland complex along the Des Plaines River. The complex includes a storm water management function and provides water quality benefits. At Delavan Lake, an 85-acre wetland was constructed on a 145-acre site. The drainage basin is 16.8 square miles. The wetland, immediately upstream from Delavan Lake, is designed to provide natural water treatment by removing sediments and nutrients before they reach the lake. The wetland system includes three sedimentation basins, a shallow marsh, a sedge meadow and wet prairie areas. Extensive sampling of the wetland will continue into the future and should provide data on the capabilities of a Wisconsin AWSMS.

When properly designed, constructed and operated, AWSMS remove pollutants from storm water and reduce peak flows reliably for many years. In addition, AWSMS provide wildlife habitat, aesthetic appeal and educational and passive recreational opportunities. To minimize adverse water quality impacts from storm water, all systems should incorporate practices that promote watershed conservation and pollution prevention.

To function effectively, AWSMS need to be properly designed, correctly installed and diligently maintained. The guidelines in this chapter should assist with these endeavors. Although "free water surface" and "subsurface flow" wetlands have been constructed to provide water quality improvements, subsurface flow AWSMS usually are not appropriate in Wisconsin due to wide fluctuations in storm water flow and seasonal variations. Therefore, this publication will discuss only free water surface AWSMS.

The guidelines have been drawn from an extensive review of national and state research, as well as the practical experience and insights of state storm water experts. It is important to realize, however, that research into AWSMS is an ongoing process and further changes in design can be expected.

Principles

Efficient pollutant removal

The principal pollutants found in urban runoff include sediment, oxygen-demanding substances (organic matter), nutrients (mainly phosphorus and nitrogen), metals (copper, lead and zinc), pesticides, hydrocarbons and trash or debris (US-EPA, 1993). The form and fate of each contaminant will be influenced by the design and geographic location of the AWSMS, the time of year, hydrologic conditions and other factors.

Studies investigating the effectiveness of wetlands to treat storm water runoff have been limited. Table 1 summarizes reported pollutant removal efficiencies for a variety of Midwestern natural and constructed wetland systems. The range of values illustrates the variability of the results and the complexity of the relationships between wetlands and water quality. In general, AWSMS are effective at removing suspended solids and pollutants that adsorb to solids, but are not as effective at removing dissolved pollutants (US-EPA, 1993).

Table 1. Average removal efficiencies for Midwestern storm water wetlands (adapted from Strecker et al., 1992)

Study & location	System name	System type	TSS	VSS	TN	TK N	Org. N	NH_3	NO_3	TP	Ortho. P	Dis. P	COD	PB	ZN	CU	CD
Brown 1985, Minnesota	Fish Lake	natural wetland & pond	95	78	-20		36	0		37		28					
	Lake Elmo	natural wetlands	88	80	38		-36	50		27		25					
	Lake Riley		-20	20	20		7	25		-43		-30					
	Spring Lake	constructed wetland		-300	-20	-14		11	-86		-7		-10				
Wotzka & Obert 1988, Minnesota	McCarrons Wetland Treatment System	constructed wetland & pond	94	94	83	85			63	78		53	93	90			
Hickok et al. 1977 Minnesota	Wayzata Wetland	natural wetland	94					-44		78				94	82	80	67
Scherger & Davis 1982 Michigan	Swift Run	natural wetland	76			20				49				83			
Barten 1987 Minnesota	Clear Lake	constructed wetland	76			25		55		54	52	40					
Oberts et al. 1989	Lake Ridge	constructed wetland	85	67	24	28			17	37	-5	8		52			
Minnesota	Carver Ravine	constructed wetland & pond	20	1	-6	-10			9	1	-3	1		6			
Hey & Barrett	Wetland 3	constructed	72						70	59							
1991, Illinois	Wetland 4	wetlands	76						42	55							
(Des Plaines	Wetland 5		89						70	69							
River Project)	Wetland 6		98						95	97							

Pollutant removal mechanisms

In general, AWSMS remove pollutants through physical, chemical and biological processes including absorption, adsorption, filtration, microbial transformation (biodegradation), precipitation, sedimentation, uptake by vegetation and volatilization. These are summarized in table 2.

Planning guidelines

Designing and constructing an effective AWSMS is a challenging task, requiring a sophisticated understanding of hydrology, soils and wetland plant ecology. The design of an AWSMS must be based on a careful analysis of many complex relationships and characteristics within the watershed and on-site. These include future land uses in the watershed, velocity and magnitude of flow, water depth and fluctuation, circulation, seasonal and climatic influences, groundwater conditions, soil permeability and the long term contribution of all systems in the watershed.

An AWSMS should be a component of larger landscape plans for watersheds and proposed developments. Upland prairie or forest buffers and grassed swale systems will enhance the quality and reduce the quantity of water reaching AWSMS. A comprehensive landscape approach also will increase the site's marketability. Aesthetically, the natural appearance of an AWSMS can provide an excellent amenity to a community or place of work. Local residents or property owners need to recog-

Table 2. Contaminant removal mechanisms in AWSMS (adapted from Watson et al., 1989 and Horner, 1992)

Mechanism	Description	Contaminant affected	Enhancement techniques
Absorption	Assimilation of gas, liquid, or dissolved substance into another substance	• phosphorus • synthetic organics • oil	• long residence times • low flow velocities
Adsorption	Adhesion of dissolved pollutants to suspended solids, sediments or vegetation. (Electrical attraction between positively charged pollutant particles and negatively charged particles such as sediments)	• phosphorus • heavy metals • synthetic organics	• shallow water depth • long residence times • sheet flow • Al and Fe soils (remove P) • Organic soils (remove metals) • circumneutral pH
Filtration	Physical entrapment of suspended particles by vegetation, biota and sediments	• organic matter • phosphorus • nitrogen • pathogens • heavy metals • suspended solids • synthetic organics	• sheet flow • low flow velocities • dense vegetation
Microbial metabolism (biodegradation)	Desirable modification of pollutants by suspended, benthic and plant-supported micro-organisms	• nitrogen • synthetic organics • organic matter • heavy metals	• scattered vegetation • permanent pool of water • high plant and soil surface area
Precipitation	Chemical reaction between dissolved pollutants and other elements in water that form insoluble substances that settle	• heavy metals • phosphorus	• low flow velocities • long residence times • high alkalinity
Sedimentation	Physical settling of particles and attached pollutants	• organic matter • phosphorus • nitrogen • pathogens • heavy metals • suspended solids • synthetic organics	• long residence times • low flow velocities • sheet flow • dense vegetation
Uptake by vegetation	Respirational uptake through plant tissue and conversion into plant biomass	• nitrogen • phosphorus • heavy metals	• dense vegetation (large surface area)
Vaporization	Evaporation to the air	• oils • chlorinated hydrocarbons • synthetic organics	• high temperature • air movement (wind) • turbulence • reduced surface films

nize that AWSMS are living ecosystems that can provide a variety of positive sensory experiences.

The hydrologic and site data generated from the planning phase site investigations will provide essential information for adequately sizing the AWSMS. To maximize pollutant reduction, the system needs to be effective during all seasons and under a wide range of hydrologic and pollutant load conditions.

To ensure that the construction of an AWSMS does not increase upstream or downstream flooding, it may be necessary to route design floods through the watershed. The effect of any modification to the existing surface and/or subsurface drainage system on upstream and downstream landowners should be evaluated. Drainage should not be adversely affected without obtaining the appropriate permissions. All applicable state and local laws and regulations pertaining to flooding (for example, floodplain zoning requirements and laws pertaining to surface and subsurface drainage) must be followed.

To prevent adverse impacts to existing wetland systems, AWSMS generally should not divert storm water flows around or away from areas that received the flow prior to land development. Water diversion could dry existing wetlands and reduce their value and functions. In general, AWSMS should attempt to maintain the existing hydrology of natural wetlands. If hydrologic changes are proposed, the impact of the proposal on existing hydrologic systems needs to be assessed.

Ideally, the site investigation and selection process should include the following (Brodie, 1989):

- Review of existing site information, including aerial photographs
- Preliminary field survey of the site
- Subsurface exploration and collection of environmental data
- Evaluation of data, potential environmental effects and regulatory requirements

Figure 1 illustrates a general methodology for selecting and evaluating a site. The most important factors to consider in the planning phase for an AWSMS include inflow water quality, land availability, hydrology, site substrate and site topography. Each factor is discussed below.

Inflow quality

To ensure that an AWSMS is appropriate for a site and that the design will optimize storm water treatment, information about the type and concentration of pollutants discharging to the site should be collected. Urban nonpoint source models can be used to estimate pollutant loadings and concentrations. Information on other pollution sources such as wildlife, atmospheric deposition and point sources should also be considered.

If it is doubtful whether the system can successfully treat the pollutant load it will receive, then another management practice should be selected. Because of their attractiveness to wildlife, AWSMS should not be used to treat water containing significant amounts of bioaccumulative or non-degradable contaminants. A periodic monitoring schedule should be established.

Land availability

There must be enough land available to handle, at a minimum, runoff from the tributary drainage area during the design rain. An estimate of land area requirements can be made using the method presented in the design guidelines section that follows.

Hydrology

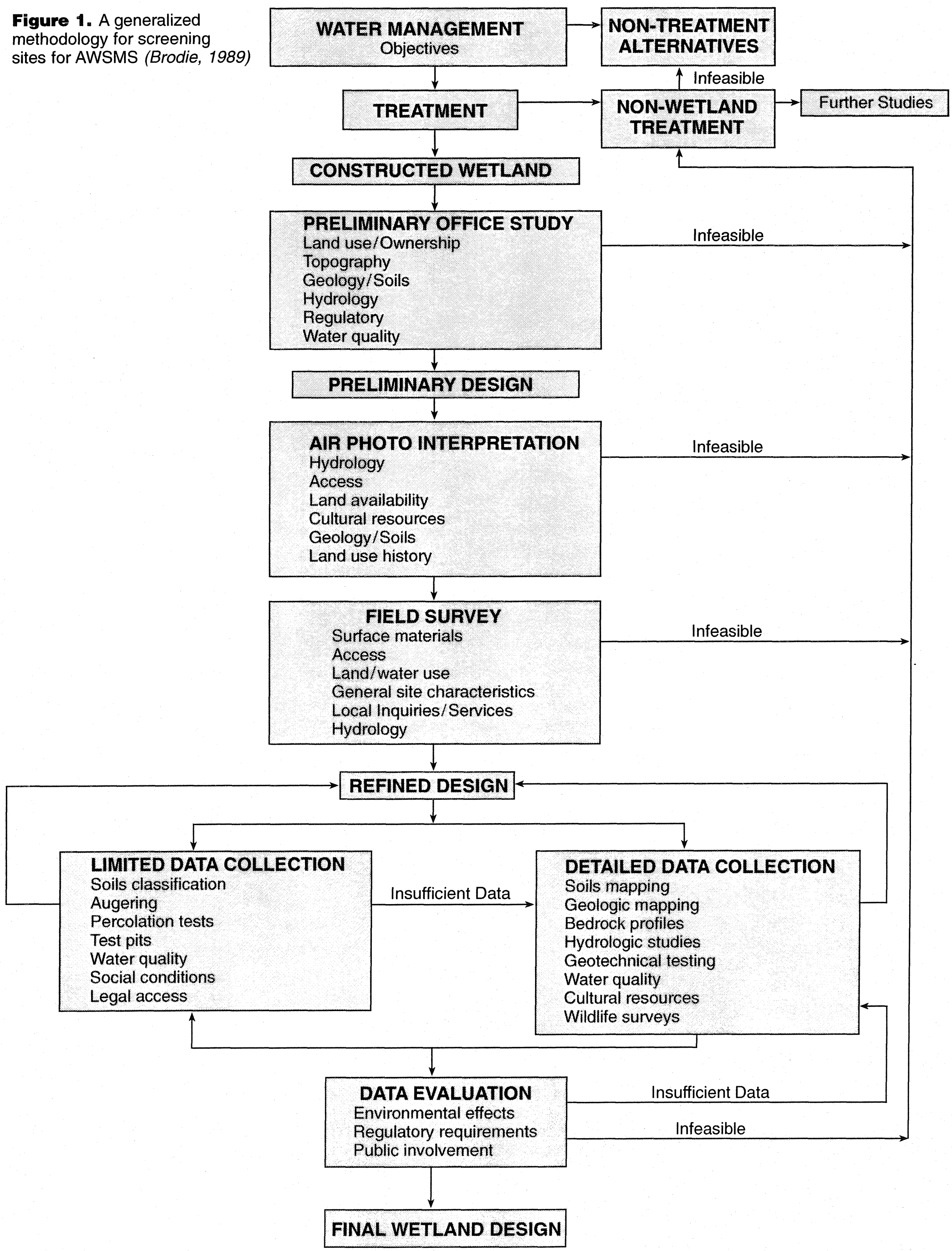
Hydrologic behavior is the most important site characteristic. If the proper hydrologic conditions exist or are developed, the chemical and biological conditions will, to a degree, respond accordingly (Mitsch and Gosselink, 1993).

AWSMS can be sited close to individual storm water sources or further downstream in a watershed. AWSMS sited in headwater areas will generally receive more irregular and less dependable inflows, potentially resulting in prolonged dry conditions.

This relative lack of flooding could prevent development of a healthy stand of wetland vegetation. However, this consideration must be weighed against the fact that if the AWSMS are distributed throughout the upstream portion of the watershed, less runoff and erosion might occur in the whole watershed as a result of storing water and sediments in the watershed uplands. Also, it may be that a landscape with a large upland buffering capacity and many small AWSMS may lend efficiency to pollutant and flow control because of the smaller amounts of contaminants and water that would need to be treated by each AWSMS.

Siting of the AWSMS further downstream in the watershed may result in increased flow to the system and less efficient pollutant removal. This must be balanced against the fact that an AWSMS located downstream in a watershed would be more likely to have permanent water and higher ancillary benefits due to more constant baseflows (Knight, 1992).

Figure 1. A generalized methodology for screening sites for AWSMS *(Brodie, 1989)*



A careful review of the watershed, including hydrology, topography and buffering capacity, should provide sufficient information to determine appropriate AWSMS sites. Adequate water should be present throughout the year to support wetland vegetation and functions.

Substrate

The ability of soils to retain water, support wetland vegetation and provide active exchange sites for adsorption of pollutants varies. Consequently, a site specific soil investigation must be completed. Investigations should provide information on soil thickness and depth, classification and composition, drainage characteristics, erosion potential and depth to bedrock or the water table. Variability in these conditions within the site should be considered. The Hydrologic Atlas series for Wisconsin, published by the U.S. Geological Survey (USGS), provides general information on surficial and bedrock geology and surface water and groundwater characteristics. For specific local information, driller construction reports should be consulted. Driller reports are available from the Wisconsin Geological and Natural History Survey (WGNHS).

Soil composition can affect AWSMS performance. For example, soils with greater extractable aluminum have greater potential for phosphorus removal than do organic soils. However, mineral soils generally have lower cation exchange capacity than organic soils. Organic soils can, therefore, remove some contaminants (such as certain metals) through ion exchange and can enhance nitrogen removal by providing an energy source and anaerobic conditions appropriate for denitrification (Mitsch & Gosselink, 1993).

Site soils and rocks should be evaluated for their use as construction materials for earthen embankments, spillways, riprap and liners. Such evaluations may include volume estimates, soil and rock sample analyses and erodibility (Brodie, 1989). In addition, soils must have sufficient stability to support embankments or other water control structures.

To ensure that the AWSMS will retain water, soil permeability rates must be estimated by conducting infiltration tests at the proposed bottom elevation of the AWSMS. Sites containing hydric soils (defined as soils that are saturated, flooded or ponded long enough during the growing season to develop anaerobic conditions in the upper part) should provide acceptable permeability rates. The USDA Natural Resources Conservation Service (NRCS) maintains lists of hydric soils. County hydric soil lists and soil survey maps may be obtained from NRCS field offices.

If hydric soils are not present, planners should investigate the feasibility of compacting the existing soil or providing a liner, and covering the area with peat, top soils, organic soils or soil amendments. Care should be taken not to accept material from sites with a history of receiving heavy pollutant loads.

Topography

To minimize costs, a location that requires minimal grading and excavating should be selected. To prevent erosion and maintain sheet flow, AWSMS should be located on sites with slopes less than 5%. Steeper slopes usually require more earth work and may be better suited for a detention basin than a wetland. USGS 7.5 minute topographic maps may be consulted for general information on area topography. For more detailed information, a site topographical survey should be conducted.

Design guidelines

Quantitative and qualitative consideration must be given to several design components. These include land requirements, configuration, substrate, hydrology, depth, gradient, pretreatment requirements, inlet and outlet structures and safety features.

An AWSMS should be designed either in combination with a detention pond or as an extended detention AWSMS. The pond/wetland (PW) system consumes less space than a wetland because the bulk of the treatment is provided by the deeper pond. This section discusses only the general design of ponds as related to AWSMS and will focus on the specifics of designing treatment wetlands. Guidelines for wet detention basins are found in *Wet Detention Basins* (G3691-2) of the *Wisconsin Storm Water Manual.*

Land requirements

The following calculation provides a rapid method for estimating the area of land needed to meet the treatment goals of the AWSMS. This calculation is based on the assumption that the AWSMS storage volume should equal the total runoff volume entering the AWSMS during the 1.5-inch rain event. The calculation is only an estimate because it is solely volumetric. It does not consider channel characteristics, slope, velocity, conveyance or similar parameters that would be evaluated in a hydraulic simulation model. However, for initial planning purposes, it should provide a valid approximation of AWSMS surface area requirements (Simon, et al., 1989). To ensure sufficient space for AWSMS structures and buffer zones, the calculated area should be increased by approximately 20%.

WE = (SA)(RU)/WD

WE = the approximate surface area of the AWSMS needed at the stage required to store water from the design storm (acres).

SA = tributary area of the watershed which will discharge to the AWSMS (acres).

RU = runoff depth predicted from the 1.5-inch rain event (feet).

WD = average storage depth of the AWSMS (feet) at the design capacity. For an approximation, the average depth can be assumed to be 2 feet.

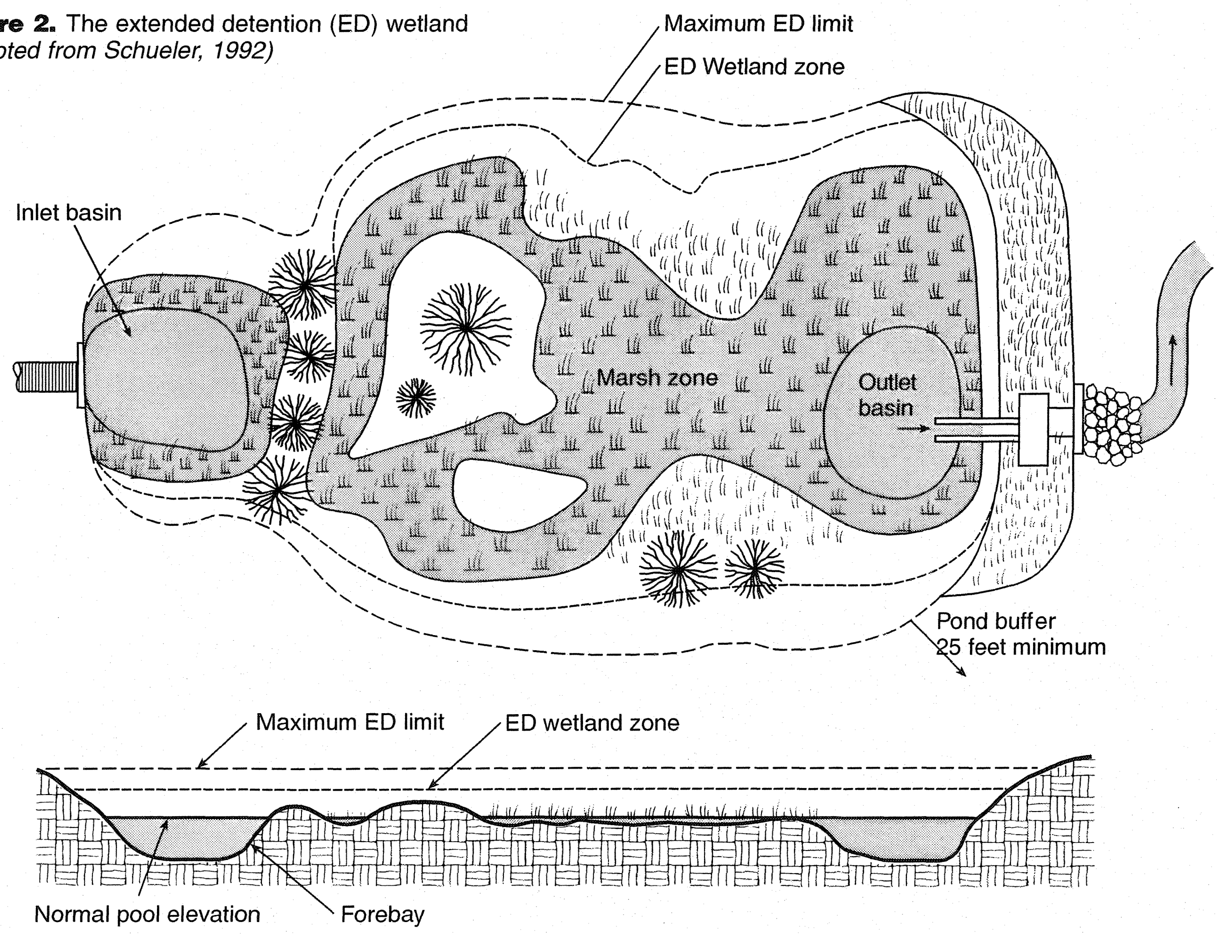
Configuration

To maximize treatment, the internal configuration of the AWSMS should be complex. Schueler (1993) reviewed nearly 60 monitoring studies that investigated the pollutant removal performance of storm water ponds and wetlands in North America. He noted that researchers consistently cited poor internal design geometry as the primary reason for low pollutant removal performance. The most common design problems included low length-to-width ratios, lack of pretreatment sediment basins, lack of structural complexity, inadequate treatment volumes and flow paths that tended to cause short circuiting.

To maximize treatment, designers should maximize the distance between the inlet and outlet and provide a high surface-area-to-volume ratio. To ensure adequate detention times, a length-to-width ratio of at least 3 to 1 is required. The configuration should enhance storm water distribution to maximize contact between storm water, substrate and vegetation.

The effectiveness of several removal mechanisms, such as sedimentation, adsorption and microbial transformation, are enhanced when AWSMS possess high surface area to volume (SA/V) ratios. The SA/V ratio can be increased by increasing surface area by adding internal structural complexity within the AWSMS (Schueler, 1992). A complex, extended detention AWSMS configuration is illustrated in figure 2.

Figure 2. The extended detention (ED) wetland *(Adapted from Schueler, 1992)*



The water level within an ED wetland can increase by as much as two feet after a storm event and then return to normal levels within 24 hours. As much as 50% of the total treatment volume can be provided as ED storage, which helps to protect downstream channels from erosion, and reduce the wetland's space requirement.

Many organisms, including valued fish species such as trout, have very low tolerances for temperature increases. If AWSMS outfall temperatures need to be reduced to protect sensitive downstream areas, the surface of the deep water areas should be shaded with floating-leaved plants such as Watershield *(Brasenia schreberi)*, Spatterdock *(Nuphar luteum)* and Duckweed *(Lemna spp.)*. However, it is important to note that shading AWSMS usually reduces the concentration of dissolved oxygen in the effluent. Providing a north-south AWSMS alignment may also reduce temperature impacts.

Substrate

The results of a site soil investigation should provide sufficient information to determine if the existing substrate will support an AWSMS or if the substrate must be modified. Care should be taken to get representative samples of the entire site, since some sites will have only localized areas of low permeability. If existing conditions are not appropriate, modifications should be considered to adjust site permeability and the growing medium for wetland vegetation.

First, sites containing soils that are too permeable must be modified to provide a subsurface permeability of 0.14-0.014 in/hr (US-EPA, 1988). These rates will effectively seal the AWSMS bottom. Sealing methods include providing a clay or plastic liner, compacting existing soils or relying on sediment deposition during AWSMS operation in NRCS hydrologic soil groups C and D (Horner, 1992).

However, if groundwater contamination is a concern, an impermeable liner should be in place before operating the AWSMS. Groundwater protection liners must be strong, thick and smooth to prevent root penetration and attachment (Steiner and Freeman, 1989).

If groundwater is within four feet of the liner, venting that allows gases on the bottom side of the liner to escape is recommended. This helps prevent the accumulation of gases that could form a large bubble and float the liner out of position in the AWSMS. Venting should also be provided in cases where groundwater fluctuations may trap air underneath the basin liner.

A second basic substrate modification involves placing hydric soils, peat, top soils (especially topsoil removed from other areas of the site during construction), organic soils or soil amendments on the liner or existing soils to provide the proper growing medium for wetland plants. To ensure enough room for roots to penetrate, the depth of soil (whether existing or modified) from the AWSMS bed to the liner should be at least one foot (Mitsch & Gosselink, 1993). Medium-fine textures, such as loams and silt loams, are optimum for establishing plants, capturing pollutants and retaining surface water.

Appropriate donor sites should be selected during the planning process. Hydric soils containing vegetative plant material or a seed bank may provide an excellent initial stand of vegetation. Although these soils enhance the diversity and speed of vegetative establishment, donor soil should not be collected from areas containing exotic species such as purple loosestrife. The donor material should be gathered at the end of the growing season if possible, and kept moist until placement (Shaver & Maxted, 1993).

Hydrology

Three basic hydrologic conditions should be considered in developing a functioning system:

1) An adequate water budget
2) Storage capacity
3) Hydraulic residence time for the critical hydroperiods when the system must function

Water budget

To estimate whether a site will receive and retain enough water to support an AWSMS, planners should collect sufficient information to estimate post-construction hydrologic inflows and outflows during each season for a variety of storm events. Because a permanent pool is essential, AWSMS should be built on a site only if inflows will equal or exceed outflows throughout most of each year.

The following hydrologic budget can be used to evaluate whether the site's hydrologic characteristics can support this system.

$$P + RO \geq ET + I + O$$

Where:

P = precipitation over the AWSMS
RO = surface runoff into the AWSMS
ET = evapotranspiration
I = infiltration to groundwater
O = outflow from the AWSMS

Precipitation data can be acquired from local weather stations. Surface runoff can be estimated by many models, as well as the simplified method described in the *Hydrology* section of this manual. Wetland evapotranspiration during the growing season can be estimated by multiplying reported Class A pan evaporation for the nearest evaporation station by a 0.8 conversion factor. Class A pan data are tabulated monthly and annually in *Climatological Data*, published by the U.S. National Oceanic and Atmospheric Administration, Asheville, North Carolina (Kadlec, 1989). Climatological data also may be obtained from the office of the Wisconsin State Climatologist. Infiltration rates should be obtained by conducting infiltration tests on the site.

If the AWSMS design will include a clay or synthetic liner to prevent groundwater contamination, infiltration can be assumed to be zero. These estimated inflows and outflows should provide enough information to determine if the current site conditions or site modifications will support the hydrologic requirements of a wetland.

These systems are generally dominated by surface water hydrology, but where groundwater inflow is a significant component it should be quantified as an input. A hydrogeologic investigation should be conducted prior to design to determine depth to high groundwater, groundwater flow direction and rate of flow, vertical and horizontal gradients, presence and extent of perched groundwater, soil descriptions, depth to bedrock and type of bedrock. Potential impacts on groundwater should be investigated. The unsaturated zone (below the ground surface and above the groundwater table) can remove pollutants depending on its thickness, particle characteristics and organic content.

Hydroperiod

Establishing the AWSMS hydroperiod is of primary importance because it determines the peformance and nature of the AWSMS. Hydroperiod is the duration of inundation measured over an annual wet or dry cycle. Seasonal and yearly patterns of flooding will be part of the hydroperiod of the AWSMS. Infrequent and non-periodic flooding and droughts are important for dispersing biological species to the AWSMS and adjusting resident species composition. After start-up, a variable hydroperiod exhibiting dry periods interspersed with flooding is a natural cycle. A fluctuating water level can often provide needed oxidation of organic sediments and can, in some cases, rejuvenate a system to higher levels of chemical reaction (Mitsch and Gosselink, 1993).

Storage

When designed for water quality control, the design volume of the AWSMS should be greater than or equal to the volume of runoff from a 1.5-inch rain under full projected watershed development. In addition, the storage volume must be sufficient to meet the peak flow discharge limitations for the 2-year, 24-hour design storm.

Hydraulic residence time

AWSMS size also should ensure an adequate hydraulic residence time (HRT), or detention of water in the AWSMS. The HRT is defined as the average time period for a particle to flow from the AWSMS inlet to the outlet. The HRT will vary with flow rate, seasonal and climatic influences, soil permeability, degree of mixing with water in storage and volume of available storage. The HRT and the amount of turbulence are important factors that affect the settling of suspended particles in the AWSMS (Martin, 1988). A site that is too small or has a rapid flushing rate provides poor trapping and may even cause re-suspension of previously deposited sediments during peak flows and the net export of particulates, nutrients and toxicants.

The theoretical detention time, which assumes a constant inflow rate and no dead storage volume, can be calculated as follows:

$$HRT = (V/Qf)$$

Where:

HRT = hydraulic residence time

V = AWSMS storage volume

Q = average discharge rate

f = void fraction (percentage of open water in AWSMS)

Kadlec et al. (1993) noted that wetland void fraction varied widely depending on the vegetation and short-circuiting of flow. A value of 0.75 is often assumed; however, a careful site analysis should provide a more accurate value.

The AWSMS should be designed to provide approximately 24 hours of detention for the 1.5-inch rain event. A 24-hour HRT will provide, at a minimum, the required 80% reduction in suspended solids from the water quality storm and peak discharge control of the 2-year storm. Extended detention facilitates denitrification (Silverman, 1989).

In addition, the velocity of flow passing through the vegetation of the AWSMS should be less than 1 foot per second in order to maximize treatment (Witthar, 1993). The design must ensure that runoff from larger storms does not wash sediments and nutrients out of the AWSMS into the receiving water.

Depth

To encourage diverse biogeochemical processes and plants, a variety of depth zones should be created within the AWSMS.

Shallow depths promote the growth and propagation of diverse wetland plants and improve the reliability of pollutant removal. Deeper water reduces vegetation growth and the effective contact time with both vegetation and soils. Generally, the shallow marsh will support the greatest density and diversity of emergent wetland plants and the highest surface area to volume ratio.

Target depths are useful in obtaining a range of depths within the AWSMS to increase the surface area to volume ratio, create nonturbulent flow conditions and increase the internal structural complexity of the AWSMS. Table 3 presents general target depth allocations for both the pond/wetland (PW) and the extended detention (ED) AWSMS designs (Schueler, 1992, MD-DNR, 1987).

Much of the sediment deposition may occur near the inlet as the incoming runoff velocity decreases upon entering the basin. Greater depths near the inlet help prevent sediment blockage and may facilitate cleanout.

The area of the shallow marsh should always be equal to or larger than the area of the deep marsh. During dry weather, the deeper AWSMS areas should contain a permanent pool approximately 2 feet deep that will minimize scour from large storms. The transition zone around the periphery of the normal pool should be very gently sloped so that it is temporarily flooded during most runoff events but drains as the detained runoff leaves the system. The area of this frequently flooded zone is greatest in the ED wetland system, where water elevations can increase 2 feet or more during storms. At capacity, the mean depth of AWSMS that conform to these depth allocations will be approximately 2 feet.

Design grades

The wetland area should be designed so that it has a very shallow sloping edge and a permanent pool. This configuration provides a variety of hydrologic conditions, with some areas permanently flooded and others temporarily flooded. These hydrologic conditions provide for the growth and propagation of diverse wetland plants and microbes and promote removal of both aerobic and anaerobic pollutants.

The AWSMS should be designed so that runoff entering the wetland will temporarily increase the normal pool elevation and spread over the transitional zone between wetland plants and upland vegetation.

This transitional zone is extremely important in terms of plant diversity, habitat and function. The maximum slope of the transition zone should be no greater than 10 horizontal to 1 vertical (10:1) and should extend at least 20 feet from the edge of the permanent pool (Shaver & Maxted, 1993).

To support vegetation and promote pollutant removal benefits, the maximum slope of both the shallow and the deep marsh should be no steeper than 10:1. To ensure stability, the maximum slope of the basins should be no steeper than 3:1. To minimize short-circuiting and ensure equal flow distribution, the lateral bed slope (across the width) should be zero (Watson & Hobson, 1989). The longitudinal bottom slope should also be essentially flat (no greater than 0.05%) (Hammer, 1992). To ensure that the AWSMS is aesthetically appealing, design grades should blend the newly created landform into the existing landscape.

Table 3. AWSMS target depths

Component	Depth below normal **Pool** (inches)	Percent of surface area PW AWSMS	ED AWSMS
Inlet basin	12–72	5	5
Outlet basin	12–72	5	5
Shallow marsh	0–12	20–25	20–40
Deep marsh	12–24	20–25	20–40
Transition zone (above normal pool)	0–24	5–15	10–30

Pretreatment components

Pretreatment components are designed to provide preliminary treatment of storm water before it enters the wetland component of the ASWMS.

Pretreatment may be an integral component of the AWSMS, helping extend the life of the wetland.

Without pretreatment, sediment may rapidly accumulate, smother vegetation and quickly decrease AWSMS storage and treatment capacity. Pretreatment will reduce stress on the aquatic components of the system, localize maintenance needs and can dampen flows through the system. Pretreatment practices should be considered at all locations where storm water runoff enters the wetland.

A sedimentation basin (similar to a forebay in a detention pond) can be used as a pretreatment component. Such a basin could be as simple as a trapezoidal trench 3 to 6 feet deep. Baffles and diversions should be strategically placed to prevent trapped sediment from becoming resuspended during subsequent storms. The basin design should include a hard bottom (compacted soil) and vehicle access so that accumulated sediments can be removed easily (Wengrzynek & Terrell, 1990; Horner, 1992).

The inlet basin should constitute at least 5% of the total AWSMS area. Where there are multiple inlets to the wetland, the total area of all the basins should be at least 5% of the AWSMS area with the individual inlet basins sized with respect to their percentage of contributing flow (Shaver & Maxted, 1993).

Additional pretreatment components may include a trash rack, an oil and grease trap and a grass filter. The trash rack is a grate designed to trap debris. The oil and grease trap may be necessary for AWSMS that will receive runoff from streets and parking lots containing concentrations of oil and grease. The grass filter may be used to trap sediments before entering the wetland and maintain sheet flow at the entrance to the wetland. The filter may include a subsurface tile drainage system to increase infiltration and maintain an aerobic root zone. Scheduled mowing and removal of grass maintains a dense sod and removes nutrients assimilated by plant growth. It is easier to reestablish grass filters than aquatic communities (Wengrzynek & Terrell, 1990).

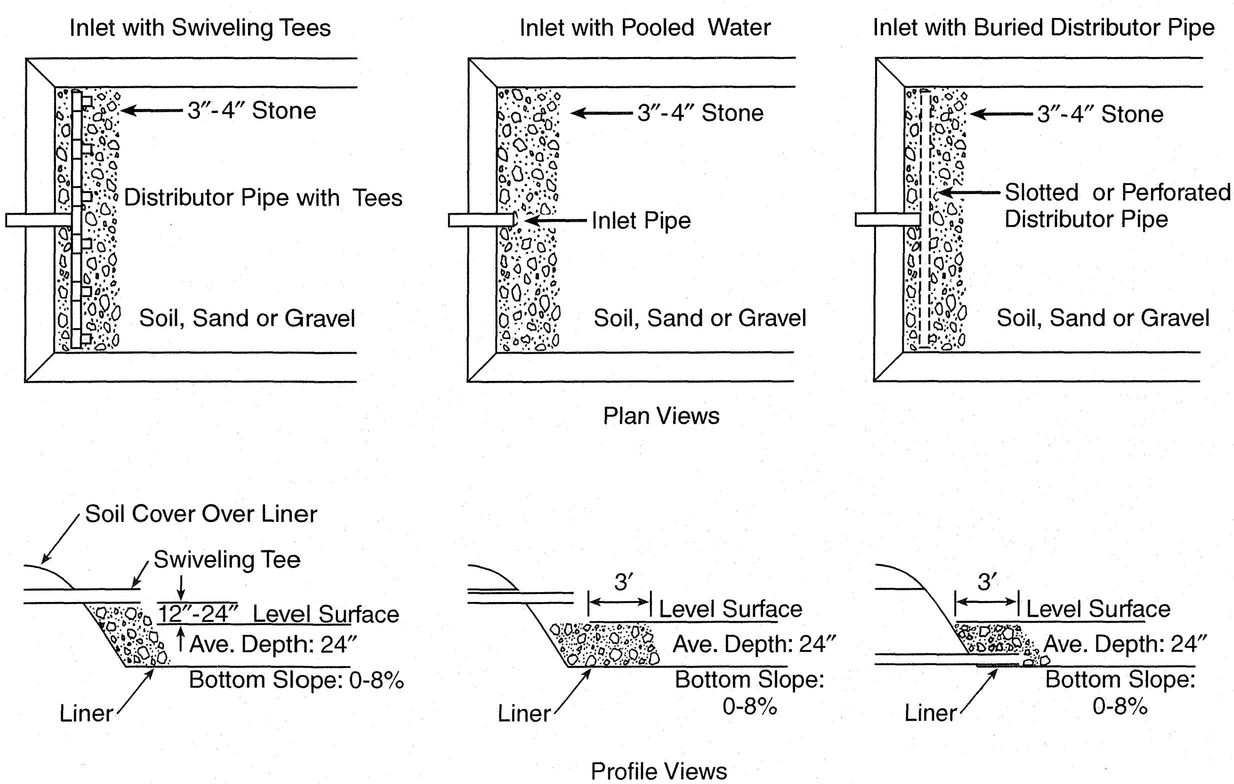
Inlets

Inlet structures should be designed to prevent high velocity discharges that could scour the AWSMS. Martin (1988) found that turbulence varies from storm to storm, depending directly on the inlet discharge structure, and less directly on rainfall intensity. According to Martin, highly turbulent inlet discharges scoured bottom sediments and caused increased pollutant loads at the outlet.

The AWSMS should also be designed to disperse, rather than channelize, flow through the system. Figure 3 depicts several types of AWSMS inlets. Controlled dispersion of the influent flow with proper diffuser pipe design can help to ensure low velocities for solids removal and even loading of the wetland so that anoxic conditions are prevented at the inlet area. The inlet can be designed so that water trickles over stepped riprap embankments to aerate the water. Use of limestone for the riprap will help buffer acid rain pH levels.

The inlet structure should be sized to handle the 2-year, 24-hour design storm flow and should be sited to minimize short circuiting. Hydrologic models should be used to estimate peak flows from design storm events for the contributing watershed. The inlet should be sized to release water at a velocity less than 1 foot per second (Witthar, 1993). An emergency spillway should be constructed to ensure that flows in excess of the design storm are safely diverted or discharged. This flow diversion structure allows the AWSMS to capture and treat the initial storm water runoff from storms larger than the 1.5-inch rain, and ensures that the hydraulic residence time needed for adequate treatment of runoff can be maintained.

Figure 3. Inlet designs for uniform stormwater distribution
(Source: modified from Watson and Hobson, 1989)



If the AWSMS configuration includes parallel cells, a flow splitter will be needed. A typical design contains parallel orifices of equal size at the same elevation in the splitter. Control orifices are sized according to the desired design flow. Options include pipes, flumes, and weirs. Valves are impractical because they require frequent adjustment. Flumes and weirs need not be standardized unless flow measurements are required. Flumes minimize clogging problems in applications with high solids, but are more expensive than weirs. Weirs are relatively inexpensive and can be easily replaced or modified to change flow to any cell.

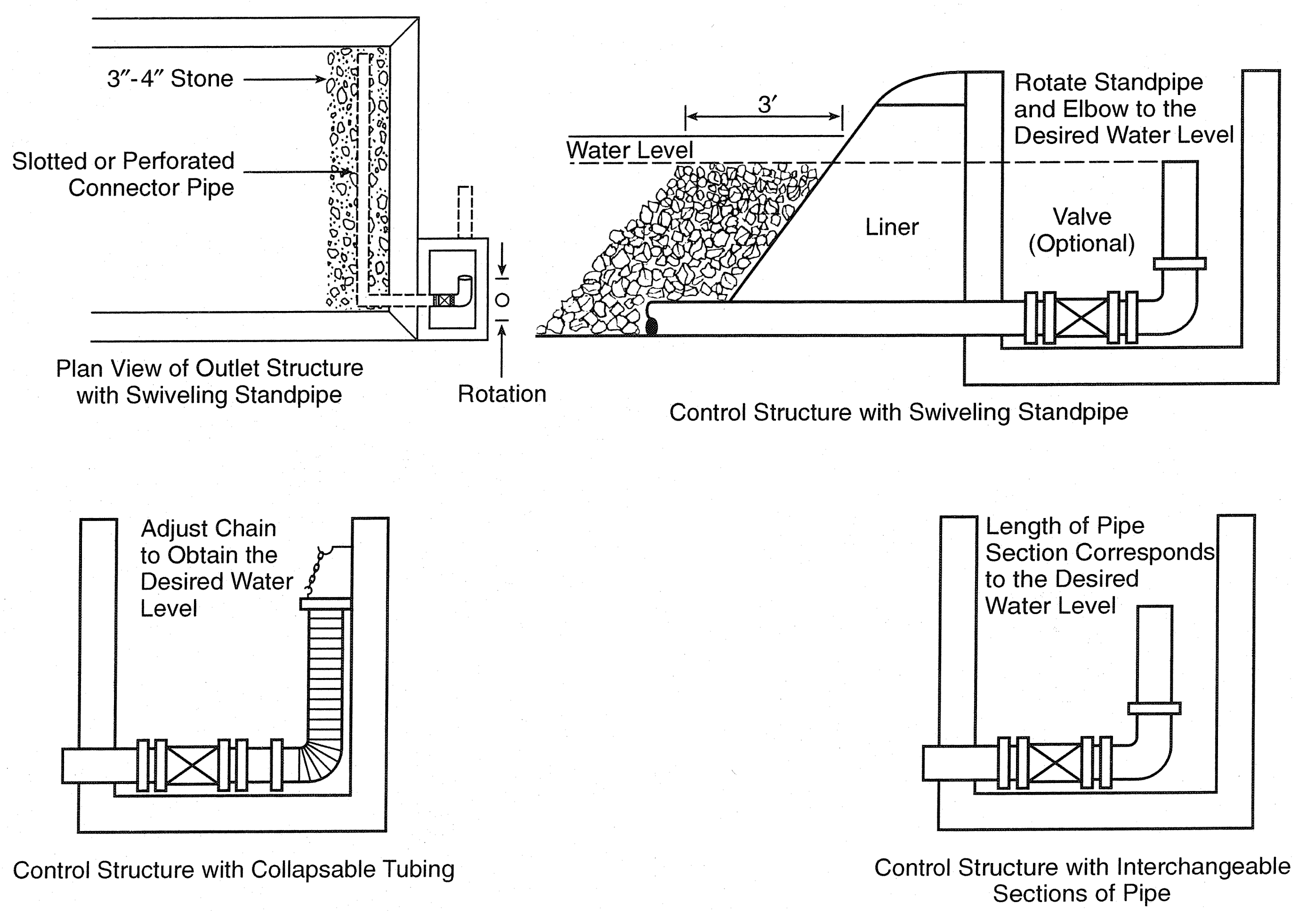
Outlets

The designer has several choices for the outlet of an AWSMS (figure 4). Where adequate maintenance and security can be maintained, outlets with removable boards may be used to control pool elevation. The use of such an outlet structure may not be appropriate in areas where vandalism may be a problem. Ideal design for water-level control allows water levels to be varied from zero (drained) to the maximum depth tolerance of desired wetland plant communities. If stop logs or weir plates are used, they should be of a type that effectively seals against leaks to help maintain water levels during periods of limited inflows. Multiple inlet and outlet weirs allow greatest hydrologic control and flexibility.

If an outlet pipe is used, it may become clogged by debris. To prevent this, a trash rack should be firmly attached to the upstream portion of the orifice. Another option is to install a reverse-sloped pipe about a foot below the permanent pool elevation. This outlet design has been found to avoid clogging (Schueler, 1992).

One drawback to this approach is the inability to see potential clogging of the pipe. All outlet pipes should include an adjustable gate valve to regulate outflow. In addition to the outlet pipe, it is advisable to install a drain capable of draining the AWSMS in 24 hours to allow for maintenance. The drain should be controlled with a lockable, adjustable gate valve, and an upward-facing inverted elbow placed on the end of the drain to extend above the bottom sediments.

Figure 4. Outlet water level control structures
(Source: modified from Watson and Hobson, 1989)



The AWSMS design should address potential problems associated with ice cover and frozen conditions. Provisions could be made for deepening water levels under the ice, draining the AWSMS and allowing baseflow to pass through quickly, routing water around the frozen AWSMS until spring thaw, or building a variable discharge outlet structure that gives flexibility depending upon winter conditions.

Vegetation

Vegetation is an essential component of all AWSMS. Microbes that transform nutrients attach to the substrate provided by vegetation. In addition, vegetative growth serves as a barrier that reduces the velocity of incoming storm water, promotes sedimentation, reduces the probability that sediments will resuspend, takes up nutrients and metals and filters incoming particulates. Decayed vegetation increases the organic content of the sediments, which promotes anaerobic decomposition and improved nitrogen removal.

AWSMS vegetation can be planted or the constructed basin can be left unplanted with the expectation that suitable vegetation will eventually develop. The primary reasons for planting vegetation are to influence the species composition and/or to establish a vegetated AWSMS as quickly as possible. Other reasons for attempting to influence species composition include water quality considerations and provision for wildlife habitat, recreational opportunities and aesthetic appeal. Establishing wetland plants requires time and money, and the plants' long term survival is uncertain. However, leaving a site open until natural colonization occurs has several disadvantages. The site will be more susceptible to erosion, and invasive exotics are more likely to colonize and dominate the site. Unless a suitable seed bank of desired wetland plant species is present on-site, it is recommended that vegetation be planted.

Because AWSMS are expected to receive wide fluctuations in inflow water quantity and quality, robust species, like cattails and bulrushes (excellent plants for water treatment) should thrive. Although the development of "high quality" species might be desirable, sensitive species cannot be expected to survive the rigors of an AWSMS. Remember, AWSMS are designed for water quality control, not necessarily to provide diverse, unique vegetation.

To avoid negative impacts to nearby natural wetland areas, native, non-invasive species should be planted in AWSMS vegetative communities. Designers should select plants adapted to the local environment, commercially available, fast growing, requiring little maintenance and, to the extent possible, aesthetically pleasing to encourage the neighborhood's acceptance of the AWSMS. Plants should also tolerate flooding, frequent saturation, low oxygen levels, high nutrient levels and variable conditions. Species that have a large stem surface area per unit bed area will provide the greatest area for storm water contact and microbe growth. Dense-growing species will reduce flow velocity and increase sedimentation and filtration. The tolerance of vegetative species to soil moisture levels may be relatively narrow, and the selection of vegetation must take this sensitivity into account.

The natural development of plant communities in zones corresponds closely to the water conditions of the AWSMS. Zones include the upland buffer, the transition zone, the storm water basin, the grass filter and the wetland itself. The wetland is further divided into zones including the emergent, submergent and floating plant areas. Additional information about shoreland plants and landscaping is available (UWEX, 1994).

Other structures and features

Extending the flow path through the AWSMS by adding dikes, berms, shallow marsh areas and multiple cells will minimize short-circuiting and enhance pollutant removal rates (Schueler, 1992). Finger dikes are commonly used in surface flow systems to create serpentine configurations and can be added in operating systems to mitigate short-circuiting. Divider dikes separate cells and attain desired length-to-width ratios.

Details for these structures are based on site-specific needs and objectives. Most are constructed of native soils, but finger dikes may also be constructed with sandbags or treated lumber. Dike design should meet the requirements of the NRCS Engineering Field Handbook (USDA-SCS, 1992). Dike freeboard should accommodate an organic matter accumulation rate of 0.5–1.5 inches per year in the AWSMS. Adequate freeboard and water level control is also necessary to provide capacity for flow beneath the expected thickness of ice cover (Hammer, 1992).

If the system is large enough, an island in the pond can act as a baffle and extend the flow path length through the AWSMS. Open water areas can be created by excavating about 3 feet below normal water level and deeper excavations can provide greater hydraulic residence times. To ensure adequate treatment, vegetated areas should greatly exceed open water areas. To prevent hydraulic short-circuiting, open water areas should not be connected along the flow path, but rather interspersed with densely vegetated shallow marsh habitat (Knight, 1992).

A buffer should be provided around the wetland both to separate the treatment area from developed areas and to reduce impacts on wildlife. The minimum buffer width should be 25 feet, measured from the maximum water surface elevation. The buffer should be sloped no steeper than 5:1. At least 75% of the buffer should be forested to discourage geese and provide better protection and habitat (Horner, 1992).

AWSMS are usually constructed by excavating and/or constructing berms. In general, AWSMS should be excavated into existing grades without the need for extensive berming. Structures such as distribution systems, berms, liners and weirs should be designed and constructed to provide reliability, safety and reasonable cost according to standard engineering techniques. Appropriate structural design and construction information is detailed in the *NRCS Engineering Field Handbook* (USDA-SCS, 1992) and *Technical Guide* (USDA-SCS, 1994).

The design should provide maintenance access roads as needed. Access roads should be designed with minimum 15-foot wide right-of-ways and slopes no steeper than 5:1. The road should be stabilized to withstand heavy equipment.

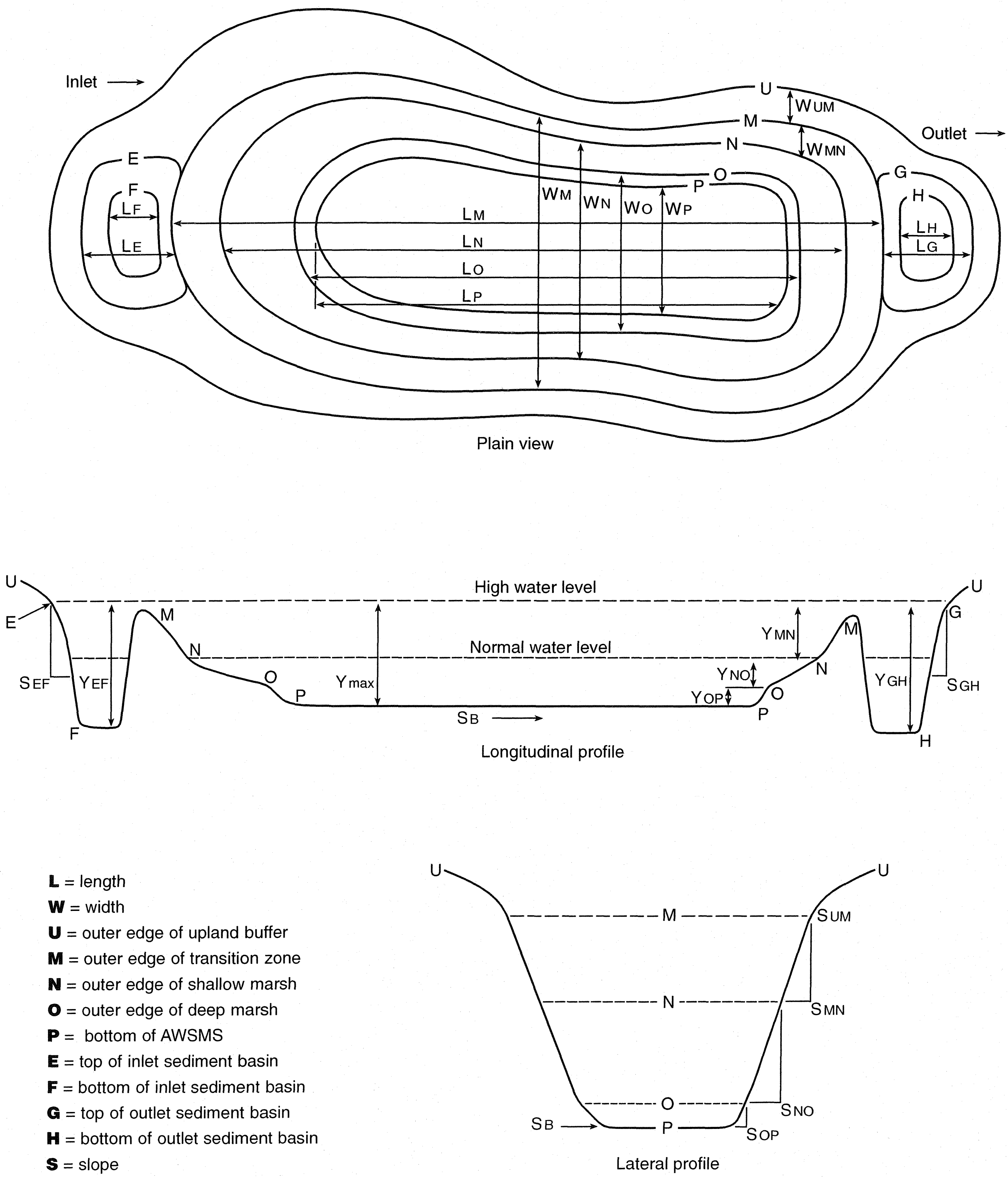
A polishing filter (a stable, relatively level vegetated site that may be grassland, wetland or forested area, either natural or constructed) may be desirable between the AWSMS outlet and the receiving body of water. This area serves as a final filter or buffer between the AWSMS and the receiving body of water.

One drawback to AWSMS, particularly in densely inhabited areas, is the potential for increased mosquito populations. This problem is minimized when water is continually flowing through the AWSMS. Manipulating the water level can also help control mosquito populations.

Safety features

The shallow depth of an AWSMS minimizes, but does not eliminate safety hazards. The shallow transition zone will promote dense vegetation growth which, in turn, will act as a natural barrier to the deeper permanent pool. Water deeper than 3 feet should not be easily accessible. The deeper area near the inlets and outlets should be constructed with safety shelves and be far enough away from the embankment so that the water is shallow in areas where there is access to the pond.

Figure 5. Simplified conceptual AWSMS dimensional design (not to scale)



Design calculations

A simple, conceptual AWSMS that illustrates the principles discussed is provided in figure 5.

Design guidelines and typical values for the design AWSMS are presented in Table 4. Each design may be unique, so the standards used in a particular design need to be carefully considered.

Analytical solutions for AWSMS dimensions satisfying the previously presented guidelines are cumbersome. Due to the above constraints, many AWSMS dimensions become fixed when a length to width ratio and mean top length or width have been selected.

Because the configuration of the AWSMS should be irregular, the mean length and mean width can be used to approximate AWSMS area. Throughout the remainder of this discussion, lengths and widths will be treated as means. Actual construction will usually follow more irregular configurations. Because depths and slopes of the wetland area are somewhat flexible, they should be selected prior to employing the following analytical solution. The following trial-and-error procedure, adapted from Walker (1987), may be employed to find the length to width ratio and the length or width to satisfy the design criteria.

Table 4. Design guidelines and typical values.

Parameter	Typical design guideline
Total suspended solids	80% removal efficiency
Peak flow	Pre-development 2-yr peak flows ≥ post-development 2-yr peak flows
Volume at capacity	Runoff volume from a 1.5 inch rain (ED wetland) or volume of runoff from detention pond at capacity (PW wetland)
Hydraulic retention time	≥ 24 hours
Void fraction	≈ 0.75
Velocity	≤ 1 ft/sec
Infiltration rate	≤ 0.14 in/hour
Total sediment basin area	= 0.1 total AWSMS area A (acres)
Area of shallow marsh	≥ area of deep marsh
Mean depth at capacity	≈ 2 ft
Maximum wetland depth	≤ 4 ft
Transition zone depth	≤ 2 ft
Shallow marsh depth	≤ 1 ft
Deep marsh depth	≤ 1 ft
Inlet sediment basin depth	≤ 6 ft
Outlet sediment basin depth	≤ 6 ft
Upland zone slope	≤ 5:1
Transition zone slope	≤ 10:1
Shallow marsh slope	≤ 10:1
Deep marsh slope	≤ 10:1
Inlet sediment basin slope	≤ 3:1
Outlet sediment basin slope	≤ 3:1
Lateral bed slope	= 0
Longitudinal bed slope	≤ 2000:1 = 0.05%
Transition zone width	≥ 20 ft
Upland buffer width	≤ 25 ft
Length to width ratio	3:1

Select trial values for the length and the length to width ratio.

Initial values of length (L) and width (W) may be estimated using an assumed length to width ratio (R) and the recommended mean depth of 2 feet (D). For example, if the design volume (V) is 2 acre-feet and a length to width ratio of 3 is selected, then:

$V = (L)\ (W)D$
$\quad = (L)\ (1/R)(L)(D)$

(2 acre-feet) (43,560 square feet/ac.)
$\quad = L(1/3L)(2)$

L = 362 feet

and

$W = 1/R(L)$
$\quad = (1/3)(362 \text{ feet})$

= 120 feet

The estimated surface area would be:

$A = (L)(W)$
$\quad = (362 \text{ feet})(120 \text{ feet}) = 43{,}440$ square feet

Site topographic features should also be considered to determine if these dimensions are compatible with topographic features.

Determine other dimensions

After trial R and L values have been selected, other AWSMS dimensions can be calculated. Designers often assume that the area of the sedimentation basins can be approximated by squares, that the area of the wetland can be approximated by a rectangle, and that the areas of the inlet and outlet sediment basins are equal. However, depending on water quality and quantity, it may be appropriate to provide a larger inlet than outlet. If the calculated values of intermediate lengths or widths are less than zero, design constraints are not feasible. The designer must return to Step 1 and adjust R and/or L or adjust wetland slopes and depths. Volumes are calculated using the average end area method where:

$V = ((A_u + A_l)/2)(D)$

Where V is the volume,

A_u is the area of the upper surface,
A_l is the area of the lower surface, and
D is the depth between the two surfaces.

To illustrate these calculations assume that a length/width ratio of 3 and a length of the upper water surface in the zone is 250 feet (Lu). The depth of water in the zone under consideration is 1.5 feet (D) and the slope of the wetland in the zone is 10:1 (z). Calculations to determine water volume would be:

Width of upper surface

$W = L/R = 250 \text{ feet}/3 = 83 \text{ feet}$

Area of upper surface

$A_u = (L_u)(W_u) = (250 \text{ feet})(83 \text{ feet}) =$ 20,750 square feet

Length of lower surface

$L_l = L_u - (2)(z)(D) = (250 \text{ feet}) - (2)(10)(1.5 \text{ feet}) = 220 \text{ feet}$

Width of lower surface

$W_l = W_u - (2)(z)(D) = (83 \text{ feet}) - (2)(10)(1.5 \text{ feet}) = 53 \text{ feet}$

Area of lower surface

$A_l = (L_l)(W_l) = (220 \text{ feet})(53 \text{ feet}) =$ 11,660 square feet

Volume in zone

$V = ((A_u + A_l)/2)(D) = ((20{,}750 + 11{,}660)/2 \text{ square feet})(1.5 \text{ feet}) =$ 24,308 cubic feet

This volume would need to be adjusted by the void fraction to determine the effective volume. Similar calculations can be made for each zone in the wetland.

Test results

The final step is to determine whether the total volume and mean depth calculated satisfy the design requirements. If so, no additional iterations are necessary. If design requirements are not met, return to step 1 and adjust the trial values of length, the length/width ratio or the slope steepness.

For example, using the same length to width ratio, the volume can be increased by increasing the length. The use of a spreadsheet to make the calculations will greatly facilitate the iterative calculations required to meet design volume requirements.

Design AWSMS configuration

As stated previously, the AWSMS configuration should be irregular, complex and blend into the natural landscape as much as possible. Areas and dimensions estimated by the above calculations should be used to guide the creation of design plans for appropriately shaped AWSMS. The design plans should be checked to ensure the AWSMS will provide sufficient storage volume and meet all design guidelines. The equations used for calculating volume increments are also applicable to irregular contours. The areas needed for the calculations should be derived from the contoured design plan using planimetry.

Construction guidelines

Proper construction is critical to efficient operation of the AWSMS. Careful supervision is imperative to ensure that grade and elevation specifications are met. Otherwise, considerable difficulty with short-circuiting and reduced treatment capacity may occur and be difficult to correct later.

Construction plans and specifications for the AWSMS should be based on information presented below. The level of detail depends on the size and complexity of the AWSMS, the physical characteristics of the site and the requirements established by regulatory agencies.

At a minimum, construction plans must include the following to ensure sufficient detail for accurate bid preparation and construction:

- Boundaries of construction activities, including clearing and grubbing limits
- Construction and maintenance access road
- Location of existing utilities (overhead and underground)
- Erosion control measures
- Quantities, locations and boundaries of borrow areas
- Trees and vegetation that will be left undisturbed
- Wildlife habitat enhancement structures
- Location, design plans and specifications, elevation, freeboard, upstream and downstream slopes, materials and permeability requirements for dikes, berms inlets, outlets and other structures
- Spillway location, elevation, type and design specifications
- Size, location, elevation, materials and type of water control structures
- Methods for determining permeabilities and other contract specifications
- Permeability requirements for pond bottom and sides including type, location and installation of liners if needed
- Elevations, contours and slopes for the AWSMS
- Location of subsurface drains
- Method of placement and type of rock, gravel, soil and limestone by elevations and depths
- Species, spacing, sources of supply and planting dates of wetland vegetation
- Seeding, mulching, sodding, liming and/or fertilizing requirements for dikes, berms and any other disturbed areas
- Provisions for on-site construction supervision
- Types and sizes and of construction equipment
- Location of endangered or threatened species, if any, and measures to avoid their disturbance.

A pre-bid conference with potential contractors is recommended to explain the concept, goals and requirements of the project. This meeting can be effective in soliciting accurate bids from qualified contractors (Tomljanovich and Perez, 1989). A pre-construction meeting with the selected contractor is also highly recommended.

Good construction techniques include use of correct equipment such as light foot-pressure, tracked vehicles for working on soft substrates, suitable soil placement equipment to achieve design grades, and suitable site preparation and planting equipment, which may range from standard farm equipment to bulldozers.

Except for liner compaction, wetland soils should not be compacted during excavation and grading. Compaction may limit root and rhizome penetration. The substrate should be soft enough to permit relatively easy insertion of plants into the soil. If the wetland soil is compacted, the soil should be disked or otherwise physically disturbed before planting and flooding.

Plants can be introduced by planting seeds, roots, rhizomes, tubers, seedlings or mature plants obtained commercially or from other sites; importing substrate and its seed bank; or relying completely on the seed bank of the original site. If permission is granted by the appropriate authorities and landowners, plants may be collected from nearby wetlands. Collecting wetland plants from public lands or public waters requires prior notice to the local DNR office, and additional restrictions may apply.

Erosion during construction should be minimized. It is particularly important that upstream construction areas implement effective erosion control plans, so that the AWSMS does not become overloaded with sediments. Upland drainage diversion structures, which pass upstream flows around the AWSMS until the site is stabilized, should be constructed.

Implementation of an effective erosion control plan during AWSMS construction will mitigate downstream impacts. The AWSMS should be constructed and planted prior to excavating the connection to the outflow channel. Refer to the Wisconsin Construction Site Erosion Control Manual for further guidance (WDNR, 1993).

Maintenance

Artificial wetland storm water management systems require maintenance for optimal performance. A detailed operation and maintenance manual, including an excavation and disposal plan for sediments, should be developed prior to construction. The manual should establish a schedule for monitoring and maintenance and, to ensure accountability, designate short- and long-term operation and maintenance responsibilities.

Operation and maintenance must be conducted by personnel familiar with the operation and maintenance manual and who know how to achieve the objectives of the AWSMS. The manual can be updated to reflect specific system characteristics learned during system operation.

The AWSMS should be designed so that maintenance needs are minimal. AWSMS are living ecosystems that will naturally evolve with time. It is important to remember that AWSMS do not become functional immediately upon construction; several years may elapse before nutrient retention is optimal.

During the one- to two-year start-up period frequent inspections and maintenance must be completed. To avoid adverse impacts to the AWSMS, the transition zone should not be mowed because grass clippings will increase nutrient loading to the AWSMS. Fertilizers and herbicides would similarly stress the system (Shaver & Maxted, 1993).

Periodic maintenance includes removing debris and litter (particularly at the inlets and outlets), monitoring water levels and plant vitality, providing structural repairs and erosion control, collecting and analyzing water quality samples, excavating and disposing accumulated sediment in the sedimentation basin and adjusting the inlet and outlet structures.

Over time the soils and vegetation of the AWSMS may reach a saturation point, limiting its ability to remove pollutants from the water. In addition, sediment accumulation could result in a loss of ponded portions of the AWSMS, or the formation of shallow channels that could reduce residence time and the mixing of storm water with pond water.

The frequency of sediment basin cleaning depends on the sediment load entering the AWSMS. Each basin should be inspected annually to ensure timely cleanout. When the sediment basin has filled to approximately 50% of its total volume, sediment should be removed, placed in an appropriate upland location and stabilized.

Planning an on-site sediment application area will save disposal costs. Cleaning pretreatment facilities such as sediment basins will significantly reduce the frequency of sediment removal needed in the AWSMS.

Maintenance remedies are available to address sediment accumulation. In some cases, the elevation of the water level in the permanent pond can be raised by raising the height of the outlet. This procedure can be repeated until the peak storage volume requirements of the basin are in danger of being compromised, at which time sediment excavation will be required to extend the life of the AWSMS.

Removal of the sediment by excavation requires draining some of the AWSMS water and could result in considerable damage to the wetland vegetation. If pretreatment measures are effective, sediment removal from the wetland should occur only infrequently. Even with cleanout, replanting may not be necessary because of the buildup of seeds within the basin.

Harvesting wetland vegetation is not recommended in Wisconsin. The disadvantages of harvesting include limiting the AWSMS configuration to that which is accessible by large equipment, increased maintenance costs and the need to dispose of the harvested vegetation. The AWSMS should not be burned because burning will release nutrients to the water.

Proper operation and maintenance of the AWSMS depends on a monitoring plan that provides information for judging the attainment of treatment objectives, performance efficiency and long-term viability. Basic elements of the monitoring plan include:

- Clearly stated treatment goals and monitoring objectives
- Statements of organizational and technical responsibilities, tasks and methods
- Quality assurance procedures
- Schedules
- Reporting products
- Resource requirements
- Budget

A well-conceived and clearly defined monitoring plan serves as a point of reference and source of perspective for maintaining a meaningful information base throughout the life of the project.

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The Wisconsin Storm Water Manual:
Artificial Wetland Storm Water Management Systems (G3691-5)

G3691-6

The Wisconsin Storm Water MANUAL

Filter Strips

The terms filter strip and buffer strip are frequently used interchangeably to describe areas of vegetation that filter sediment, organic matter and other pollutants from surface water or groundwater. However, there is a difference between the terms.

Buffer strips are vegetated areas located alongside streams or lakes, designed to optimize habitat and food supply for aquatic plants and animals. Often undisturbed, they act as screens for water bodies. They can prevent streambank erosion, provide shade for light and temperature control and filter nutrients, sediment and other particulates.

Filter strips, on the other hand, are more highly engineered vegetated strips intended to "treat" storm water runoff. A filter strip may be located downslope of an impervious or disrupted site, and be designed to remove sediment, nutrients and other particulate pollutants. They are often used in combination with other management practices as a pretreatment unit.

Filter strips are often graded for optimum slope and planted in grasses that tolerate wet conditions up to 24 hours. An existing natural area may meet the site requirements and can be dedicated as the treatment unit with little modification. Minimally, the natural area will require grading around its perimeter and provisions for a level spreader. The benefit of a filter strip as a wildlife habitat is secondary to its treatment capability.

Filter strips differ from grass swales in the type of flow they are designed to handle. They should be designed for sheet flow over a fairly level, rectangular, vegetated area as opposed to grass swales that carry concentrated flow through concave, vegetated channels. Historically, swales have served as conveyance devices, but now they are used both for water quality and water quantity control. Filter strips reduce the flow velocity, increase the time of concentration, remove pollutants and infiltrate runoff into the soil.

While there is some overlap in the operation of a filter and a buffer strip, this manual concerns treatment management practices and therefore focuses on filter strips.

Principles

Filter strips are capable of treating water quality volume from a small drainage area. As water flows through a well-designed filter strip, sediment and pollutants are removed by filtering, infiltration and settling of particulates due to the slow water velocity. Because of the low flow velocity and the high percentage of sheet flow receiving filtration, pollutant removal rates through filter strips should be higher than through a grassed swale. However, field monitoring shows highly variable soluble pollutant removal and sediment, and nutrient removal rates similar to a swale (SEWRPC, 1991). Research is also unclear on whether grassed or wooded filter strips are more effective in removing pollutants. Forest floor organic matter can trap and hold pollutants more effectively, but forested filter strips do not provide a dense grass mat and may therefore need to be longer to achieve the same detention time (Schueler, 1987).

Planning guidelines

Removal efficiencies vary with the nature of the material being removed, length and slope of filter, soil permeability, size and characteristic of the drainage area, type of vegetative cover, and runoff velocity. A well designed facility is one where sheet flow is maintained and the runoff velocity is in the low to moderate range (less than 2.0 feet per second). In highly urbanized areas the incoming flow velocity is often too high for effective removal of sediment. A device upslope of the strip may be required to slow the influent velocity or to spread the inflow uniformly over the width of the filter.

Filter strips are best used in combination systems where they act as pretreatment units for infiltration basins and trenches. They function well in low slopes and residential areas of 5 acres or less. Multiple strips can be put in for drainage areas greater than 5 acres, but no one strip should receive flows from more than a 5-acre area. To improve the removal efficiency, the engineer can decrease the slope steepness or increase the length of the strip and improve sheetflow through the filter.

Like other infiltration practices, filter strips lower runoff velocity, lengthen the time of concentration and contribute to groundwater recharge. More specific advantages include low cost, ease of maintenance and a relatively low land requirement for the pollutant removal achieved. If an existing area is available on-site, the cost can be nearly negligible.

Sheet flow is critical to successful filter strip operation. However, filter strips have a high potential for channeling flow through the filter, which reduces efficiency and may result in failure. A shortened life span can be caused by lack of proper maintenance, improper location, poor vegetative cover and poor design (for example, not large enough for the contributing area). Filter strips have limited ability to control runoff or to remove dissolved nutrients. Finally, filter strips cannot reliably achieve total suspended solids removal rates comparable to a primary treatment facility. Therefore, they are recommended for use in a train with infiltration or detention systems (Schueler, 1992).

Design considerations

The most critical design consideration is the maintenance of sheet flow across the filter strip. Flows must be spread through the use of a level spreader, stone trench or curb cuts to create sheet flow at the head of the filter strip. The drainage area contributing to the filter strip should be less than 5 acres or the flow may be too great, increasing the likelihood of channelized flow. Design velocities are usually less than 3 feet per second, with velocities less than 2 feet per second desired for maximum effectiveness.

Pollutant removals will be greater if infiltration occurs at the site, which depends on soil permeability. Groundwater protection should be considered in the selection of an infiltrating filter strip. Recommendations for site evaluation include determining depth to groundwater and bedrock, classification of the soils and an in-field evaluation of the infiltration rate. The slope of the strip itself will affect pollutant removal rates. The strip should be as flat as possible without causing standing water, although slopes up to 10% have been used. If the site is favorable for infiltration and the filter strip is designed for it, the same site restrictions listed in the infiltration basin section of this manual are applicable.

Three different approaches to determining the minimum length of the filter have been suggested.

1) The width of the strip (perpendicular to the flow) should be a minimum of 20 feet. The length should be 50–75 feet with an increase of 4 feet for every one percent of slope on the strip (Schueler,1992).
2) Design for a 20-minute detention time (or travel time through the strip) to achieve 85% removal of TSS, as reported by EPA for overland flow treatment of wastewater (US-EPA,1980).
3) Use the same length in the filter strip as in the contributing impervious area (Galli, 1992).

The best approach to use in calculating strip length is still subject to debate.

Vegetation should be selected for ease of establishment, ability to create a dense mat, erosion resistance, water tolerance and non-invasive qualities. Contact local authorities for recommendations for your area and conditions. Maintain the grass at a height of 6–12 inches (Schueler, 1992). Frequent inspection of the filter and periodic maintenance are important.

The desirable depth of flow in a filter strip is 0.5 inches across its surface area to maintain sheet flow. Depths greater than this will reduce pollutant removal and may result in channelization (Horner, 1988). The filter strip needs to drain within 24 hours to encourage vegetative vitality. Dry periods are necessary to re-establish an aerobic soil profile (MD-DOE, 1984). Similar to the grassed swale, these management practices are often sited in residential areas. Consequently, there may be a concern from homeowners that these strips will create nuisance situations like mosquito and snake infestations.

Winter operation is variable and snowmelt or rain on frozen ground cannot be treated effectively by a filter strip. Filter strips may not be very effective for construction site erosion, because the sediment load may be more than a strip can assimilate. Also, prevention of channelization or short-circuiting may be difficult.

Design calculations

As with other management practices, a hydrologic analysis is required to determine the design peak flow. When the design flow has been established, the continuity and Manning's equation may be used to calculate the width of the strip (perpendicular to flow). Because of the need to maintain sheet flow across the strip, the design flow depth (y) should be no more than 0.5 inches or 0.04 ft. The design equations may be combined as written as:

$q = (1.486/n)(A)(R^{0.667})(S^{0.5})$, where:

q = Design runoff flow rate (cfs)

n = Manning's coefficient (dimensionless)

A = Cross-sectional area (ft^2)

R = Hydraulic radius (ft)

s = Longitudinal slope (ft/ft)

For a wide, shallow channel the hydraulic radius is approximately equal to the depth (y).

Manning's roughness coefficient (n) should be selected on the basis of vegetative species and density. A roughness coefficient of approximately 0.25 to 0.30 is often appropriate for sheet flow through a filter (USDA-SCS, 1984).

Calculate the length of the filter strip using one of the three criteria previously presented. Finally, include a level spreader at the upper end of the filter strip with provisions to prevent flows from bypassing the strip. The use of berms every 50 to 100 feet perpendicular to the top edge of the strip will help prevent channelized flow across the top and direct the flow more uniformly across the strip. Evenly grading the top edge is also important. The maximum permissible velocity for erosion prevention for a variety of grasses is typically greater than the design velocity for filters, so erosion should not be a problem.

Construction guidelines

The area to be used for the filter strip should be protected during construction by using diversions or upstream sediment traps. Also protect the natural infiltrative capacity of the soils from compaction during construction by using oversize tires and lightweight equipment. Clear the area of all large materials that would interfere with the ability to shape and grade the site. The topsoil at the site should be used to the maximum extent possible and the selection of vegetation should be appropriate to the soils, climate and desired texture.

Grasses and legumes produce a dense mat that resists erosion, maintains slow velocities through the filter and retains sediment from the runoff. Seed or sod the vegetation, using lime and fertilizer as indicated by soil tests to ensure establishment at the site.

Native grasses and legumes will provide a denser root mat and ultimately greater stability, but native plants take longer to establish. Native plants should not be fertilized during establishment. Plant species should be non-invasive relative to the local vegetation. Using plantings to encourage wildlife is not recommended unless it can be done without compromising the filter strip's treatment capability.

Maintenance

The primary maintenance requirement of a vegetative strip is mowing. Vegetation should not be cut shorter than the design flow depth, and heavy equipment that causes unnecessary compaction should not be used for mowing. Mow when the soil is firm to prevent rutting. Mowing offers an opportunity to clean off debris, harvest some of the nutrients captured in the grasses and to check for rills. Mow as little as 2 or 3 times a year, although tall grasses will tend to fall over and cause poor flow conditions. If the filter strip drains to a water body, minimize the use of fertilizers and pesticides.

The site should be inspected 2–3 times per year and after every major storm for rilling or short circuiting and erosion of the filter bed which would cause poor treatment or diversion of flows. If the strip has accumulated sediment in significant proportions, it should be removed and the bed regraded, seeded or otherwise revegetated. The level spreader should also be checked for accumulation of litter or sediment. Filter strips should not hold standing water. Any pockets that may form should be filled and regraded.

Locating filter strips in residential areas may pose difficulties in maintenance. The municipality should have responsibility for the mowing and inspection, but their desire to leave grasses long and natural may conflict with residential expectations.

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The Wisconsin Storm Water Manual: Filter Strips (G3691-6)

G3691-7

The Wisconsin Storm Water MANUAL

Vegetated water courses can be designed to achieve several goals including water conveyance, treatment, and/or infiltration. While the approach to design is similar for all these goals, choose design parameters in accordance with the objective of the specific project. For example, the maximum permissible velocity might be used for conveyance channels, but a much lower velocity may be used for channel design where the primary purpose is water treatment or infiltration.

Grassed Swales

Grassed swales are concave, vegetated conveyance systems that can improve water quality through infiltration and filtering. A swale can be a natural depression or a constructed, parabolic or trapezoidal channel. Swales are designed to treat the water quality volume and to provide stable conveyance of higher flows. Placing check dams at regular intervals along the channel length perpendicular to the direction of flow may enhance treatment capability.

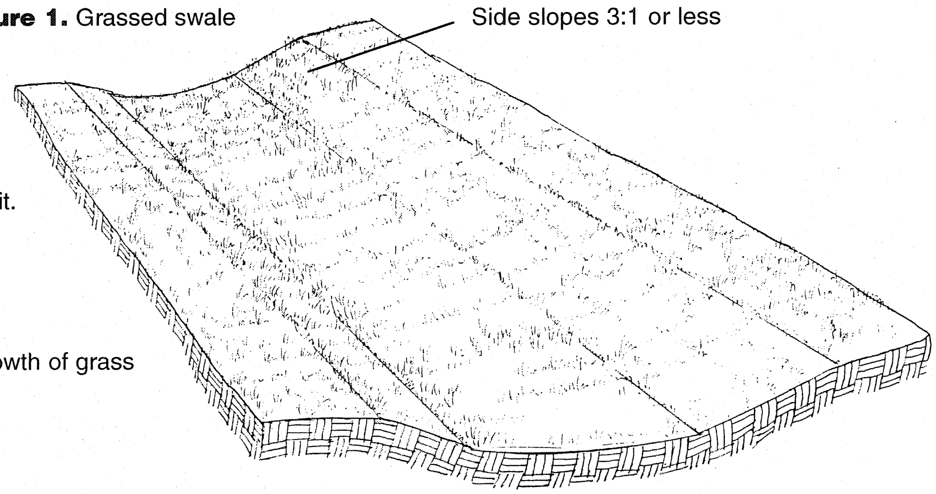
Grassed swales are appropriate alternatives to curb and gutter in residential settings, industrial parks, institutional areas and highway medians. They are used in combination with other management practices to provide pretreatment and flow attenuation. The amount of flow they receive may need to be limited for maximum treatment capability. In addition, if located on permeable soils, a portion of the flow will infiltrate into the ground, decreasing the runoff volume. Historically, swales have been designed as conveyance facilities. This chapter discusses the swale's ability to carry water while stressing the design parameters that can enhance pollutant removal.

Principles

Grass swales employ sedimentation and biofiltration as their primary pollutant removal mechanisms. Biofiltration includes filtration, infiltration, adsorption and biological uptake. When soils allow significant infiltration, pollutant removal mechanisms include adsorption of heavy metals and phosphorus onto soil particles and biological metabolism of organic pollutants.

Grassed swales encourage deposition of sand and soil aggregates if the velocity through the swale is less than 1.5 feet per second (ft/s). Even at this low velocity swales will not be effective in removing primary clay and silt particles. Velocities in excess of approximately 5 to 8 ft/s may reduce treatment effectiveness and may induce erosion. Table 1 illustrates typical removal efficiencies of a well designed, well maintained, conventional swale.

Figure 1. Grassed swale



Source: Modified from Galli, 1992

Table 1. Estimated removal efficiencies for grass swales

Pollutant	% Removal
Total suspended solids	70%
Total phosphorous	30%
Total nitrogen	25%
Trace metals	50–90%

(Schueler, 1992)

Field tests of swales, with and without check dams, have shown mixed results. In some cases, pollutant levels in the discharge water from the swale have increased. This may be due to over-fertilization and pesticide/herbicide use on lawns that drain to the swale. Leaching from culverts may increase metal levels as well. It is expected, however, that the use of check dams to increase detention and sedimentation will result in significant improvement in pollutant removal.

Planning guidelines

Grassed swales are appropriate in low and medium density residential areas in place of, or as supplements to, curb and gutter. Industrial and institutional areas (such as schools, hospitals, etc.) may also be suitable. Culverts may need to be constructed under driveways, modifying the flow pattern across the swale.

Swales can be used for 0- to 50-acre sites with the number and length of swales dictated by the topography and flows from the contributing area. The width of a swale depends on the flow rate and velocity through the swale. Minimum length and width requirements to achieve water quality improvements may limit the use of a swale at some sites.

Soil suitability is a key factor in a system designed for infiltration. While infiltration is desirable, highly permeable soils increase the potential for groundwater contamination. The volume of water that infiltrates into the soil may be greatly reduced because of compaction during construction. Compacted soil, coupled with the short residence time, reduces the potential for infiltration. Increasing the residence time and careful construction maximizes a swale's infiltration capability. If a swale is designed to infiltrate, the site must meet the same criteria for soil type, infiltration rate and separation to groundwater and bedrock as an infiltration basin or trench.

A swale may be considered a pretreatment unit and used in combination with other management practices to achieve the desired water quality treatment level for the 1.5-inch rain, and peak shaving for the 2-year, 24-hour storm. Swales designed for conveyance are often constructed to pass the peak flows from a 10-year, 24-hour duration storm or greater.

A maximum drain time of 24 hours is recommended to alleviate homeowner concerns about standing water and the possible nuisance conditions standing water may encourage. Homeowner education may be necessary to discourage use of the swale for leaf piling, burning or trash disposal. In some areas, homeowners are responsible for mowing in and around the swale. However, this practice should be discouraged because grass height is an important operational parameter, directly related to the design depth and velocity of the flow in the channel. Homeowners typically do not have mowers capable of cutting grasses above 6 inches. The local unit of government should carry the responsibility for mowing the swale and maintaining the buffer area.

Local officials should consider the advantages and disadvantages of swales over curb and gutter in a residential area. On the positive side, swales cost less, drain rain from the road surface more efficiently, improve water quality, attenuate flows and provide groundwater recharge. Conversely, they require more right-of-way and may not be compatible with sidewalk design. They need to be seeded and mowed and can be damaged by snowplows and parking. They cannot be used on poorly drained soils, steep slopes or where the swale has no outlet such as a stream, lake or storm sewer system.

A long swale serving a small watershed, where the swale density (feet of swale per acre of drainage basin) is large, will encourage infiltration. However, if the number of driveways and therefore the number of culverts is too great, the swale's effectiveness diminishes (MD-DOE,1984b).

Design considerations

Swales have been used as conveyance systems for many years. If swales are intended for use as practices not only for conveyance but also for water quality improvement, use the following design criteria.

Soils. The soil's infiltrating capability is a factor in locating swales. As with infiltration basins and trenches, swale infiltration rates measured in the field should be between 0.5 and 5.0 inches per hour (in./hr.). The suitable soils are sand, loamy sand, sandy loam, loam and silty loam, Types A, B and C soils (with some restrictions on A and C soils). Coarse, highly permeable soils provide little treatment capability and soils with very low permeabilities do not provide adequate infiltration during the short retention time in the channel. The erodibility of the soil is also a consideration in designing a grassed swale since the swale must be able to carry the estimated flows without eroding the swale or destroying the vegetation.

Shape. Swales should be parabolic or trapezoidal in shape. If adequate capacity is available to handle peak design flows, check dams may be used to increase in-channel detention. Parabolic swales are most like nature and less prone to meander under low flow conditions. Trapezoidal swales provide additional area for infiltration but may tend to meander at low flows and eventually revert to a parabolic form. Triangular channels provide little area for infiltration and are prone to erode since flows are concentrated. A swale should be designed for small to moderate storm events. If possible, the natural drainage patterns should be maintained and left undisturbed, provided the soils are stable.

Dimensions. The side slopes in the channel should be 3:1 (horizontal:vertical) or more to increase surface area, and to provide stability and access to equipment. Slopes 4:1 or flatter are safer for mowing equipment. A minimum swale length of at least 200 feet is necessary to achieve particulate pollutant removal. Velocities during the 1.5-inch rain event should not exceed 1.5 ft/s if deposition is to occur.

Vegetative cover. A dense vegetative cover slows the flow of water through the swale and increases treatment. Choose vegetation that can be maintained on the site and can tolerate being wet for 24 hours. Do not mow the grass beneath the water quality design depth. The swale velocity must not exceed erosive levels for the vegetative cover in the channel (see table 2 for examples of grasses and their related velocities).

Slopes. Swales are limited by the area's topography. Slopes of less than 5% are desirable. Erosion, channelization or diversion around the check dam can occur whenever flows exceed stabilization design considerations. Unless the swale is very carefully constructed and maintained, slopes of less than 1% may result in excessive ponding unless an underdrain is provided. Slopes in excess of 4% often result in high velocity, concentrated flows unless check dams are present. Infrequent overtopping of the swale may be tolerated if the flooding does not damage the swale or nearby property. An initial field survey will determine the natural grade of the area and other unique landscape features or traffic patterns that may affect the alignment and/or capacity of the swale.

Design calculations

The design of swales intended for water quality improvement must include the following elements:

- The water quality volume and design flow rates must be calculated using an appropriate hydrologic model.
- The accepted design criteria for water quality improvements are the 1.5-inch rain event and the 2-yr, 24-hour event for peak shaving. Using these criteria, the volume of water to be treated is equal to the 1.5-inch rain upland runoff volume, plus the rain that falls on the surface of the swale, minus the infiltration volume from the swale bottom surface. Flows above the treatment storm should pass through or around the swale.
- The desired shape is parabolic or trapezoidal. Design calculations will vary with the channel geometry.
- The capacity calculation for the peak discharge must use the continuity equation and Manning's Equation or design tables for channel dimensions using appropriate retarding factors.
- A maximum ponding time (Tp), of 24 hours is appropriate in residential areas.
- A maximum depth based on soil infiltration rate, f, must be calculated using dmax = (f)(Tp).
- The maximum velocity must not exceed that causing erosion of the vegetative liner. A treatment velocity of approximately 1.5 ft/s for the water quality event is desirable to encourage pollutant removal and infiltration.

Table 2. Permissible velocities for various ground covers.

			Permissible velocity (feet/second)	
No.	**Cover**	**Slope range** (%)	**Erosion resistant soils**	**Easily eroded soils**
1	Bermudagrass (Bynodon dactylon)	0–5	8	6
2	Kentucky 31 Tall fescue (Festuca arundinacea)	0–5	7	5
3	Grass-legume mixture	0–5	5	4
4	Red fescue Redtop (Agrostis Alba) Lespedeza servicea Alfalfa	0–5	3.5	2.5
5	Annuals* Common Lespedeza Sudangrass	0–5	3.5	2.5
6	Rock riprap section (for temporary construction)	5–10	8	6.5

**Annuals are used on mild slopes (less than 3 %) or as temporary protection until permanent covers are established. Use on slopes steeper than 5 % is not recommended.*

Source: Modified from USDA-SCS, 1988

- Side slopes should be at 3:1 or flatter for a vegetative liner and 2:1 for rip rap.
- Check dams may be installed if adequate capacity is maintained for design storms.

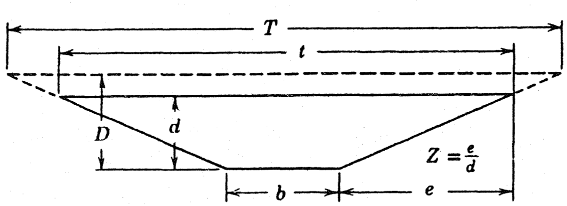
Swale design is described by the USDA Natural Resources Conservation Service (USDA-SCS, 1988). When designing the swale for conveyance and/or water quality, the drainage area must be delineated and broken into reaches or areas where the grade or vegetation changes. Runoff rates and volumes are calculated using an appropriate hydrologic model. The slope of the swale can be determined from topographic maps, profiles or cross sections. The swale is first designed to be able to pass the peak flows safely. The design capacity is calculated using Manning's formula or the look-up tables (tables 4a-4h) based on the vegetative retardance factors as described below (Table 3). The swale cross section and the liner preference are selected based on site conditions. For water quality, a parabolic shape and vegetative liner are preferred.

The design tables that follow are from the *Engineering Field Manual* (USDA-SCS, 1988). Table 3 classifies the different vegetative liners according to their retardance value. Using these relationships and table 2 for permissive velocity, the dimensions of the swale can be determined from tables 4a-4h. These tables assume a retardance no greater than a short grass (designated as D) when developing the safe velocity. The design capacity is based on the design height and type of vegetation, with a retardance of C or B for taller and/or thicker grasses. Tables 4a-4h provide the top width (in feet), depth (in feet) and velocity (in feet/second) at capacity when the grass is tall, for a given peak flow, safe velocity and grade. The peak flow in these tables assumes a 10-year, 24-hour storm event. The dimensions given in these tables do not account for the depth of the vegetative liner, sedimentation and freeboard. This additional space requirement must be taken into consideration when designing the swale.

Both the inlet and outlet of a swale must be designed with the intent of dissipating the flow energy, particularly if the flow comes from a concrete structure and enters a vegetative lining. Every swale must discharge to a stable outlet such as a waterway, open channel, subsurface system or similar management practice. The outlet must be constructed in such a way as to prevent scour in the receiving unit.

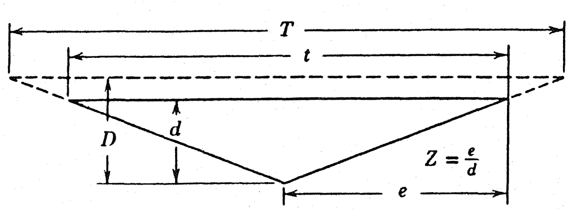
Figure 2. Channel cross section, wetted perimeter, hydraulic radius and top formulas.

Note: Freeboard = $D-d$ for all sections



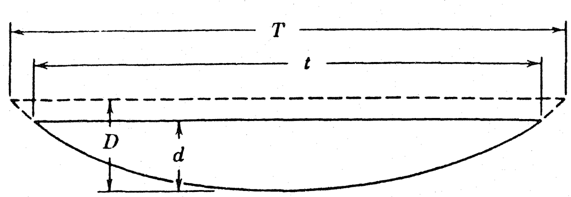
Cross-Sectional Area a	Wetted Perimeter, p	Hydraulic Radius $R=\frac{a}{p}$	Top Width
$bd + Zd^2$	$b + 2d\sqrt{Z^2+1}$	$\frac{bd + Zd^2}{b + 2d\sqrt{Z^2+1}}$	$t = b + 2dZ$ $T = b + 2DZ$

Trapezoidal cross section



Cross-Sectional Area a	Wetted Perimeter, p	Hydraulic Radius $R=\frac{a}{p}$	Top Width
Zd^2	$2d\sqrt{Z^2+1}$	$\frac{Zd}{2\sqrt{Z^2+1}}$ or $\frac{d}{2}$ approx.	$t = 2dZ$ $T = \frac{D}{d}t$

Triangular cross section



Cross-Sectional Area a	Wetted Perimeter, p	Hydraulic Radius $R=\frac{a}{p}$	Top Width
$\frac{2}{3}td$	$t + \frac{8d^2}{3t}$	$\frac{t^2d}{1.5t^2 + 4d^2}$ or $\frac{2d}{3}$ approx.	$t = \frac{a}{0.67d}$ $T = t\left(\frac{D}{d}\right)^{1/2}$

Parabolic cross section

Source: USDA-SCS, undated

Table 3. Classification of vegetation cover as to degree of retardance

Retardance	Cover	Condition
A	Weeping lovegrass	Excellent stand, tall (average 30 inches)
	Reed canarygrass or yellow bluestem ischaemum	Excellent stand, tall (average 36 inches)
B	Smooth bromegrass	Good stand, mowed, average 12 to 15 inches)
	Bermuda grass	Good stand, tall (average 12 inches)
	Native grass mixture (little bluestem, blue grama and other long and short Midwest grasses)	Good stand, unmowed
	Tall fescue	Good stand, unmowed (average 18 inches)
	Sericea lespedeza	Good stand, not woody, tall (average 19 inches)
	Grass-legume mixture — timothy, smooth bromegrass or orchardgrass	Good stand, uncut (average 20 inches)
	Reed canarygrass	Good stand, uncut (average 12 to 15 inches)
	Tall fescue, with birdsfoot trefoil or ladino clover	Good stand, uncut (average 18 inches)
	Blue grama	Good stand, uncut (average 13 inches)
C	Bahiagrass	Good stand, uncut (6 to 8 inches)
	Bermudagrass	Good stand, mowed (average 6 inches)
	Redtop	Good stand, headed (15 to 20 inches)
	Grass-legume mixture — summer (orchardgrass, redtop, Italian ryegrass and common lespedeza)	Good stand, uncut (6 to 8 inches)
	Centipedegrass	Very dense cover (average 6 inches)
	Kentucky bluegrass	Good stand, headed (6 to 12 inches)
D	Bermudagrass	Good stand, cut to 2.5-inch height
	Red fescue	Good stand, headed (12 to 18 inches)
	Buffalograss	Good stand, uncut (3 to 6 inches)
	Grass-legume mixture— fall, spring (orchardgrass, redtop, Italian ryegrass and common lespedeza)	Good stand, uncut (4 to 5 inches)
	Sericea lespedeza or Kentucky Bluegrass	Good stand, cut to 2-inch height Very good stand before cutting
E	Bermudagrass	Good stand, cut to 1.5-inch height
	Bermudagrass	Burned stubble

Source: USDA-SCS, 1988

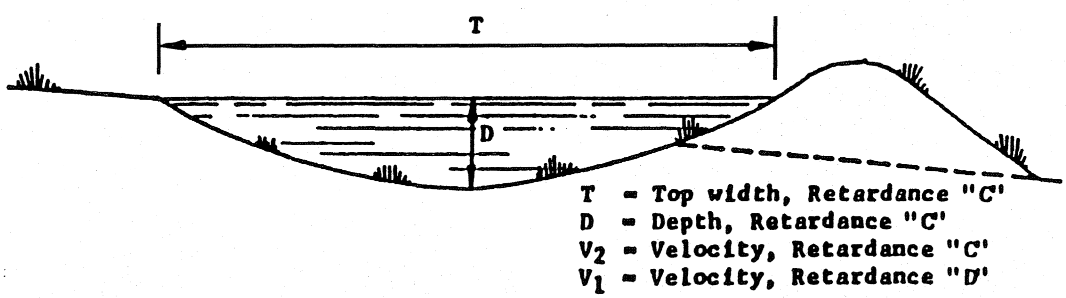
Table 4a. Parabolic waterway design.

V1 FOR RETARDANCE "D", TOP WIDTH (T), DEPTH' (D) AND V2 FOR RETARDANCE "C"

GRADE 2.00 PERCENT

Q	V1=2.0			V1=2.5			V1=3.0			V1=3.5			V1=4.0			V1=4.5			V1=5.0			V1=5.5			V1=6.0		
CFS	T	D	V2	T	D	V2	T	D	V2	T	D	V2	T	D	V2	T	D	V2	T	D	V2	T	D	V2	T	D	V2
5	5.9	0.9	1.5																								
10	12.4	0.8	1.5	8.1	0.9	2.0	5.9	1.0	2.5																		
15	18.5	0.8	1.5	12.3	0.9	2.0	9.3	1.0	2.5	6.8	1.1	3.0	4.7	1.4	3.5												
20	24.7	0.8	1.5	16.7	0.9	2.0	12.5	1.0	2.5	9.4	1.1	3.0	7.0	1.2	3.6	4.7	1.5	4.1									
25	30.8	0.8	1.5	20.8	0.9	2.0	15.9	1.0	2.4	11.8	1.1	3.0	9.0	1.2	3.5	6.8	1.3	4.1									
30	37.0	0.8	1.5	25.0	0.9	2.0	19.0	1.0	2.5	14.3	1.1	3.0	11.0	1.2	3.5	8.5	1.3	4.1	6.4	1.5	4.7						
35	43.2	0.8	1.5	29.1	0.9	2.0	22.2	1.0	2.5	16.9	1.0	3.0	12.9	1.1	3.5	10.1	1.3	4.1	7.8	1.4	4.7						
40	49.3	0.8	1.5	33.3	0.9	2.0	25.3	1.0	2.5	19.3	1.0	3.0	14.8	1.1	3.5	11.6	1.3	4.1	9.1	1.4	4.7	7.1	1.6	5.2			
45	55.5	0.8	1.5	37.4	0.9	2.0	28.5	1.0	2.5	21.7	1.0	3.0	16.7	1.1	3.5	13.1	1.3	4.1	10.4	1.4	4.7	8.2	1.6	5.2			
50	61.7	0.8	1.5	41.6	0.9	2.0	31.7	1.0	2.5	24.1	1.0	3.0	18.8	1.1	3.5	14.7	1.2	4.1	11.7	1.4	4.7	9.3	1.5	5.3	7.1	1.8	5.8
55	67.8	0.8	1.5	45.7	0.9	2.0	34.8	1.0	2.5	26.5	1.0	3.0	20.7	1.1	3.5	16.2	1.2	4.1	12.9	1.4	4.7	10.4	1.5	5.3	8.2	1.7	5.8
60	74.0	0.8	1.5	49.9	0.9	2.0	38.0	1.0	2.5	28.9	1.0	3.0	22.6	1.1	3.5	17.7	1.2	4.1	14.1	1.4	4.7	11.4	1.5	5.3	9.2	1.7	5.8
65	80.2	0.8	1.5	54.0	0.9	2.0	41.1	1.0	2.5	31.4	1.0	3.0	24.5	1.1	3.5	19.5	1.2	4.1	15.4	1.3	4.7	12.4	1.5	5.3	10.1	1.7	5.8
70	86.3	0.8	1.5	58.2	0.9	2.0	44.3	1.0	2.5	33.8	1.0	3.0	26.3	1.1	3.5	21.0	1.2	4.1	16.6	1.3	4.7	13.5	1.5	5.3	11.0	1.6	5.8
75	92.5	0.8	1.5	62.3	0.9	2.0	47.5	1.0	2.5	36.2	1.0	3.0	28.2	1.1	3.5	22.4	1.2	4.1	17.8	1.3	4.7	14.5	1.5	5.3	11.8	1.6	5.8
80	98.7	0.8	1.5	66.5	0.9	2.0	50.6	1.0	2.5	38.6	1.0	3.0	30.1	1.1	3.5	23.9	1.2	4.1	19.0	1.3	4.7	15.5	1.5	5.3	12.7	1.6	5.8
85	104.8	0.8	1.5	70.6	0.9	2.0	53.8	1.0	2.5	41.0	1.0	3.0	32.0	1.1	3.5	25.4	1.2	4.1	20.3	1.3	4.7	16.5	1.5	5.3	13.6	1.6	5.8
90	111.0	0.8	1.5	74.8	0.9	2.0	57.0	1.0	2.5	43.4	1.0	3.0	33.8	1.1	3.5	26.9	1.2	4.1	21.8	1.3	4.6	17.5	1.5	5.3	14.4	1.6	5.8
95	117.2	0.8	1.5	78.9	0.9	2.0	60.1	1.0	2.5	45.8	1.0	3.0	35.7	1.1	3.5	28.4	1.2	4.1	23.0	1.3	4.6	18.6	1.5	5.3	15.3	1.6	5.8
100	123.3	0.8	1.5	83.1	0.9	2.0	63.3	1.0	2.5	48.2	1.0	3.0	37.6	1.1	3.5	29.9	1.2	4.1	24.2	1.3	4.6	19.6	1.5	5.3	16.2	1.6	5.8
105	129.5	0.8	1.5	87.3	0.9	2.0	66.4	1.0	2.5	50.6	1.0	3.0	39.5	1.1	3.5	31.4	1.2	4.1	25.4	1.3	4.6	20.6	1.5	5.3	17.0	1.6	5.8
110	135.7	0.8	1.5	91.4	0.9	2.0	69.6	1.0	2.5	53.0	1.0	3.0	41.3	1.1	3.5	32.9	1.2	4.1	26.6	1.3	4.7	21.6	1.4	5.3	17.9	1.6	5.8
115	141.8	0.8	1.5	95.6	0.9	2.0	72.8	1.0	2.5	55.4	1.0	3.0	43.2	1.1	3.5	34.4	1.2	4.1	27.9	1.3	4.7	22.6	1.4	5.3	16.7	1.6	5.8
120	148.0	0.8	1.5	99.7	0.9	2.0	75.9	1.0	2.5	57.9	1.0	3.0	45.1	1.1	3.5	35.9	1.2	4.1	29.1	1.3	4.7	23.9	1.4	5.2	19.5	1.6	5.8
125	154.1	0.8	1.5	103.9	0.9	2.0	79.1	1.0	2.5	60.3	1.0	3.0	47.0	1.1	3.5	37.4	1.2	4.1	30.3	1.3	4.7	24.8	1.4	5.2	20.4	1.6	5.8
130	160.3	0.8	1.5	108.0	0.9	2.0	82.3	1.0	2.5	62.7	1.0	3.0	48.8	1.1	3.5	38.9	1.2	4.1	31.5	1.3	4.7	25.8	1.4	5.3	21.2	1.6	5.8
135	166.5	0.8	1.5	112.2	0.9	2.0	85.4	1.0	2.5	65.1	1.0	3.0	50.7	1.1	3.5	40.3	1.2	4.1	32.7	1.3	4.7	26.8	1.4	5.3	22.1	1.6	5.9
140	172.6	0.8	1.5	116.3	0.9	2.0	88.6	1.0	2.5	67.5	1.0	3.0	52.6	1.1	3.5	41.8	1.2	4.1	33.9	1.3	4.7	27.8	1.4	5.3	22.9	1.6	5.8
145	178.8	0.8	1.5	120.5	0.9	2.0	91.8	1.0	2.5	69.9	1.0	3.0	54.5	1.1	3.5	43.3	1.2	4.1	35.1	1.3	4.7	28.8	1.4	5.3	23.7	1.6	5.8
150	185.0	0.8	1.5	124.6	0.9	2.0	94.9	1.0	2.5	72.3	1.0	3.0	56.4	1.1	3.5	44.8	1.2	4.1	36.3	1.3	4.7	29.8	1.4	5.3	24.6	1.6	5.8

PARABOLIC WATERWAY DESIGN
(RETARDANCE "D" AND "C")



Note – Depth "D" does not include allowance for freeboard and settlement.

Table 4b.

V1 FOR RETARDANCE "D". TOP WIDTH (T), DEPTH' (D) AND V2 FOR RETARDANCE "C"

GRADE 3.00 PERCENT

Q CFS	V1=2.0			V1=2.5			V1=3.0			V1=3.5			V1=4.0			V1=4.5			V1=5.0			V1=5.5			V1=6.0		
	T	D	V2	T	D	V2	T	D	V2	T	D	V2	T	D	V2	T	D	V2	T	D	V2	T	D	V2	T	D	V2
5	7.4	0.7	1.4	4.9	0.8	1.9	3.2	1.0	2.3																		
10	15.1	0.7	1.4	10.2	0.8	1.9	7.6	0.8	2.4	5.7	0.9	2.9	4.0	1.1	3.4												
15	22.6	0.7	1.4	15.6	0.8	1.9	11.5	0.8	2.4	8.8	0.9	2.9	6.7	1.0	3.4	5.1	1.1	4.0									
20	30.1	0.7	1.4	20.7	0.8	1.9	15.5	0.8	2.4	11.8	0.9	2.9	9.2	0.9	3.4	7.2	1.0	4.0	5.5	1.2	4.6						
25	37.6	0.7	1.4	25.9	0.8	1.9	19.4	0.8	2.4	15.0	0.9	2.9	11.6	0.9	3.4	9.2	1.0	4.0	7.2	1.1	4.6	5.6	1.3	5.1			
30	45.1	0.7	1.4	31.1	0.8	1.9	23.3	0.8	2.4	18.0	0.9	2.9	14.0	0.9	3.4	11.1	1.0	4.0	8.9	1.1	4.6	7.1	1.2	5.2	5.3	1.5	5.7
35	52.7	0.7	1.4	36.2	0.8	1.9	27.1	0.8	2.4	21.0	0.9	2.9	16.5	0.9	3.4	13.0	1.0	4.0	10.5	1.1	4.6	8.4	1.2	5.2	6.7	1.4	5.7
40	60.2	0.7	1.4	41.4	0.8	1.9	31.0	0.8	2.4	24.0	0.9	2.9	18.9	0.9	3.4	14.9	1.0	4.0	12.0	1.1	4.6	9.8	1.2	5.2	7.9	1.3	5.7
45	67.7	0.7	1.4	46.6	0.8	1.9	34.9	0.8	2.4	27.0	0.9	2.9	21.2	0.9	3.4	17.0	1.0	4.0	13.6	1.1	4.6	11.1	1.2	5.2	9.1	1.3	5.7
50	75.2	0.7	1.4	51.8	0.8	1.9	38.8	0.8	2.4	29.9	0.9	2.9	23.6	0.9	3.4	18.9'	1.0	4.0	15.2	1.1	4.6	12.4	1.2	5.2	10.2	1.3	5.7
55	82.8	0.7	1.4	56.9	0.8	1.9	42.6	0.8	2.4	32.9	0.9	2.9	25.9	0.9	3.4	20.8	1.0	4.0	16.7	1.1	4.6	13.7	1.2	5.2	11.3	1.3	5.7
60	90.3	0.7	1.4	62.1	0.8	1.9	46.5	0.8	2.4	35.9	0.9	2.9	28.3	0.9	3.4	22.7	1.0	4.0	18.5	1.1	4.5	14.9	1.2	5.2	12.4	1.3	5.7
65	97.8	0.7	1.4	67.3	0.8	1.9	50.4	0.8	2.4	38.9	0.9	2.9	30.6	0.9	3.4	24.6	1.0	4.0	20.0	1.1	4.5	16.2	1.2	5.2	13.5	1.3	5.7
70	105.3	0.7	1.4	72.4	0.8	1.9	54.3	0.8	2.4	41.9	0.9	2.9	33.0	0.9	3.4	26.4	1.0	4.0	21.5	1.1	4.5	17.5	1.2	5.2	14.5	1.3	5.7
75	112.8	0.7	1.4	77.6	0.8	1.9	58.1	0.8	2.4	44.9	0.9	2.9	35.3	0.9	3.4	28.3	1.0	4.0	23.1	1.1	4.5	19.1	1.2	5.1	15.6	1.3	5.7
80	120.4	0.7	1.4	82.8	0.8	1.9	62.0	0.8	2.4	47.9	0.9	2.9	37.7	0.9	3.4	30.2	1.0	4.0	24.6	1.1	4.5	20.3	1.2	5.1	16.7	1.3	5.7
85	127.9	0.7	1.4	88.0	0.8	1.9	65.9	0.8	2.4	50.9	0.9	2.9	40.1	0.9	3.4	32.1	1.0	4.0	26.1	1.1	4.5	21.6	1.2	5.1	17.8	1.2	5.7
90	135.4	0.7	1.4	93.1	0.8	1.9	69.8	0.8	2.4	53.9	0.9	2.9	42.4	0.9	3.4	34.0	1.0	4.0	27.7	1.1	4.5	22.9	1.2	5.1	18.9	1.2	5.7
95	142.9	0.7	1.4	98.3	0.8	1.9	73.6	0.8	2.4	56.9	0.9	2.9	44.8	0.9	3.4	35.9	1.0	4.0	29.2	1.1	4.5	24.1	1.2	5.1	20.2	1.2	5.7
100	150.5	0.7	1.4	103.5	0.8	1.9	77.5	0.8	2.4	59.9	0.9	2.9	47.1	0.9	3.4	37.8	1.0	4.0	30.7	1.1	4.5	25.4	1.2	5.1	21.2	1.2	5.7
105	158.0	0.7	1.4	108.7	0.8	1.9	81.4	0.8	2.4	62.8	0.9	2.9	49.5	0.9	3.4	39.6	1.0	4.0	32.3	1.1	4.5	26.7	1.2	5.1	22.3	1.2	5.7
110	165.5	0.7	1.4	113.8	0.8	1.9	85.3	0.8	2.4	65.8	0.9	2.9	51.8	0.9	3.4	41.5	1.0	4.0	33.8	1.1	4.6	27.9	1.2	5.1	23.3	1.2	5.7
115	173.0	0.7	1.4	119.0	0.8	1.9	89.1	0.8	2.4	68.8	0.9	2.9	54.2	0.9	3.4	43.4	1.0	4.0	35.4	1.1	4.6	29.2	1.2	5.1	24.4	1.2	5.7
120	180.5	0.7	1.4	124.2	0.8	1.9	93.0	0.8	2.4	71.8	0.9	2.9	56.5	0.9	3.4	45.3	1.0	4.0	36.9	1.1	4.6	30.5	1.2	5.1	25.5	1.2	5.7
125	188.1	0.7	1.4	129.4	0.8	1.9	96.9	0.8	2.4	74.8	0.9	2.9	58.9	0.9	3.4	47.2	1.0	4.0	38.4	1.1	4.6	31.7	1.2	5.1	26.5	1.2	5.7
130	195.6	0.7	1.4	134.5	0.8	1.9	100.8	0.8	2.4	77.8	0.9	2.9	61.2	0.9	3.4	49.1	1.0	4.0	40.0	1.1	4.6	33.0	1.2	5.1	27.6	1.2	5.7
135	203.1	0.7	1.4	139.7	0.8	1.9	104.6	0.8	2.4	80.8	0.9	2.9	63.6	0.9	3.4	51.0	1.0	4.0	41.5	1.1	4.6	34.3	1.2	5.1	28.6	1.2	5.7
140	210.6	0.7	1.4	144.9	0.8	1.9	108.5	0.8	2.4	83.8	0.9	2.9	66.0	0.9	3.4	52.8	1.0	4.0	43.0	1.1	4.6	35.6	1.2	5.1	29.7	1.2	5.7
145	218.2	0.7	1.4	150.1	0.8	1.9	112.4	0.8	2.4	86.8	0.9	2.9	68.3	0.9	3.4	54.7	1.0	4.0	44.6	1.1	4.6	36.8	1.2	5.1	30.7	1.2	5.7
150	225.7	0.7	1.4	155.2	0.8	1.9	116.3	0.8	2.4	89.8	0.9	2.9	70.7	0.9	3.4	56.6	1.0	4.0	46.1	1.1	4.6	38.1	1.2	5.1	31.8	1.2	5.7

PARABOLIC WATERWAY DESIGN
(RETARDANCE "D" AND "C")

Table 4c.

V1 FOR RETARDANCE "D". TOP WIDTH (T), DEPTH' (D) AND V2 FOR RETARDANCE "C"

GRADE 4.00 PERCENT

Q CFS	V1=2.0			V1=2.5			V1=3.0			V1=3.5			V1=4.0			V1=4.5			V1=5.0			V1=5.5			V1=6.0		
	T	D	V2	T	D	V2	T	D	V2	T	D	V2	T	D	V2	T	D	V2	T	D	V2	T	D	V2	T	D	V2
5	8.5	0.6	1.4	5.9	0.7	1.8	4.1	0.8	2.3																		
10	17.2	0.6	1.4	12.1	0.7	1.8	8.8	0.7	2.3	6.7	0.8	2.8	5.2	0.9	3.3	3.8	1.0	3.9									
15	25.8	0.6	1.4	18.1	0.7	1.8	13.4	0.7	2.3	10.3	0.8	2.8	8.1	0.8	3.4	6.4	0.9	3.9	4.9	1.0	4.5						
20	34.4	0.6	1.4	24.2	0.7	1.8	17.8	0.7	2.3	13.9	0.8	2.8	10.9	0.8	3.4	9.7	C.9	3 9	6.9	1.0	4.5	5.5	1.1	5.0			
25	43.0	0.6	1.4	30.2	0.7	1.9	22.3	0.7	2.3	17.4	0.8	2.8	13.8	0.8	3.3	10.9	0.9	3.9	8.8	1.0	4.5	7.1	1.0	5.1	5.7	1.2	5.6
30	51.6	0.6	1.4	36.3	0.7	1.9	26.7	0.7	2.3	20.8	0.8	2.8	16.5	0.8	3.3	13.2	0.9	3.9	10.7	0.9	4.5	8.7	1.0	5.1	7.1	1.1	5.6
35	60.2	0.6	1.4	42.3	0.7	1.9	31.1	0.7	2.3	24.3	0.8	2.8	19.3	0.8	3.4	15.6	0.9	3.9	12.5	0.9	4.5	10.3	1.0	5.0	8.4	1.1	5.6
40	68.8	0.6	1.4	48.3	0.7	1.9	35.6	0.7	2.3	27.8	0.8	2.8	22.0	0.8	3.4	17.8	0.9	3.9	14.4	0.9	4.5	11.8	1.0	5.0	9.8	1.1	5.7
45	77.4	0.6	1.4	54.4	0.7	1.9	40.0	0.7	2.4	31.2	0.8	2.8	24.8	0.8	3.4	20.0	0.9	3.9	16.4	0.9	4.4	13.3	1.0	5.0	11.1	1.1	5.7
50	86.0	0.6	1.4	60.4	0.7	1.9	44.5	0.7	2.4	34.7	0.8	2.8	27.5	0.8	3.4	22.2	0.9	3.9	18.2	0.9	4.4	14.9	1.0	5.0	12.3	1.1	5.7
55	94.6	0.6	1.4	66.5	0.7	1.9	48.9	0.7	2.4	38.2	0.8	2.8	30.3	0.8	3.4	24.4	0.9	3.9	20.0	0.9	4.4	16.6	1.0	5.0	13.6	1.1	5.7
60	103.2	0.6	1.4	72.5	0.7	1.9	53.4	0.7	2.4	41.7	0.8	2.8	33.0	0.8	3.4	26.6	0.9	3.9	21.8	0.9	4.5	18.1	1.0	5.0	14.9	1.1	5.7
65	111.8	0.6	1.4	78.5	0.7	1.9	57.8	0.7	2.4	45.1	0.8	2.8	35.8	0.8	3.4	28.9	0.9	3.9	23.6	0.9	4.5	19.6	1.0	5.0	16.2	1.1	5.7
70	120.4	0.6	1.4	84.6	0.7	1.9	62.3	0.7	2.4	48.6	0.8	2.8	38.6	0.8	3.4	31.1	0.9	3.9	25.4	0.9	4.5	21.1	1.0	5.0	17.7	1.1	5.6
75	129.0	0.6	1.4	90.6	0.7	1.9	66.7	0.7	2.4	52.1	0.8	2.8	41.3	0.8	3.4	33.3	0.9	3.9	27.2	0.9	4.5	22.6	1.0	5.0	19.0	1.1	5.6
80	137.6	0.6	1.4	96.7	0.7	1.9	71.2	0.7	2.4	55.5	0.8	2.8	44.1	0.8	3.4	35.5	0.9	3.9	29.1	0.9	4.5	24.1	1.0	5.0	20.2	1.1	5.6
85	146.2	0.6	1.4	102.7	0.7	1.9	75.6	0.7	2.4	59.0	0.8	2.8	46.8	0.8	3.4	37.7	0.9	3.9	30.9	0.9	4.5	25.6	1.0	5.0	21.5	1.1	5.6
90	154.8	0.6	1.4	108.7	0.7	1.9	80.0	0.7	2.4	62.5	0.8	2.8	49.6	0.8	3.4	39.9	0.9	3.9	32.7	0.9	4.5	27.1	1.0	5.0	22.8	1.1	5.6
95	163.4	0.6	1.4	114.8	0.7	1.9	84.5	0.7	2.4	65.9	0.8	2.8	52.3	0.8	3.4	42.2	0.9	3.9	34.5	0.9	4.5	28.6	1.0	5.0	24.0	1.1	5.6
100	172.0	0.6	1.4	120.8	0.7	1.9	88.9	0.7	2.4	69.4	0.8	2.8	55.1	0.8	3.4	44.4	0.9	3.9	36.3	0.9	4.5	30.1	1.0	5.0	25.3	1.1	5.6
105	180.6	0.6	1.4	126.9	0.7	1.9	93.4	0.7	2.4	72.9	0.8	2.8	57.8	0.8	3.4	46.6	0.9	3.9	38.1	0.9	4.5	31.6	1.0	5.0	26.5	1.1	5.6
110	189.2	0.6	1.4	132.9	0.7	1.9	97.8	0.7	2.4	76.3	0.8	2.8	60.6	0.8	3.4	48.8	0.9	3.9	39.9	0.9	4.5	33.1	1.0	5.0	27.8	1.1	5.6
115	197.8	0.6	1.4	138.9	0.7	1.9	102.3	0.7	2.4	79.8	0.8	2.8	63.3	0.8	3.4	51.0	0.9	3.9	41.7	0.9	4.5	34.6	1.0	5.0	29.0	1.1	5.6
120	206.4	0.6	1.4	145.0	0.7	1.9	106.7	0.7	2.4	83.3	0.8	2.8	66.1	0.8	3.4	53.3	0.9	3.9	43.6	0.9	4.5	36.1	1.0	5.0	30.2	1.1	5.7
125	215.0	0.6	1.4	151.0	0.7	1.9	111.2	0.7	2.4	86.8	0.8	2.8	68.8	0.8	3.4	55.5	0.9	3.9	45.4	0.9	4.5	37.6	1.0	5.0	31.5	1.1	5.7
130	223.7	0.6	1.4	157.1	0.7	1.9	115.6	0.7	2.4	90.2	0.8	2.8	71.6	0.8	3.4	57.7	0.9	3.9	47.2	0.9	4.5	39.1	1.0	5.0	32.7	1.1	5.7
135	232.3	0.6	1.4	163.1	0.7	1.9	120.1	0.7	2.4	93.7	0.8	2.8	74.3	0.8	3.4	59.9	0.9	3.9	49.0	0.9	4.5	40.6	1.0	5.0	34.0	1.1	5.7
140	240.9	0.6	1.4	169.1	0.7	1.9	124.5	0.7	2.4	97.2	0.8	2.8	77.1	0.8	3.4	62.1	0.9	3.9	50.8	0.9	4.5	42.1	1.0	5.0	35.2	1.1	5.7
145	249.5	0.6	1.4	175.2	0.7	1.9	129.0	0.7	2.4	100.6	0.8	2.8	79.8	0.8	3.4	64.3	0.9	3.9	52.6	0.9	4.5	43.6	1.0	5.0	36.5	1.1	5.7
150	258.1	0.6	1.4	181.2	0.7	1.9	133.4	0.7	2.4	104.1	0.8	2.8	82.6	0.8	3.4	66.6	0.9	3.9	54.4	0.9	4.5	45.1	1.0	5.0	37.8	1.1	5.7

PARABOLIC WATERWAY DESIGN
(RETARDANCE "D" AND "C")

Table 4d.

V1 FOR RETARDANCE "D". TOP WIDTH (T), DEPTH' (D) AND V2 FOR RETARDANCE "C"

GRADE 5.00 PERCENT

Q CFS	V1=2.0			V1=2.5			V1=3.0			V1=3.5			V1=4.0			V1=4.5			V1=5.0			V1=5.5			V1=6.0		
	T	D	V2	T	D	V2	T	D	V2	T	D	V2	T	D	V2	T	D	V2	T	D	V2	T	D	V2	T	D	V2
5	9.5	0.6	1.4	6.7	0.6	1.8	4.7	0.7	2.3	3.5	0.8	2.8															
10	19.0	0.6	1.4	13.7	0.6	1.8	9.7	0.7	2.3	7.6	0.7	2.8	6.0	0.8	3.3	4.7	0.8	3.8	3.4	1.0	4.4						
15	28.5	0.6	1.4	20.5	0.6	1.8	14.8	0.7	2.3	11.7	0.7	2.8	9.2	0.7	3.3	7.3	0.8	3.8	5.9	0.9	4.4	4.7	1.0	5.0			
20	38.0	0.6	1.4	27.3	0.6	1.8	19.7	0.7	2.3	15.5	0.7	2.8	12.4	0.7	3.3	9.9	0.8	3.8	8.0	0.9	4.4	6.5	0.9	4.9	5.3	1.0	5.5
25	47.5	0.6	1.4	34.1	0.6	1.8	24.6	0.7	2.3	19.4	0.7	2.8	15.5	0.7	3.3	12.6	0.8	3.8	10.1	0.8	4.4	8.3	0.9	5.0	6.8	1.0	5.6
30	57.0	0.6	1.4	40.9	0.6	1.8	29.5	0.7	2.3	23.3	0.7	2.8	18.6	0.7	3.3	15.1	0.8	3.8	12.2	0.8	4.4	10.1	0.9	5.0	8.3	1.0	5.6
35	66.5	0.6	1.4	47.7	0.6	1.8	34.4	0.7	2.3	27.2	0.7	2.8	21.7	0.7	3.3	17.6	0.8	3.8	14.5	0.8	4.4	11.8	0.9	5.0	9.8	1.0	5.6
40	76.0	0.6	1.4	54.6	0.6	1.8	39.4	0.7	2.3	31.0	0.7	2.8	24.8	0.7	3.3	20.1	0.8	3.8	16.5	0.8	4.4	13.6	0.9	5.0	11.3	1.0	5.5
45	85.5	0.6	1.4	61.4	0.6	1.8	44.3	0.7	2.3	34.9	0.7	2.8	27.9	0.7	3.3	22.6	0.8	3.8	18.6	0.8	4.4	15.5	0.9	4.9	12.8	1.0	5.5
50	95.0	0.6	1.4	68.2	0.6	1.8	49.2	0.7	2.3	38.8	0.7	2.8	31.0	0.7	3.3	25.1	0.8	3.8	20.6	0.8	4.4	17.2	0.9	4.9	14.3	1.0	5.5
55	104.6	0.6	1.4	75.0	0.6	1.8	54.1	0.7	2.3	42.7	0.7	2.8	34.1	0.7	3.3	27.6	0.8	3.8	22.7	0.8	4.4	18.9	0.9	4.9	15.9	0.9	5.5
60	114.1	0.6	1.4	81.8	0.6	1.8	59.0	0.7	2.3	46.6	0.7	2.8	37.2	0.7	3.3	30.1	0.8	3.8	24.7	0.8	4.4	20.6	0.9	4.9	17.3	0.9	5.5
65	123.6	0.6	1.4	88.6	0.6	1.8	63.9	0.7	2.3	50.4	0.7	2.8	40.3	0.7	3.3	32.6	0.8	3.8	26.8	0.8	4.4	22.3	0.9	4.9	18.8	0.9	5.5
70	133.1	0.6	1.4	95.5	0.6	1.8	68.9	0.7	2.3	54.3	0.7	2.8	43.4	0.7	3.3	35.1	0.8	3.8	28.9	0.8	4.4	24.0	0.9	4.9	20.2	0.9	5.5
75	142.6	0.6	1.4	102.3	0.6	1.8	73.8	0.7	2.3	58.2	0.7	2.8	46.5	0.7	3.3	37.7	0.8	3.8	30.9	0.8	4.4	25.7	0.9	4.9	21.6	0.9	5.5
80	152.1	0.6	1.4	109.1	0.6	1.8	78.7	0.7	2.3	62.1	0.7	2.8	49.6	0.7	3.3	40.2	0.8	3.8	33.0	0.8	4.4	27.4	0.9	4.9	23.1	0.9	5.5
85	161.6	0.6	1.4	115.9	0.6	1.8	83.6	0.7	2.3	65.9	0.7	2.8	52.7	0.7	3.3	42.7	0.8	3.8	35.0	0.8	4.4	29.1	0.9	5.0	24.5	0.9	5.5
90	171.1	0.6	1.4	122.7	0.6	1.8	88.5	0.7	2.3	69.8	0.7	2.8	55.8	0.7	3.3	45.2	0.8	3.8	37.1	0.8	4.4	30.9	0.9	5.0	26.0	0.9	5.5
95	180.6	0.6	1.4	129.6	0.6	1.8	93.4	0.7	2.3	73.7	0.7	2.8	58.9	0.7	3.3	47.7	0.8	3.8	39.2	0.8	4.4	32.6	0.9	5.0	27.4	0.9	5.5
100	190.1	0.6	1.4	136.4	0.6	1.8	98.4	0.7	2.3	77.6	0.7	2.8	62.0	0.7	3.3	50.2	0.8	3.8	41.2	0.8	4.4	34.3	0.9	5.0	28.8	0.9	5.5
105	199.6	0.6	1.4	143.2	0.6	1.8	103.3	0.7	2.3	81.5	0.7	2.8	65.1	0.7	3.3	52.7	0.8	3.8	43.3	0.8	4.4	36.0	0.9	5.0	30.3	0.9	5.5
110	209.1	0.6	1.4	150.0	0.6	1.8	108.2	0.7	2.3	85.3	0.7	2.8	68.2	0.7	3.3	55.2	0.8	3.8	45.3	0.8	4.4	37.7	0.9	5.0	31.7	0.9	5.5
115	218.6	0.6	1.4	156.8	0.6	1.8	113.1	0.7	2.3	89.2	0.7	2.8	71.3	0.7	3.3	57.7	0.8	3.8	47.4	0.8	4.4	39.4	0.9	5.0	33.2	0.9	5.5
120	228.1	0.6	1.4	163.6	0.6	1.8	118.0	0.7	2.3	93.1	0.7	2.8	74.3	0.7	3.3	60.2	0.8	3.8	49.5	0.8	4.4	41.1	0.9	5.0	34.6	0.9	5.5
125	237.6	0.6	1.4	170.5	0.6	1.8	123.0	0.7	2.3	97.0	0.7	2.8	77.4	0.7	3.3	62.7	0.8	3.8	51.5	0.8	4.4	42.8	0.9	5.0	36.0	0.9	5.5
130	247.1	0.6	1.4	177.3	0.6	1.8	127.9	0.7	2.3	100.8	0.7	2.8	80.5	0.7	3.3	65.2	0.8	3.8	53.6	0.8	4.4	44.6	0.9	5.0	37.5	0.9	5.5
135	256.6	0.6	1.4	184.1	0.6	1.8	132.8	0.7	2.3	104.7	0.7	2.8	83.6	0.7	3.3	67.8	0.8	3.8	55.6	0.8	4.4	46.3	0.9	5.0	38.9	0.9	5.5
140	266.1	0.6	1.4	190.9	0.6	1.8	137.7	0.7	2.3	108.6	0.7	2.8	86.7	0.7	3.3	70.3	0.8	3.8	57.7	0.8	4.4	48.0	0.9	5.0	40.4	0.9	5.5
145	275.6	0.6	1.4	197.7	0.6	1.8	142.6	0.7	2.3	112.5	0.7	2.8	89.8	0.7	3.3	72.8	0.8	3.8	59.8	0.8	4.4	49.7	0.9	5.0	41.8	0.9	5.5
150	285.1	0.6	1.4	204.6	0.6	1.8	147.5	0.7	2.3	116.4	0.7	2.8	92.9	0.7	3.3	75.3	0.8	3.8	61.8	0.8	4.4	51.4	0.9	5.0	43.2	0.9	5.5

PARABOLIC WATERWAY DESIGN
(RETARDANCE "D" AND "C")

Table 4e.

V1 FOR RETARDANCE "D". TOP WIDTH (T), DEPTH' (D) AND V2 FOR RETARDANCE "B"

GRADE 2.00 PERCENT

Q CFS	V1=2.0			V1=2.5			V1=3.0			V1=3.5			V1=4.0			V1=4.5			V1=5.0			V1=5.5			V1=6.0		
	T	D	V2	T	D	V2	T	D	V2	T	D	V2	T	D	V2	T	D	V2	T	D	V2	T	D	V2	T	D	V2
5	7.1	1.2	0.9																								
10	14.7	1.2	0.9	9.5	1.3	1.2	7.0	1.4	1.5																		
15	22.0	1.2	0.9	14.5	1.3	1.2	10.8	1.4	1.5	8.0	1.5	1.9	5.5	1.9	2.1												
20	29.3	1.2	0.9	19.6	1.2	1.2	14.6	1.3	1.5	10.9	1.5	1.9	8.1	1.6	2.3	5.5	2.1	2.6									
25	36.6	1.2	0.9	24.4	1.2	1.2	18.5	1.3	1.5	13.8	1.4	1.9	10.4	1.6	2.3	7.9	1.8	2.7									
30	43.9	1.2	0.9	29.3	1.2	1.2	22.2	1.3	1.6	16.6	1.4	1.9	12.7	1.5	2.3	9.7	1.7	2.7	7.3	2.0	3.1						
35	51.2	1.2	0.9	34.2	1.2	1.2	25.8	1.3	1.6	19.6	1.4	1.9	14.9	1.5	2.3	11.5	1.7	2.7	8.9	1.9	3.2						
40	58.5	1.2	0.9	39.0	1.2	1.2	29.5	1.3	1.6	22.4	1.4	1.9	17.1	1.5	2.3	13.3	1.6	2.8	10.4	1.8	3.2	8.0	2.1	3.6			
45	65.8	1.2	0.9	43.9	1.2	1.2	33.2	1.3	1.6	25.2	1.4	1.9	19.3	1.5	2.3	15.0	1.6	2.8	11.8	1.8	3.2	9.2	2.0	3.7			
50	73.1	1.2	0.9	48.8	1.2	1.2	36.8	1.3	1.6	28.0	1.4	1.9	21.7	1.5	2.3	16.7	1.6	2.8	13.2	1.8	3.2	10.5	1.9	3.7	7.9	2.3	4.1
55	80.4	1.2	0.9	53.6	1.2	1.2	40.5	1.3	1.6	30.7	1.4	1.9	23.9	1.5	2.3	18.5	1.6	2.8	14.6	1.7	3.2	11.7	1.9	3.7	9.2	2.2	4.1
60	87.7	1.2	0.9	58.5	1.2	1.2	44.2	1.3	1.6	33.5	1.4	1.9	26.0	1.5	2.3	20.2	1.6	2.8	16.0	1.7	3.2	12.8	1.9	3.7	10.2	2.1	4.1
65	95.0	1.2	0.9	63.4	1.2	1.2	47.9	1.3	1.6	36.3	1.4	1.9	28.2	1.5	2.3	22.1	1.6	2.8	17.4	1.7	3.3	14.0	1.9	3.7	11.3	2.1	4.2
70	102.3	1.2	0.9	68.2	1.2	1.2	51.6	1.3	1.6	39.1	1.4	1.9	30.3	1.5	2.3	23.8	1.6	2.8	18.8	1.7	3.3	15.2	1.9	3.7	12.3	2.1	4.2
75	109.6	1.2	0.9	73.1	1.2	1.2	55.2	1.3	1.6	41.9	1.4	1.9	32.5	1.5	2.3	25.5	1.6	2.8	20.1	1.7	3.3	16.2	1.8	3.7	13.2	2.0	4.2
80	116.9	1.2	0.9	78.0	1.2	1.2	58.9	1.3	1.6	44.7	1.4	1.9	34.6	1.5	2.3	27.2	1.6	2.8	21.5	1.7	3.3	17.4	1.8	3.8	14.2	2.0	4.2
85	124.2	1.2	0.9	82.9	1.2	1.2	62.6	1.3	1.6	47.4	1.4	1.9	36.8	1.5	2.3	28.9	1.6	2.8	22.9	1.7	3.3	18.5	1.8	3.8	15.1	2.0	4.2
90	131.5	1.2	0.9	87.7	1.2	1.2	66.3	1.3	1.6	50.2	1.4	1.9	39.0	1.5	2.3	30.6	1.6	2.8	24.6	1.7	3.2	19.6	1.8	3.8	16.1	2.0	4.2
95	138.8	1.2	0.9	92.6	1.2	1.2	69.9	1.3	1.6	53.0	1.4	1.9	41.1	1.5	2.3	32.3	1.6	2.8	25.9	1.7	3.3	20.8	1.8	3.8	17.0	2.0	4.2
100	146.1	1.2	0.9	97.5	1.2	1.2	73.6	1.3	1.6	55.8	1.4	1.9	43.3	1.5	2.3	34.0	1.6	2.8	27.3	1.7	3.3	21.9	1.8	3.8	18.0	2.0	4.2
105	153.4	1.2	0.9	102.3	1.2	1.2	77.3	1.3	1.6	58.6	1.4	1.9	45.4	1.5	2.3	35.7	1.6	2.8	28.6	1.7	3.3	23.0	1.8	3.8	18.9	2.0	4.2
110	160.7	1.2	0.9	107.2	1.2	1.2	81.0	1.3	1.6	61.4	1.4	1.9	47.6	1.5	2.3	37.3	1.6	2.8	30.0	1.7	3.3	24.1	1.8	3.8	19.8	2.0	4.2
115	168.0	1.2	0.9	112.1	1.2	1.2	84.7	1.3	1.6	64.2	1.4	1.9	49.8	1.5	2.3	39.0	1.6	2.8	31.3	1.7	3.3	25.3	1.8	3.8	20.8	2.0	4.2
120	175.3	1.2	0.9	117.0	1.2	1.2	88.3	1.3	1.6	67.0	1.4	1.9	51.9	1.5	2.3	40.7	1.6	2.8	32.7	1.7	3.3	26.7	1.8	3.7	21.7	2.0	4.2
125	182.6	1.2	0.9	121.8	1.2	1.2	92.0	1.3	1.6	69.7	1.4	1.9	54.1	1.5	2.3	42.4	1.6	2.8	34.1	1.7	3.3	27.8	1.8	3.7	22.6	1.9	4.3
130	189.9	1.2	0.9	126.7	1.2	1.2	95.7	1.3	1.6	72.5	1.4	1.9	56.2	1.5	2.3	44.1	1.6	2.8	35.4	1.7	3.3	28.9	1.8	3.7	23.6	1.9	4.3
135	197.3	1.2	0.9	131.6	1.2	1.2	99.4	1.3	1.6	75.3	1.4	1.9	58.4	1.5	2.3	45.8	1.6	2.8	36.8	1.7	3.3	30.0	1.8	3.7	24.5	1.9	4.3
140	204.6	1.2	0.9	136.5	1.2	1.2	103.1	1.3	1.6	78.1	1.4	1.9	60.6	1.5	2.3	47.5	1.6	2.8	38.1	1.7	3.3	31.1	1.8	3.7	25.4	1.9	4.3
145	211.9	1.2	0.9	141.3	1.2	1.2	106.7	1.3	1.6	80.9	1.4	1.9	62.7	1.5	2.3	49.2	1.6	2.8	39.5	1.7	3.3	32.3	1.8	3.7	26.4	1.9	4.3
150	219.2	1.2	0.9	146.2	1.2	1.2	110.4	1.3	1.6	83.7	1.4	1.9	64.9	1.5	2.3	50.9	1.6	2.8	40.8	1.7	3.3	33.4	1.8	3.7	27.3	1.9	4.3

PARABOLIC WATERWAY DESIGN
(RETARDANCE "D" AND "B")

Table 4f.

V1 FOR RETARDANCE "D". TOP WIDTH (T), DEPTH' (D) AND V2 FOR RETARDANCE "B"

GRADE 3.00 PERCENT

Q CFS	V1=2.0			V1=2.5			V1=3.0			V1=3.5			V1=4.0			V1=4.5			V1=5.0			V1=5.5			V1=6.0		
	T	D	V2	T	D	V2	T	D	V2	T	D	V2	T	D	V2	T	D	V2	T	D	V2	T	D	V2	T	D	V2
5	8.8	1.0	0.8	5.8	1.1	1.1	3.9	1.5	1.3																		
10	18.0	1.0	0.8	12.1	1.1	1.2	8.9	1.1	1.5	6.6	1.3	1.8	4.7	1.5	2.1												
15	27.0	1.0	0.8	18.3	1.1	1.2	13.5	1.1	1.5	10.3	1.2	1.8	7.9	1.3	2.2	6.0	1.5	2.5									
20	35.9	1.0	0.8	24.4	1.1	1.2	18.2	1.1	1.5	13.8	1.2	1.8	10.7	1.3	2.2	8.3	1.4	2.6	6.4	1.6	3.0						
25	44.9	1.0	0.8	30.5	1.1	1.2	22.8	1.1	1.5	17.5	1.2	1.8	13.4	1.3	2.2	10.6	1.4	2.6	8.3	1.5	3.0	6.4	1.7	3.4			
30	53.9	1.0	0.8	36.6	1.1	1.2	27.3	1.1	1.5	20.9	1.2	1.8	16.2	1.3	2.2	12.8	1.4	2.6	10.2	1.5	3.0	8.1	1.6	3.5	6.0	1.9	3.9
35	62.8	1.0	0.8	42.7	1.1	1.2	31.8	1.1	1.5	24.4	1.2	1.8	19.1	1.2	2.2	15.0	1.3	2.6	12.0	1.4	3.0	9.6	1.6	3.5	7.6	1.7	3.9
40	71.8	1.0	0.8	48.8	1.1	1.2	36.4	1.1	1.5	27.9	1.2	1.8	21.9	1.2	2.2	17.2	1.3	2.6	13.8	1.4	3.1	11.1	1.5	3.5	9.0	1.7	4.0
45	80.8	1.0	0.8	54.9	1.0	1.2	40.9	1.1	1.5	31.3	1.2	1.8	24.6	1.2	2.2	19.6	1.3	2.6	15.5	1.4	3.1	12.6	1.5	3.5	10.2	1.7	4.0
50	89.7	1.0	0.8	60.9	1.0	1.2	45.4	1.1	1.5	34.8	1.2	1.8	27.3	1.2	2.2	21.8	1.3	2.6	17.3	1.4	3.1	14.1	1.5	3.5	11.5	1.6	4.0
55	98.7	1.0	0.8	67.0	1.0	1.2	50.0	1.1	1.5	38.3	1.2	1.8	30.0	1.2	2.2	24.0	1.3	2.6	19.1	1.4	3.1	15.5	1.5	3.5	12.7	1.6	4.0
60	107.7	1.0	0.8	73.1	1.0	1.2	54.5	1.1	1.5	41.8	1.2	1.8	32.7	1.2	2.2	26.1	1.3	2.6	21.0	1.4	3.1	16.9	1.5	3.6	14.0	1.6	4.0
65	116.6	1.0	0.8	79.2	1.0	1.2	59.0	1.1	1.5	45.2	1.2	1.8	35.5	1.2	2.2	28.3	1.3	2.6	22.8	1.4	3.1	18.4	1.5	3.6	15.1	1.6	4.0
70	125.6	1.0	0.8	85.3	1.0	1.2	63.6	1.1	1.5	48.7	1.2	1.8	38.2	1.2	2.2	30.5	1.3	2.6	24.5	1.4	3.1	19.8	1.5	3.6	16.3	1.6	4.1
75	134.6	1.0	0.8	91.4	1.0	1.2	68.1	1.1	1.5	52.2	1.2	1.9	40.9	1.2	2.2	32.6	1.3	2.6	26.2	1.4	3.1	21.5	1.5	3.5	17.5	1.6	4.1
80	143.6	1.0	0.8	97.5	1.0	1.2	72.7	1.1	1.5	55.7	1.2	1.9	43.6	1.2	2.2	34.8	1.3	2.6	28.0	1.4	3.1	23.0	1.5	3.5	18.8	1.6	4.1
85	152.5	1.0	0.8	103.6	1.0	1.2	77.2	1.1	1.5	59.1	1.2	1.9	46.3	1.2	2.2	37.0	1.3	2.6	29.7	1.4	3.1	24.4	1.5	3.6	20.0	1.6	4.1
90	161.5	1.0	0.8	109.7	1.0	1.2	81.7	1.1	1.5	62.6	1.2	1.9	49.1	1.2	2.2	39.1	1.3	2.6	31.5	1.4	3.1	25.8	1.5	3.6	21.2	1.6	4.1
95	170.5	1.0	0.8	115.8	1.0	1.2	86.3	1.1	1.5	66.1	1.2	1.9	51.8	1.2	2.2	41.3	1.3	2.6	33.2	1.4	3.1	27.2	1.5	3.6	22.6	1.6	4.0
100	179.5	1.0	0.8	121.9	1.0	1.2	90.8	1.1	1.5	69.6	1.2	1.9	54.5	1.2	2.2	43.5	1.3	2.6	35.0	1.4	3.1	28.7	1.5	3.6	23.8	1.6	4.0
105	188.4	1.0	0.8	128.0	1.0	1.2	95.4	1.1	1.5	73.0	1.2	1.9	57.2	1.2	2.2	45.6	1.3	2.6	36.7	1.4	3.1	30.1	1.5	3.6	25.0	1.6	4.0
110	197.4	1.0	0.8	134.1	1.0	1.2	99.9	1.1	1.5	76.5	1.2	1.9	60.0	1.2	2.2	47.8	1.3	2.6	38.4	1.4	3.1	31.5	1.5	3.6	26.2	1.6	4.0
115	206.4	1.0	0.8	140.1	1.0	1.2	104.4	1.1	1.5	80.0	1.2	1.9	62.7	1.2	2.2	50.0	1.3	2.6	40.2	1.4	3.1	32.9	1.5	3.6	27.4	1.6	4.0
120	215.3	1.0	0.8	146.2	1.0	1.2	109.0	1.1	1.5	83.5	1.2	1.9	65.4	1.2	2.2	52.2	1.3	2.6	41.9	1.4	3.1	34.4	1.5	3.6	28.6	1.6	4.0
125	224.3	1.0	0.8	152.3	1.0	1.2	113.5	1.1	1.5	86.9	1.2	1.9	68.1	1.2	2.2	54.3	1.3	2.6	43.7	1.4	3.1	35.8	1.5	3.6	29.8	1.6	4.0
130	233.3	1.0	0.8	158.4	1.0	1.2	118.1	1.1	1.5	90.4	1.2	1.9	70.9	1.2	2.2	56.5	1.3	2.6	45.4	1.4	3.1	37.2	1.5	3.6	30.9	1.6	4.0
135	242.3	1.0	0.8	164.5	1.0	1.2	122.6	1.1	1.5	93.9	1.2	1.9	73.6	1.2	2.2	58.7	1.3	2.6	47.2	1.4	3.1	38.6	1.5	3.6	32.1	1.6	4.1
140	251.2	1.0	0.8	170.6	1.0	1.2	127.1	1.1	1.5	97.4	1.2	1.9	76.3	1.2	2.2	60.8	1.3	2.6	48.9	1.4	3.1	40.1	1.5	3.6	33.3	1.6	4.1
145	260.2	1.0	0.8	176.7	1.0	1.2	131.7	1.1	1.5	100.9	1.2	1.9	79.0	1.2	2.2	63.0	1.3	2.6	50.7	1.4	3.1	41.5	1.5	3.6	34.5	1.6	4.1
150	269.2	1.0	0.8	182.8	1.0	1.2	136.2	1.1	1.5	104.3	1.2	1.9	81.7	1.2	2.2	65.2	1.3	2.6	52.4	1.4	3.1	42.9	1.5	3.6	35.7	1.6	4.1

PARABOLIC WATERWAY DESIGN
(RETARDANCE "D" AND "B")

ble 4g.

V1 FOR RETARDANCE "D". TOP WIDTH (T), DEPTH' (D) AND V2 FOR RETARDANCE "B"

GRADE 4.00 PERCENT

Q CFS	V1=2.0			V1=2.5			V1=3.0			V1=3.5			V1=4.0			V1=4.5			V1=5.0			V1=5.5			V1=6.0		
	T	D	V2	T	D	V2	T	D	V2	T	D	V2	T	D	V2	T	D	V2	T	D	V2	T	D	V2	T	D	V2
5	10.1	0.9	0.8	7.0	1.0	1.1	4.9	1.1	1.4																		
10	20.5	0.9	0.8	14.4	0.9	1.1	10.3	1.0	1.4	7.9	1.1	1.8	6.1	1.2	2.1	4.5	1.4	2.4									
15	30.7	0.9	0.8	21.5	0.9	1.1	15.7	1.0	1.4	12.0	1.1	1.8	9.4	1.1	2.1	7.4	1.2	2.5	5.8	1.4	2.8						
20	40.9	0.9	0.8	28.6	0.9	1.1	20.9	1.0	1.4	16.3	1.0	1.8	12.6	1.1	2.1	10.1	1.2	2.5	8.0	1.3	2.9	6.3	1.4	3.3			
25	51.1	0.9	0.8	35.8	0.9	1.1	26.1	1.0	1.4	20.3	1.0	1.8	16.0	1.1	2.1	12.7	1.2	2.5	10.2	1.3	2.9	8.2	1.4	3.4	6.5	1.5	3.8
30	61.3	0.9	0.8	42.9	0.9	1.1	31.4	1.0	1.4	24.4	1:0	1.8	19.2	1.1	2.1	15.2	1.2	2.5	12.3	1.3	2.9	10.0	1.3	3.4	8.1	1.5	3.8
35	71.5	0.9	0.8	50.1	0.9	1.1	36.6	1.0	1.4	28.3	1.0	1.8	22.4	1.1	2.1	18.0	1.2	2.5	14.4	1.2	2.9	11.7	1.3	3.4	9.6	1.4	3.8
40	81.8	0.9	0.8	57.2	0.9	1.1	41.8	1.0	1.5	32.4	1.0	1.8	25.6	1.1	2.1	20.6	1.2	2.5	16.5	1.2	2.9	13.5	1.3	3.4	11.1	1.4	3.8
45	92.0	0.9	0.8	64.4	0.9	1.1	47.0	1.0	1.5	36.4	1.0	1.8	28.8	1.1	2.1	23.1	1.2	2.5	18.8	1.2	2.9	15.2	1.3	3.4	12.6	1.4	3.9
50	102.2	0.9	0.8	71.5	0.9	1.1	52.2	1.0	1.5	40.5	1.0	1.8	32.0	1.1	2.1	25.7	1.2	2.5	20.9	1.2	2.9	17.0	1.3	3.4	14.0	1.4	3.9
55	112.4	0.9	0.8	78.7	0.9	1.1	57.5	1.0	1.5	44.5	1.0	1.8	35.2	1.1	2.1	28.2	1.2	2.5	23.0	1.2	2.9	18.9	1.3	3.4	15.4	1.4	3.9
60	122.6	0.9	0.8	85.8	0.9	1.1	62.7	1.0	1.5	48.5	1.0	1.8	38.4	1.1	2.2	30.8	1.2	2.5	25.1	1.2	2.9	20.6	1.3	3.4	16.9	1.4	3.9
65	132.8	0.9	0.8	93.0	0.9	1.1	67.9	1.0	1.5	52.6	1.0	1.8	41.5	1.1	2.2	33.4	1.2	2.5	27.2	1.2	2.9	22.3	1.3	3.4	18.3	1.4	3.9
70	143.1	0.9	0.8	100.1	0.9	1.1	73.1	1.0	1.5	56.6	1.0	1.8	44.7	1.1	2.2	35.9	1.2	2.5	29.2	1.2	2.9	24.0	1.3	3.4	20.0	1.4	3.9
75	153.3	0.9	0.8	107.3	0.9	1.1	78.3	1.0	1.5	60.7	1.0	1.8	47.9	1.1	2.2	38.5	1.2	2.5	31.3	1.2	2.9	25.7	1.3	3.4	21.4	1.4	3.9
80	163.5	0.9	0.8	114.4	0.9	1.1	83.6	1.0	1.5	64.7	1.0	1.8	51.1	1.1	2.2	41.0	1.2	2.5	33.4	1.2	2.9	27.4	1.3	3.4	22.8	1.4	3.9
85	173.7	0.9	0.8	121.6	0.9	1.1	88.8	1.0	1.5	68.8	1.0	1.8	54.3	1.1	2.2	43.6	1.2	2.5	35.5	1.2	2.9	29.1	1.3	3.4	24.2	1.4	3.9
90	183.9	0.9	0.8	128.7	0.9	1.1	94.0	1.0	1.5	72.8	1.0	1.8	57.5	1.1	2.2	46.2	1.2	2.5	37.6	1.2	2.9	30.8	1.3	3.4	25.7	1.4	3.9
95	194.1	0.9	0.8	135.9	0.9	1.1	99.2	1.0	1.5	76.8	1.0	1.8	60.7	1.1	2.2	48.7	1.2	2.5	39.7	1.2	2.9	32.5	1.3	3.4	27.1	1.4	3.9
100	204.4	0.9	0.8	143.0	0.9	1.1	104.4	1.0	1.5	80.9	1.0	1.8	63.9	1.1	2.2	51.3	1.2	2.5	41.7	1.2	2.9	34.2	1.3	3.4	28.5	1.3	3.9
105	214.6	0.9	0.8	150.2	0.9	1.1	109.7	1.0	1.5	84.9	1.0	1.8	67.1	1.1	2.2	53.9	1.2	2.5	43.8	1.2	2.9	35.9	1.3	3.4	29.9	1.3	3.9
110	224.8	0.9	0.8	157.4	0.9	1.1	114.9	1.0	1.5	89.0	1.0	1.8	70.3	1.1	2.2	56.4	1.2	2.5	45.9	1.2	2.9	37.6	1.3	3.4	31.3	1.3	3.9
115	235.0	0.9	0.8	164.5	0.9	1.1	120.1	1.0	1.5	93.0	1.0	1.8	73.5	1.1	2.2	59.0	1.2	2.5	48.0	1.2	2.9	39.3	1.3	3.4	32.7	1.3	3.9
120	245.2	0.9	0.8	171.7	0.9	1.1	125.3	1.0	1.5	97.1	1.0	1.8	76.7	1.1	2.2	61.5	1.2	2.5	49.9	1.2	3.0	41.0	1.3	3.4	34.2	1.3	3.9
125	255.5	0.9	0.8	178.8	0.9	1.1	130.5	1.0	1.5	101.1	1.0	1.8	79.9	1.1	2.2	64.1	1.2	2.5	52.0	1.2	3.0	42.7	1.3	3.4	35.6	1.3	3.9
130	265.7	0.9	0.8	186.0	0.9	1.1	135.8	1.0	1.5	105.1	1.0	1.8	83.0	1.1	2.2	66.7	1.2	2.5	54.1	1.2	3.0	44.4	1.3	3.4	37.0	1.3	3.9
135	275.9	0.9	0.8	193.1	0.9	1.1	141.0	1.0	1.5	109.2	1.0	1.8	86.2	1.1	2.2	69.2	1.2	2.5	56.1	1.2	3.0	46.1	1.3	3.4	38.4	1.3	3.9
140	286.1	0.9	0.8	200.3	0.9	1.1	146.2	1.0	1.5	113.2	1.0	1.8	89.4	1.1	2.2	71.8	1.2	2.5	58.2	1.2	3.0	47.8	1.3	3.4	39.9	1.3	3.9
145	296.3	0.9	0.8	207.4	-0.9	1.1	151.4	1.0	1.5	117.3	1.0	1.8	92.6	1.1	2.2	74.4	1.2	2.5	60.3	1.2	3.0	49.6	1.3	3.4	41.3	1.3	3.9
150	306.5	0.9	0.8	214.6	0.9	1.1	156.7	1.0	1.5	121.3	1.0	1.8	95.8	1.1	2.2	76.9	1.2	2.5	62.4	1.2	3.0	51.3	1.3	3.4	42.7	1.3	3.9

PARABOLIC WATERWAY DESIGN
(RETARDANCE "D" AND "B")

Table 4h.

V1 FOR RETARDANCE "D". TOP WIDTH (T), DEPTH' (D) AND V2 FOR RETARDANCE "B"

GRADE 5.00 PERCENT

Q CFS	V1=2.0			V1=2.5			V1=3.0			V1=3.5			V1=4.0			V1=4.5			V1=5.0			V1=5.5			V1=6.0		
	T	D	V2	T	D	V2	T	D	V2	T	D	V2	T	D	V2	T	D	V2	T	D	V2	T	D	V2	T	D	V2
5	11.3	0.8	0.8	8.0	0.9	1.1	5.6	1.0	1.4	4.2	1.1	1.6															
10	22.5	0.8	0.8	16.3	0.9	1.1	11.5	0.9	1.4	8.9	1.0	1.7	7.0	1.0	2.0	5.5	1.2	2.4	4.0	1.4	2.6						
15	33.7	0.8	0.8	24.3	0.9	1.1	17.4	0.9	1.4	13.7	1.0	1.7	10.7	1.0	2.1	8.5	1.1	2.4	6.8	1.2	2.8	5.4	1.3	3.2			
20	45.0	0.8	0.8	32.4	0.9	1.1	23.2	0.9	1.4	18.2	1.0	1.7	14.5	1.0	2.1	11.5	1.1	2.5	9.3	1.1	2.8	7.5	1.2	3.2	6.1	1.4	3.7
25	56.2	0.8	0.8	40.5	0.9	1.1	28.9	0.9	1.4	22.8	1.0	1.7	18.1	1.0	2.1	14.6	1.1	2.4	11.7	1.1	2.8	9.6	1.2	3.3	7.8	1.3	3.7
30	67.4	0.8	0.8	48.7	0.9	1.1	34.7	0.9	1.4	27.3	1.0	1.7	21.7	1.0	2.1	17.5	1.1	2.4	14.1	1.1	2.8	11.6	1.2	3.3	9.5	1.3	3.7
35	78.7	0.8	0.8	56.8	0.9	1.1	40.5	0.9	1.4	31.8	1.0	1.7	25.3	1.0	2.1	20.4	1.0	2.5	16.7	1.1	2.8	13.6	1.2	3.3	11.2	1.3	3.7
40	89.9	0.8	0.8	64.9	0.9	1.1	46.3	0.9	1.4	36.4	1.0	1.7	28.8	1.0	2.1	23.3	1.0	2.5	19.1	1.1	2.8	15.6	1.2	3.3	12.9	1.2	3.7
45	101.1	0.8	0.8	73.0	0.9	1.1	52.1	0.9	1.4	40.9	1.0	1.7	32.4	1.0	2.1	26.2	1.0	2.5	21.5	1.1	2.8	17.7	1.2	3.3	14.6	1.2	3.7
50	112.4	0.8	0.8	81.1	0.9	1.1	57.9	0.9	1.4	45.5	1.0	1.7	36.0	1.0	2.1	29.1	1.0	2.5	23.9	1.1	2.8	19.7	1.2	3.3	16.2	1.2	3.7
55	123.6	0.8	0.8	89.2	0.9	1.1	63.6	0.9	1.4	50.0	1.0	1.7	39.6	1.0	2.1	32.0	1.0	2.5	26.2	1.1	2.8	21.7	1.2	3.3	18.0	1.2	3.8
60	134.8	0.8	0.8	97.3	0.9	1.1	69.4	0.9	1.4	54.5	1.0	1.7	43.2	1.0	2.1	34.9	1.0	2.5	28.6	1.1	2.8	23.6	1.2	3.3	19.7	1.2	3.8
65	146.1	0.8	0.8	105.4	0.9	1.1	75.2	0.9	1.4	59.1	1.0	1.7	46.8	1.0	2.1	37.8	1.0	2.5	31.0	1.1	2.8	25.6	1.2	3.3	21.3	1.2	3.8
70	157.3	0.8	0.8	113.5	0.9	1.1	81.0	0.9	1.4	63.6	1.0	1.7	50.4	1.0	2.1	40.7	1.0	2.5	33.4	1.1	2.8	27.5	1.2	3.3	22.9	1.2	3.8
75	168.6	0.8	0.8	121.6	0.9	1.1	86.8	0.9	1.4	68.2	1.0	1.7	54.0	1.0	2.1	43.6	1.0	2.5	35.8	1.1	2.8	29.4	1.2	3.3	24.5	1.2	3.8
80	179.8	0.8	0.8	129.7	0.9	1.1	92.6	0.9	1.4	72.7	0.9	1.7	57.6	1.0	2.1	46.5	1.0	2.5	38.1	1.1	2.8	31.4	1.2	3.3	26.2	1.2	3.8
85	191.0	0.8	0.8	137.8	0.9	1.1	98.3	0.9	1.4	77.3	0.9	1.7	61.2	1.0	2.1	49.4	1.0	2.5	40.5	1.1	2.8	33.3	1.2	3.3	27.8	1.2	3.8
90	202.3	0.8	0.8	145.9	0.9	1.1	104.1	0.9	1.4	81.8	0.9	1.7	64.9	1.0	2.1	52.3	1.0	2.5	42.9	1.1	2.8	35.3	1.2	3.3	29.4	1.2	3.8
95	213.5	0.8	0.8	154.0	0.9	1.1	109.9	0.9	1.4	86.3	0.9	1.7	68.5	1.0	2.1	55.2	1.0	2.5	45.3	1.1	2.8	37.2	1.2	3.3	31.1	1.2	3.8
100	224.7	0.8	0.8	162.1	0.9	1.1	115.7	0.9	1.4	90.9	0.9	1.7	72.1	1.0	2.1	58.1	1.0	2.5	47.7	1.1	2.8	39.2	1.2	3.3	32.7	1.2	3.8
105	236.0	0.8	0.8	170.2	0.9	1.1	121.5	0.9	1.4	95.4	0.9	1.7	75.7	1.0	2.1	61.0	1.0	2.5	50.0	1.1	2.8	41.1	1.2	3.3	34.3	1.2	3.8
110	247.2	0.8	0.8	178.3	0.9	1.1	127.3	0.9	1.4	100.0	0.9	1.7	79.3	1.0	2.1	64.0	1.0	2.5	52.4	1.1	2.8	43.1	1.2	3.3	36.0	1.2	3.8
115	258.5	0.8	0.8	186.4	0.9	1.1	133.0	0.9	1.4	104.5	0.9	1.7	82.9	1.0	2.1	66.9	1.0	2.5	54.8	1.1	2.8	45.0	1.2	3.3	37.6	1.2	3.8
120	269.7	0.8	0.8	194.6	0.9	1.1	138.8	0.9	1.4	109.1	0.9	1.7	86.5	1.0	2.1	69.8	1.0	2.5	57.2	1.1	2.8	47.0	1.2	3.3	39.2	1.2	3.8
125	280.9	0.8	0.8	202.7	0.9	1.1	144.6	0.9	1.4	113.6	0.9	1.7	90.1	1.0	2.1	72.7	1.0	2.5	59.6	1.1	2.8	48.9	1.2	3.3	40.9	1.2	3.8
130	292.2	0.8	0.8	210.8	0.9	1.1	150.4	0.9	1.4	118.2	0.9	1.7	93.7	1.0	2.1	75.6	1.0	2.5	61.9	1.1	2.8	50.9	1.2	3.3	42.5	1.2	3.8
135	303.4	0.8	0.8	218.9	0.9	1.1	156.2	0.9	1.4	122.7	0.9	1.7	97.3	1.0	2.1	78.5	1.0	2.5	64.3	1.1	2.8	52.9	1.2	3.3	44.1	1.2	3.8
140	314.6	0.8	0.8	227.0	0.9	1.1	162.0	0.9	1.4	127.2	0.9	1.7	100.9	1.0	2.1	81.4	1.0	2.5	66.7	1.1	2.8	54.8	1.2	3.3	45.8	1.2	3.8
145	325.9	0.8	0.8	235.1	0.9	1.1	167.8	0.9	1.4	131.8	0.9	1.7	104.5	1.0	2.1	84.3	1.0	2.5	69.1	1.1	2.8	56.8	1.2	3.3	47.4	1.2	3.8
150	337.1	0.8	0.8	243.2	0.9	1.1	173.5	0.9	1.4	136.3	0.9	1.7	108.1	1.0	2.1	87.2	1.0	2.5	71.5	1.1	2.8	58.7	1.2	3.3	49.0	1.2	3.8

PARABOLIC WATERWAY DESIGN
(RETARDANCE "D" AND "B")

Design examples

Parabolic vegetated waterways

A vegetated waterway must be able to carry the design flow and resist erosion. When the grass is long and unmowed, the velocity will be at a minimum and is represented by V_2 in the design tables. Erosion occurs when the grass is short and the velocity is high (V_1 from the tables). A design using the tables results in a channel with adequate capacity when the grass is long and thick, that resists erosion when mowed and that has adequate freeboard during the design flow.

To use the tables, first determine the peak rate of runoff from the design storm (Q in cfs), field verify the channel slope and select the desired grass liner. The permissible velocity (V_1) is based on the liner. Using the required capacity (Q) and the channel slope, with the permissible velocity (V_1), you can determine the top width (T in feet) and the depth (D in feet) for the correct parabolic section.

Design problem 1

Design a parabolic waterway on erosion resistant soils planted in redtop, with a channel slope of 2% and a peak runoff of 20 cfs.

Solution

Using table 2 to determine the permissible velocity of redtop as 3.5 ft/sec and given Q as 20 cfs, look up T and D from table 4a for a 2% slope and "C" retardance (from table 3 for redtop).

T = 9.4 feet, D = 1.1 feet

The design engineer will need to determine the reduction in volume after flows pass through the swale. This reduction is a function of the dynamic percolation rate, the rain duration, the volume of runoff coming to the swale and the area of the swale. The dynamic infiltration rate is typically assumed to be approximately half of the static infiltration rate measured by an in-field double ring infiltrometer test. The following equations assume the swale was designed for infiltration and that it is neither too steep nor too short (Pitt, 1989).

The ratio of the infiltration volume over the runoff volume should be a fraction less than one for the rainfall event used in the design. If the ratio is greater than one, the swale is larger than it needs to be for that rain event; that is, more is infiltrating than is coming in. Before calculating the volume reduction, this fraction must equal one or less (use one if the ratio is greater than one). The ratio (A) of infiltration volume over runoff volume is a result of the following dimensions:

$$A = \frac{\text{infiltration rate (ft/hr)} \times \text{swale density (ft/acre)} \times \text{swale width (ft)} \times \text{basin area (acres)} \times \text{runoff duration (hr)}}{\text{runoff volume (ft}^3\text{)}}$$

The infiltration rate is the dynamic percolation rate as described earlier. The swale density is the feet of swale per acre of drainage area and should be determined on a case-by-case basis. Some swale density values observed by Pitt (1989) follow.

Land use	Swale density (ft/acre)
Low density residential	160
Medium density residential	350
Shopping centers	280
Industrial	125

The swale width is the wetted width, and the basin area is the area served by swales. Using techniques described in the hydrology chapter of this manual, runoff duration is equal to 0.9 + (0.98) x t when t is the duration of the precipitation event (hours).

A second calculation is the ratio of the area served by the swale over the total drainage basin (B).

$$B = \frac{\text{area served by swales (acres)}}{\text{area of the drainage basin (acres)}}$$

A X B = C the study area runoff reduction due to the grassed swale (calculated as a fraction).

The runoff volume multiplied by one minus this reduction fraction (1-C) equals the runoff volume after drainage controls (swales).

Design problem 2

The runoff volume from a 50-acre, medium-density residential area for a 4-hour rain event is equal to 190,000 ft^3. The in-field double ring infiltrometer test indicates an infiltration rate of 3.0 in/hr for the soils in the area. Only 25 acres of the total area will be served by swales. The swale dimensions will be the same as in the previous example (top width equals 9.4 feet). What is the runoff volume after swales?

Solution

1. The dynamic infiltration rate is assumed to be ½ the measured value or 1.5 in/hr.
2. The wetted width (p) using the calculation in figure 2 for a parabolic channel is 9.74 feet.
3. The runoff duration is equal to 0.9 + .98(4) or 4.8 hours.
4. The infiltration volume over the runoff volume equals:

$$\frac{(1.5 \text{ in/hr})(350 \text{ ft/ac.})(9.74 \text{ ft.})(25 \text{ ac.})(4.8 \text{ hr})(1\text{ft}/12\text{in})}{(190000 \text{ ft}^3)}$$

$$= 0.27$$

5. The area served by swales divided by the total area equals

   25ac./50 ac. = 0.5

6. The expected reduction in flow as a result of the swales is

(.27)(0.5) = 0.135, or 13.5% reduction.

7. The remaining runoff volume after swales is

   190,000 ft^3) (1-.135) = 164,350 ft^3.

Construction guidelines

Plans and specifications for construction must include the swale location, alignment, grade, depth, width, seeding specification and dates, underdrains (if applicable), inlet and outlet structures, schedule for installation and inspection and maintenance requirements.

Site preparation consists of excavation, filling, shaping and grading. Construction site runoff should be diverted around the swale, and upland slopes should be stabilized prior to start-up of the swale to protect water quality and reduce the potential for early clogging. The site should be stripped of unsuitable material and areas smoothed by equipment should be scarified. Care must be taken if fill material is required in areas where unsuitable material was removed. Compaction of an infiltrative surface must be avoided. Heavy equipment is discouraged and equipment with oversized tires is preferred.

Soils should be tilled prior to seeding or sodding. Locating the swale in a sunny location over soils of sufficient depth and texture is essential if a healthy, vigorous grass mat is to develop. Grasses in a swale should be selected for their high stem density, drought and salt tolerance, well-branched top growth, nonbunching characteristics, root systems that can withstand temporary flooding, stems that can resist flattening and aggressive growth. Flow should be kept out of the swale until the vegetation is well established. Seeding should include the use of lime, fertilizer, mulch and tackifiers to hold down the seed until it germinates.

Check dams

Selection of check dams and their proper installation may determine the channel's stability and the swale's effectiveness in storing flows. Low-head, ported or notched check dams at heights less than 12 inches are preferred. Earth and stone check dams require more maintenance and do not last as long. Stone piled downstream of the dam will prevent downstream scour. Construction of a sediment trap or vegetative filter strip ahead of the swale provides additional protection against sediment build-up.

Maintenance

A detailed operation and maintenance manual for the specific swales should be provided to the responsible party. The primary maintenance responsibility of a grassed swale is care of the vegetative liner. Vegetative liners have intensive maintenance requirements. Establishment of sod or seed requires regular attention until the mat is dense and mature. Pesticides and fertilizer should be used in moderation, and only if important in establishing or maintaining a dense vegetation.

Mowing is necessary to encourage growth but must be done at the correct height for the swale's operating depth. Grass should be maintained at a minimum height of 6 inches; more importantly, grass must be maintained at a height above the operating depth for the

1.5-inch rain. Depending on the natural height of the selected grasses, mowing may be infrequent or unnecessary.

It is very important to inspect for channelization or undesirable woody vegetation. Because of the slow design flow velocities of swales designed primarily for water quality improvement and infiltration, sediment may accumulate in the bottom. Sediment removal may be necessary, but take care to minimize serious disturbance of the vegetation. After sediment is removed, reseed bare spots immediately.

Homeowners or homeowners' associations have sometimes been expected to maintain the grass. However, homeowners may mow at varying heights (since most do not have a sickle-bar type mower to maintain a 6-inch minimum height) or too short, which damages the grass mat. For maintenance consistent with the design and purpose of the swale, mowing and other maintenance responsibilities should stay with the local government.

Inspections should occur seasonally and after major storms. In addition to looking for sedimentation, the maintenance crew should look for bypassing around check dams. Channels and low spots should be regraded and seeded. (If a swale needs to come on line in a short amount of time, use sod rather than seed.)

Crews should also check for nuisance conditions such as mosquitoes, weeds, woody growth and trash dumping which can occur in a relatively short period of time. Post signs to inform local residents of the swale's purpose and to discourage dumping of leaves or parking on the edge of the swale. Curb blocks installed an appropriate distance from the swale will discourage parking. Swales along highways or in median strips are subjected to salting, so the vegetation should be salt tolerant. If salt is a problem and a vigorous grass mat has not developed, the area may need to be stripped and a different seed mixture used.

Swale sites generally do not have a high habitat potential. If wildlife habitat is desired and space is available, a no-mow buffer strip around the swale of 10-12 feet can serve as habitat and improve swale performance. Maintenance requirements for this area are minimal, but take care to discourage undesirable plants in the buffer strip from invading the swale.

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