LOW IMPACT DEVELOPMENT

DESIGN GUIDANCE MANUAL

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Watershed Management Services

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Low Impact Development Design Guidance Manual

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ACRONYMS AND ABBREVIATIONS

AAC .... Alaska Administrative Code
ARC .... Auckland Regional Council
ASTM .... American Society for Testing and Materials
BMP .... Best Management Practice
DDG .... Drainage Design Guidelines
EPA .... United States Environmental Protection Agency
EPDM .... ethylene propylene diene monomer
F .... Fahrenheit
ft. .... Feet
in .... Inches
LID .... Low Impact Development
MCBUR .... Monolithic multi–ply hot asphalt mineral surfaced built–up–roof
MDEP .... Massachusetts Department of Environmental Protection
MMC .... Minneapolis Metropolitan Council
MOA .... Municipality of Anchorage
PSAT .... Puget Sound Action Team
PVC .... Polyvinylchloride
SCS .... Soil Conservation Service
TPO .... thermoplastic olefin
TSS .... Total Suspended Solids
US .... United States
WMS .... Watershed Management Services

VARIABLES AND CONSTANTS

A .... Contributing Area (feet^2)
A_a .... Contributing Area (acres)
A_s .... Soak–Away Pit Footprint (feet^2)
A_r .... Rain Garden Footprint (feet^2)
A_i .... Trench Footprint (feet^2)
A_o .... Orifice Opening Area (feet^2)
C .... Runoff Coefficient per the Drainage Design Guidelines
C_d .... Coefficient of Discharge
D_s .... Soak–Away Pit Depth (feet)
D_i .... Infiltration Trench Depth (feet)
D_r .... Total Depth of Rain Garden Without Subdrain (feet)
D_{rs} .... Total Depth of Rain Garden With Subdrain (feet)
E_d .... Depth of the Engineered Soils (feet)
F_d .... Freeboard (inches)
G .... Grate Reduction Factor
g .... Gravitational Constant (feet/second^2)
H .... Head (feet)
h_b .... Height of the Berm (feet)
I .... Design Infiltration Rate (inches/hour)
I_e .... Infiltration Rate of Engineered Soils (inches/hour)
L_f .... With of Filter Strip Parallel to Flow Path (feet)
L_i .... Length of Infiltration Trench (feet)
L_r .... Approximate Length of Rain Garden Along the Axis of the Subdrain (feet)
N .... Manning's "n" Roughness Coefficient
N_o .... Number of Orifices
N_s .... Number of Outfall Structures
n_s .... Storage Media Void Ratio
P .... Target Precipitation (inches)
P_d .... Depth of Ponded Water (inches)
P_s .... Perimeter of the Stand Pipe (feet)
Q .... Flow Rate (feet^3/second)
q .... Volumetric Discharge per Foot Width (feet^3/second-foot)
S .... Slope of Filter Strip (ft/ft)
S_d .... Depth Required for Subdrain Diameter and Drain Rock (feet)
t .... Retention Time (hours)
TIV .... Target Infiltration Volume (feet^3)
T_t .... Travel Time through Filter Strip (minutes)
V_w .... Volume of a Wedge (feet^3)
V .... Velocity (feet/second)
W_f .... Width of Filter Strip Parallel to Flow (feet)
W_{fp} .... Width of Filter Strip Perpendicular to Flow Path (feet)
W_i .... Width of Infiltration Trench (feet)
W_{if1} .... Width of Infiltration Trench and One Filter Strip (feet)
W_{if2} .... Width of Infiltration Trench and Two Filter Strips (feet)
W_r .... Width of Rain Garden (feet)
Y .... Allowable Depth of Flow (inches)

1. Introduction

The *Low Impact Development Design Guidance Manual* has been developed by the Municipality of Anchorage (MOA) to provide the engineering and development community with additional guidance for the design of infiltration controls introduced in the MOA *Drainage Design Guidelines*, 2007 (DDG). This manual also introduces other infiltration controls for consideration such as constructed wetlands and pervious pavements. The application of such mechanisms and strategies is referred to as Low Impact Development (LID). This manual is a starting point for the development of a comprehensive design guidance manual that will include additional information on a wider range of infiltration and storm water management mechanisms and strategies.

LID is a storm water management strategy that focuses on maintaining or restoring the natural hydraulic functions of a site for the purpose of water resources protection. LID uses a decentralized approach that disperses flows and manages runoff closer to where it originates, as opposed to collecting storm water in a piped or channelized network and managing it at a large–scale "end of pipe" location. This management practice focuses on mimicking the natural retention, filtration, and infiltration mechanisms that storm water runoff would encounter on an undeveloped site. Therefore, the most important factor to consider in the application of LID to site design is the preservation of native vegetation and natural drainage features.

> *"An essential part of the LID approach is conserving portions of the site in its predeveloped state to preserve the hydrologic functions of the site. To achieve this, site planners should identify and preserve areas that most affect hydrology, such as streams, wetlands, floodplains, steep slopes, and high–permeability soils. The development layout should be adjusted to reduce, minimize, and disconnect the total impervious area. Finally, on–site options for handling runoff from the impervious areas should be employed before conventional off–site storm water practices are used."*
> *(MOA, 2004)*

In addition to the importance of preserving native vegetation and natural drainage features, gains are made in the effort to mimic natural conditions by reducing and or disconnecting proposed impervious surfaces. Areas of pavement that can be easily broken up into multiple disconnected impervious surfaces include traffic lanes, parking lots, and paved walkways. Traffic lanes can be separated by pervious medians that receive runoff from roadway surfaces. Parking lots can be designed to incorporate vegetated strips of land to collect and convey runoff. Paved walkways can be separated from roadways by vegetated strips of land providing not only opportunities for infiltration but also increase pedestrian safety.

While water quality treatment is not the principle purpose of LID, these practices also provide water quality benefits. Overall reduction in surface runoff reduces the volume of runoff that can potentially transport pollutants. Infiltration as an LID technique reduces the mass of pollutants by filtration of particles and adsorption of chemicals to soil.

The DDG provides general guidance for the design of LID elements including: infiltration surfaces, basins, and trenches, and soak–away pits. This LID manual provides additional guidance for the design of the following LID elements: filter strips (a type of infiltration surface), rain gardens (a type of infiltration basin), infiltration trenches, and soak–away pits. This manual also includes discussions of other LID elements that are applicable for storm water treatment in the Anchorage area.

The design guidance presented in this manual is based in part on the requirements presented in the DDG. When performing the design of an LID element, guidance presented in both manuals should be followed. This guidance is provided to facilitate and encourage the usage of LID elements in development and redevelopment projects within the MOA. **The guidance provided in this manual is not intended to supplant professional judgment.**

1.1 *Costs and Benefits of LID*

In 2007, the Environmental Protection Agency (EPA) published a report titled *Reducing Storm Water Costs* through *Low Impact Development: Strategies and Practices* (EPA, 2007). The report compares the projected or known costs of LID practices with those of conventional storm water management approaches. The EPA defines "traditional approaches" to storm water management as those that typically involve hard infrastructure such as curbs, gutters, and piping.

The report indicates that LID techniques can significantly reduce infrastructure costs by eliminating the need for extensive storm water infrastructure such as underground conveyance systems. The report also notes that by infiltrating or evaporating runoff, LID techniques can reduce the size and cost of flood control structures. In some circumstances, LID practices can offset the costs associated with regulatory requirements for storm water control. However, it should be noted that LID techniques may in some cases result in higher costs due to expensive plant materials, additional site preparation, soil amendments, construction of underdrains, and increased project management costs. Other cost considerations include the amount of land required to implement LID practices and potential additional maintenance requirements.

The above–mentioned cost consideration notwithstanding, case studies reviewed in the EPA report demonstrate that LID practices can reduce project costs and improve the overall environmental performance of a development. Though not all the benefits of the LID applications were monetized, with a few exceptions, LID practices were shown to be both fiscally and environmentally beneficial to communities. In a few case studies, initial project costs were higher than those for conventional designs. In most cases, however, significant savings were realized due to reduced costs for site grading and preparation, storm water infrastructure, and site paving. Total capital cost savings ranged from 15 to 80% when LID techniques were used.

The project benefits that were not monetized in the EPA study include improved aesthetics, expanded recreational opportunities, increased property values due to the desirability of the lots and their proximity to open space, increased marketing potential, and faster sales. These are all positive impacts that LID can bring to the surrounding community. On a municipal level, the EPA case studies indicate benefits such as reduced runoff volumes and pollutant loadings to downstream waters, and reduced incidences of combined sewer overflows. These benefits save taxpayer dollars and reduce pollution in downstream waters that support wildlife and recreation. This manual is intended to give the design community some of the design tools necessary to implement LID on residential, commercial, and transportation related projects so that both the monetary and non–monetary benefits discussed here can be realized.

In addition to the benefits discussed above, LID elements such as rain gardens and filter strips can be used to meet drainage requirements in the DDG as well as Title 21 landscaping requirements.

1.2 *Class V Injection Wells*

In order to provide clarification on which storm water infiltration practices/technologies have the potential to be regulated as Class V injection wells by the Underground Injection Control Program, the EPA released a memorandum addressing the subject in June 2008. The memorandum generally states that LID elements with depths less than the longest plan view dimension are not considered Class V injection wells. The June 2008 memorandum is provided in Appendix A of this manual.

1.3 *How to Use this Manual*

It is not necessary for designers to read every section of this manual to design a particular LID element. After reading Section 1, designers may turn to the section that addresses the particular LID element of interest. However, being familiar with the design considerations associated with each LID element will greatly assist designers in the proper selection of the element best suited for a particular application.

1.3.1 General Structure of the Design Guidance Sections

This manual contains four major LID design guidance sections.

- Section 2: Rain Gardens – Shallow depressions planted with vegetation, underlain by either local or engineered soils and, in some cases, a subdrain and/or impermeable liner.

- Section 3: Infiltration Trenches – Rectangular excavations lined with geotextile filter fabric and filled with coarse stone aggregate that serve as underground infiltration reservoirs for sheet flow runoff from impervious surfaces such as parking areas.

- Section 4: Soak–away Pits – Small excavations lined with filter fabric filled with coarse stone aggregate that serve as underground infiltration reservoirs for runoff from roof tops.

- Section 5: Filter Strips – Gently sloped, vegetated areas designed to decelerate, filter, and intercept sheet flow storm water runoff.

The development of a proper LID element design can be accomplished by following the guidance provided in Sections 2, 3, 4, and 5 of this manual. Section 6 is provided to introduce additional LID elements for consideration. While the guidance provided in Section 6 is not as in–depth as that provided in the other sections, the information should be adequate to assist designers in the appropriate application and design for these elements.

A brief description of the LID element is provided at the beginning of each section. The design process is then presented in three major sections: preliminary site evaluation, preliminary design, and final design. In the preliminary site evaluation subsection, the minimum considerations to be evaluated to establish that a site is, or is not, a good candidate for the use of the particular LID element are presented. These considerations are in addition to the basic site evaluation considerations presented in Subsection 1.4. At the end of each preliminary site evaluation subsection, a checklist is introduced to assist designers in conducting a preliminary site evaluation. In the preliminary design subsection, the minimum considerations to be evaluated during the preliminary design of each LID element are presented. Where necessary, these discussions include equations to be used during the preliminary design. At the end of each preliminary design subsection, a calculation table is introduced to assist designers in conducting a preliminary design. In the final design subsection, the minimum considerations to be addressed during the final design are discussed.

Design examples for each of the four LID elements are provided in the appendices of this manual. Each design example starts with a brief description of the theoretical site being considered for the application of the particular LID element. The description is followed by a checklist for an example preliminary site evaluation. The preliminary design example is then presented using a preliminary design calculation table. In the final design example sections, discussions are provided of how the minimum considerations presented in each section are to be addressed in the final design. Conceptual design figures are also presented.

1.3.2 Selecting an LID Element

Rain gardens, infiltration trenches, and soak–away pits are suitable for applications where infiltration of the adjusted 1–year, 24–hour storm event is desired. In Figure 2–1 of the DDG, it can be seen that small and large projects that completely infiltrate runoff from the base 1–year, 24–hour storm are exempt from the requirement of on–site extended detention. Additionally, the infiltration of runoff from the adjusted 1–year, 24–hour storm may fulfill the requirement for water quality protection. Thus, by incorporating these LID elements into small and large developments, designers can potentially limit the amount of infrastructure required to meet the requirements listed in the DDG.

Filter strips are suitable for applications where treatment of the first flush of runoff is desired to meet the water quality requirements presented in the DDG. Filter strips are also suitable for use as pretreatment devices upstream of other LID elements such as rain gardens and infiltration trenches.

Each of the elements presented in Sections 2 through 5 of this manual are suitable to a wide range of applications. Table 1 below provides some suggestions for suitable applications for each element. To perform a detailed evaluation of whether or not a particular LID element is suitable for application to a particular site or portion of a site, performance of a preliminary site evaluation and a preliminary site design is required.

Table 1 – Suggested Suitable Applications for LID Elements

LID Element	Parking Lot Runoff	Roof Top Runoff	Roadway Runoff	Airport Drainage	Residential Development	Pretreatment
Rain Gardens	Yes	Yes	Yes	No	Yes	No
Infiltration Trenches	Yes	Yes	Yes	Yes	Yes	No
Soak–away Pits	No	Yes	No	No	Yes	No
Filter Strips	Yes	No	Yes	Yes	Yes	Yes

1.4 *Basic Site Evaluation Considerations*

The considerations listed below should be included in the site evaluation for each of the LID elements in Section 2 through Section 5. Considerations specific to the particular elements are listed under the preliminary site evaluation discussion within each section.

1.4.1 Infiltration Rate of the Surrounding Soil

The utility of LID elements such as rain gardens, infiltration trenches, and soak–away pits is dependent on the rate at which the local soil can infiltrate storm water. To operate properly, these LID elements should completely infiltrate storm water runoff from a particular event prior to the start of another precipitation event. Thus, soils with low infiltration rates are not desirable. Conversely, to provide adequate treatment for storm water and protect groundwater aquifers, excessively high infiltration rates are not desirable.

Infiltration rates must be estimated based on site investigations. Infiltration testing includes soil borings or test pits in the vicinity of the proposed facility as well as physical in-situ infiltration tests. Acceptable methods for performing this testing are specified in the DDG. The acceptable range of measured infiltration rates of soils in an area being considered for use of these LID elements is 0.3 to 8 inches/hour (MOA, 2007a; MOA, 2004). These infiltration rates must be representative of the soil at the bottom of the proposed facility (MOA, 2007a). The minimum infiltration rate does not apply to rain gardens with impermeable liners (known as “lined rain gardens”).

For design purposes, the measured infiltration rate of soils is adjusted using a factor of safety to account for soil non-homogeneity and to reflect reduction in infiltration capacity over the life of the facility. Equations in this manual use design rather than measured infiltration rates and 1 inch per hour is specified as the maximum design infiltration rate.

Use of higher design infiltration rates may be allowed, based on site specific investigation performed in accordance with the DDG and with an appropriate factor of safety (MN PCA, 2008; WI DNR, 2004).

1.4.2 Separation Distance from Wells and Surface Water

Due to water quality concerns, it is necessary to consider the proximity of LID elements to drinking water wells and surface waters. According to 18 Alaska Administrative Code (AAC) 80.020, Table A, LID elements including unlined rain gardens, infiltration trenches, and soak–away pits must be separated by a horizontal distance of 200 feet from Class A or B wells and 100 feet from Class C wells. In order to protect surface water, these elements should be located at least 100 horizontal feet from the bank of any adjacent surface waters. These considerations do not apply to lined rain gardens.

1.4.3 Depth to Groundwater

To protect groundwater resources, it is important to provide ample separation between LID elements and the surface of the local groundwater table. The minimum separation distance between the seasonal high groundwater table elevation and the bottom of infiltration trenches and soak–away pits is 4 feet. The minimum separation distance between the seasonal high groundwater table elevation and the surface of an unlined rain garden is 4 feet. Due to difficulties with rain garden construction at or near the groundwater surface, the minimum separation distance between the bottom of lined rain gardens and the seasonal high groundwater table elevation is 2 feet.

1.4.4 Depth to Bedrock or Relatively Impervious Soils

Bedrock or Hydrologic Soil Group Class D soils directly below the bottom of LID elements can have undesirable effects, such as limiting the infiltrative capacity of the element, or in the case of highly fractured bedrock, allowing untreated discharge to reach groundwater. To reduce the possibility of limited infiltration or treatment due to the presence of bedrock or impervious soils, the minimum separation distance between these materials and the bottom of unlined rain gardens, infiltration trenches, and soak–away pits is 3 feet (MOA, 2007a).

1.4.5 Separation Distance from Foundations and Road Subgrades

To limit the possibility of damage to permanent structures through frost heave and other freeze–thaw mechanisms, unlined rain gardens, infiltration trenches, and soak–away pits must be either outside of the zone of influence of foundations and road subgrades or separated from these structures by a horizontal distance of 20 feet (Caraco, 1997). The zone of influence refers to the area of the surrounding subgrade that is critical to proper function and support of the overlying and/or adjacent foundation or road subgrade. The zone of influence can be defined as the area bounded within a 3–dimensional surface extending at a 1:1 slope down and away from the outer edge of a foundation or road subgrade. An additional horizontal setback may be required when there is potential for surface seepage due to the vertical elevation difference between the bottom of the infiltration facility and adjacent land or property due to steep slopes or retaining walls.

1.5 *Construction Considerations*

Construction of the LID elements discussed in Sections 2 through 5 of this manual shall incorporate the considerations discussed below in addition to those provided in the construction considerations discussion presented in the section specific to each LID element.

1.5.1 Excavation

Care must be taken during the excavation of areas for LID elements to assure that the existing infiltrative capacity of the soil is not reduced due to compaction. Excavation should be performed by machinery operating adjacent to the excavated area, if possible. When it is necessary for excavation equipment to operate within the footprint of an LID element, lightweight, low ground contact pressure equipment should be used. Heavy equipment with narrow tracks, narrow tires or large lugged, high pressure tires should not be allowed on the bottom of the excavations. Following excavation, the base of the excavation should be ripped to refracture the soil to a minimum of 12 inches (PSAT, 2003).

1.5.2 Excess Sediment

Care must be taken to assure that LID elements are not overburdened with sediment generated by construction in adjacent areas. LID elements should not be used as sediment control facilities for construction. Runoff from adjacent construction should be directed away from LID elements with temporary diversion swales or other protection. Flow to newly constructed LID elements should not be allowed until all of the contributing area is stabilized according to the satisfaction of the engineer (PSAT, 2003).

1.6 *Separation from Underground Utilities*

Generally, LID elements should have the following separation distances from underground utilities:

Wastewater – 10 feet	Drinking Water – 10 feet
Electric – 6 feet	Gas – 6 feet

Deviation from these separation distances may be granted at the discretion of the MOA Project Management and Engineering Department and in cooperation with the utility company or companies.

1.7 *Equations*

This document contains a number of design equations that are provided to assist the development community in the proper design of the LID elements presented in this manual. Many of these equations have been developed specifically for application in the MOA, and thus will not be found in other LID guidance documents. A brief discussion of each equation, including an explanation of constants, is provided in Appendix B.

1.8 *LID Design Notes*

The following design notes are common to the design of rain gardens, infiltration trenches, soak away pits, and filter strips.

- Rainfall Depth: The guidance provided in this manual has been developed in part to assist the development community in the design of LID elements capable of infiltrating the base 1–year, 24–hour event. Thus, the rainfall depths used in the design of LID elements have not been multiplied by an orographic factor, as discussed in Chapter 2 of the Municipality of Anchorage *Design Criteria Manual*. However, it should be noted that flood bypass structures associated with LID elements should apply the appropriate orographic factor according to the project location.

- Runoff Coefficient per the DDG: The preliminary design process for rain gardens, infiltration trenches, soak–away pits, and filter strips, requires the calculation and input of the Runoff Coefficient. The term “Runoff Coefficient” is used in this document to refer to the “Rational Method Coefficient” as described in the DDG. In all cases, the Runoff Coefficient is to be calculated according to guidance contained in the DDG.

- Soil Infiltration Rates: The design of rain gardens, infiltration trenches, soak–away pits, and filter strips requires knowledge of the local infiltration rate. In addition, when engineered soil is used in a rain garden design, the design process requires knowledge of the infiltration rate of the engineered soil. Designers are referred to guidance provided in the DDG to measure local soil infiltration rates. Measured infiltration rates should be adjusted to design infiltration rates using appropriate factors of safety (WI DNR, 2004). For estimation of the infiltration rate for engineered soils, designers are referred to Appendix C of this manual.

- Overflow Structures: In all cases, overflow structures for LID elements should be designed and sized to assure that during a 100–year 24–hour storm water is provided a clear, safe, non–destructive path to an appropriately sized conveyance system without causing any kind of localized flooding.

- Target Infiltration Volume (TIV): The term Target Infiltration Volume is used in this manual to define the target volume for design of LID elements. The term is similar to Water Quality Volume used in accordance with the common language of LID. However, TIV is not necessarily equivalent to the DCM Chapter 2 criterion for water quality protection volume.

2. Rain Gardens

Note: This section provides specific guidance for the development community for the design and construction of rain gardens. Guidance more appropriate for homeowners who wish to incorporate a rain garden into their landscaping is provided in the MOA publication Rain Gardens: A How–To Manual for Homeowners in the Municipality of Anchorage (www.anchorageraingardens.org).

A rain garden is a shallow depression planted with vegetation, underlain by either local or engineered soils and, in some cases, a subdrain and/or impermeable liner. Rain gardens are intended to temporarily retain and treat storm water runoff through filtration and other mechanisms.

Rain gardens are an extremely versatile LID element and several variations exist. Two variations of rain gardens are discussed in this section: those that have an impermeable liner (lined rain gardens) and those that do not (unlined rain gardens). Lined rain gardens, or those that are underlain by relatively impervious soils, will require subdrain systems. Unlined rain gardens do not necessarily require a subdrain system. Impervious liners are sometimes required to protect groundwater or to protect adjacent foundations. Conceptual profile drawings of both types of rain gardens are presented in Figure 1.

The soil within a rain garden serves as the filtration medium and also provides a rooting area for the rain garden plants. The rain garden plants play an important role in the storm water treatment process, as they encourage infiltration (if the rain garden is not lined) and provide treatment for pollutants, such as total petroleum hydrocarbons, through the process of phytoremediation (PSAT, 2003). In addition to their value as storm water treatment devices, rain gardens can be designed as attractive landscaping features.

Rain gardens are a good choice to treat and/or infiltrate runoff from impervious parking lots, both high– and low–density housing developments and recreation areas. They can also be used in high–density urban applications when the proper precautions are taken to protect adjacent foundations. Rain gardens are capable of removing fine suspended solids as well as other pollutants including copper, lead, zinc, phosphorous, and nitrogen (ARC, 2003).

In order for rain gardens to be effective, they must be designed to meet the geologic, vertical, and horizontal constraints of a site. The process of developing an appropriate rain garden design based on local site constraints is presented in the following sections.

2.1 *The Rain Garden Design Process*

The rain garden design process involves preliminary site evaluation, preliminary and final design, the basic site evaluation considerations discussed in Subsection 1.4, and the following more specific considerations.

Figure 1 – Conceptual Rain Garden Profiles

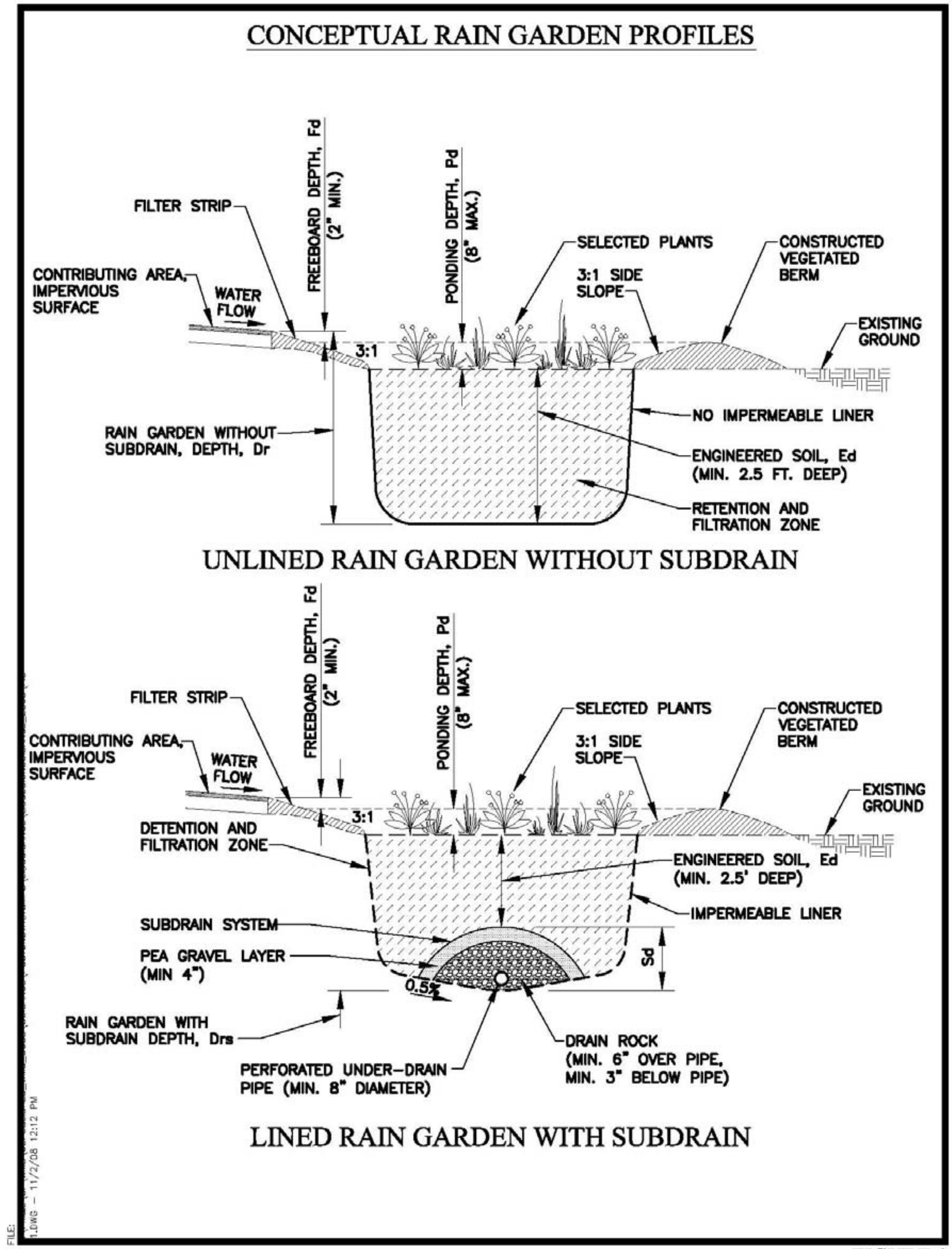


FIGURE 1

2.1.1 Preliminary Site Evaluation – Rain Gardens

The following subsections present the minimum site–specific factors, in addition to those discussed in Subsection 1.4, that are to be considered when evaluating a site for the potential use of a rain garden to treat storm water runoff. The minimum considerations presented below do not include typical engineering considerations such as utility conflicts and are not a substitute for sound engineering judgment.

2.1.1.a Runoff Source

Rain gardens are intended to treat runoff from urban and suburban drainage areas where pollutant loads are related primarily to residential, parking, and road surface runoff. Rain gardens are not appropriate to receive runoff from industrial facilities or areas where runoff is likely to contain industrial pollutants.

2.1.1.b Contributing Area

Because of the difficulty of providing retention and infiltration of runoff from a large area within the relatively small footprint of a rain garden, it is necessary to limit the size of the area contributing runoff. Generally, a single rain garden should not be designed to receive runoff from areas larger than 5 acres (MMC, 2001). It is possible to treat runoff from very large areas if multiple rain gardens or rain gardens in combination with other LID elements are used.

2.1.1.c Slope of Available Area for Rain Garden

Rain gardens are generally difficult to construct on steep sites. This is because the surface of a rain garden must be designed to be relatively level to promote infiltration evenly across the surface of the garden. For this reason, the maximum recommended slope of an area where a rain garden will be placed is 5 % (MDEP, 1997).

2.1.1.d Available Area

A fundamental consideration to make when evaluating a site for use of a rain garden is whether or not there will be adequate space available. A general rule of thumb is that a rain garden will require an area that is approximately 10% of the total contributing area (PSAT, 2003). While the exact area required for a rain garden can only be established through the design process, this generalization is a good starting point to use during the preliminary site evaluation process.

2.1.1.e Down Gradient Slope

It’s important to consider the slope of adjacent properties that are down gradient of the site to limit the possibility of seepage from the subgrade to the ground surface at lower elevations. For this reason, unlined rain gardens should not be used when the average slope of an adjacent down gradient property is 12% or greater (MOA, 2007c). This consideration does not apply to lined rain gardens.

In order to assist designers in the evaluation of sites for use of a rain garden, a checklist of each of the above considerations, as well as those discussed in Subsection 1.4, is provided in Table 2. A site must meet all of the requirements discussed in these subsections to be a candidate for the use of a rain garden.

2.1.2 Preliminary Design Considerations – Rain Gardens

If the preliminary site evaluation indicates that the site is a good candidate for the use of a rain garden to treat storm water, the preliminary design can be carried out to establish the approximate dimensions of the rain garden. Knowing the required dimensions of the rain garden will allow for further evaluation of whether or not there is adequate space within the site to accommodate one. There are several important considerations to be made when performing a preliminary design. Descriptions of the minimum preliminary design considerations are provided in the subsections below.

2.1.2.a Target Treatment Volume

The target treatment volume will ultimately determine the surface area for the rain garden. The target treatment volume is referred to in this manual as the Target Infiltration volume. This volume is a function of the contributing area, runoff coefficient, and target precipitation. The equation relating the three variables is presented below.

$$TIV = \frac{A * P * C}{12}$$ *Equation 2.1*

TIV = Target Infiltration Volume (feet^3)
A = Contributing Area (feet^2), generally less than 5 acres
P = Target Precipitation (inches), 1.1 for the 1–Year, 24–Hour Storm
C = Runoff Coefficient per the DDG

2.1.2.b Ponding Depth and Freeboard

Both the design and function of a rain garden rely on the garden's ability to temporarily store a known depth of water at the surface. The maximum allowable ponding depth for rain gardens is 8 inches (MOA, 2007a). In addition to this ponding depth, a freeboard of 2 inches is also required.

Table 2 – Rain Gardens – Preliminary Site Evaluation Checklist

Site Location:			**Evaluated by:**			
Date:						
Considerations	**Applies to Lined Rain Garden?**	**Applies to Rain Gardens with Subdrains?**	**Requirement/ Recommendation**	**Site Conditions /Notes**	**Pass /Fail**	**Data Source**
Soil Infiltration	Y	N	Measured soil infiltration rate must be between 0.3 and 8 in/hr.			
Proximity to Class A and B Wells	N	Y	Rain garden must be located at least 200 feet from Class A and B wells.			
Proximity to Class C Well	N	Y	Rain garden must be located at least 100 feet from Class C wells.			
Proximity to Surface Waters	N	Y	Rain garden should be located at least 100 feet from surface waters.			
Depth to Seasonal High Groundwater Level	Y	Y	4 feet or more below the top of an unlined rain garden and 2 feet or more below the top of a lined rain garden..			
Depth To Bedrock	N	Y	Bedrock must be 3 foot or more below the bottom of a rain garden.			
Proximity to Building Foundations	N	Y	Rain garden must be located outside of the zone of influence or at least 20 feet from building foundations.			
Proximity to Road Subgrades	N	Y	Rain garden must be located outside of the zone of influence or at least 20 feet from road subgrades.			
Runoff Source	Y	Y	Rain garden is not to receive runoff containing industrial pollutants.			
Contributing Area	Y	Y	The contributing area must be less than 5 acres.			
Available Area Slope	Y	Y	The slope must be less than or equal to 5%.			
Available Area	Y	Y	The area available for treatment must be at least 10% of the total contributing area.			
Down Gradient Slope	N	Y	Average slope of adjacent down gradient property must be less than 12%.			

2.1.2.c Rain Garden Footprint and Geometry

The rain garden footprint is the total area of the rain garden in plan view. The rain garden footprint is a function of the target treatment volume, ponding depth, and side slopes. The recommended side slope for a rain garden is 3:1 (horizontal: vertical). The equation for determining the rain garden footprint is provided below.

$$A_r = \left(\frac{12 * TIV}{P_d} \right) * \left(0.26 * I_e^{-0.53} \right) \qquad \textit{Equation 2.2}$$

A_r = Rain Garden Footprint (feet^2)
TIV = Target Infiltration Volume (feet^3), Equation 2.1
P_d = Depth of Ponded Water (inches), 8 inches maximum
I_e = Infiltration Rate of Engineered Soils (inches/hour)*, 1.0 inches/hour

**Note: For unlined rain gardens without subdrains, substitute variable I_e with I, the design infiltration rate of the native soil.*

Rain gardens are an extremely versatile LID element in terms of plan view geometry. They can take nearly any shape to fit within the site plan. While there is a great deal of freedom associated with specifying the shape of a rain garden, it is important to consider that runoff discharging to the rain garden (typically along the long side of the garden) should be spread evenly across the surface of the garden to promote infiltration across the entire garden surface.

2.1.2.d Depth of Engineered Soils

The engineered soils within a rain garden provide a medium for infiltration and plant growth. In order for the soil to provide adequate treatment, the minimum depth of engineered soils within a rain garden is 2.5 feet (PSAT, 2003).

2.1.2.e Overflow Structure

All rain gardens must incorporate some kind of emergency overflow structure that will safely transmit any storm water to an appropriately sized storm water conveyance system when ponding depths are exceeded. Overflow structures may include perimeter weirs and/or stand pipes. Depending on the nature of the overflow structure, an underground conveyance system may be necessary, which should be determined at the preliminary design stage.

2.1.2.f Subdrain

Some rain gardens will include a subdrain system. Subdrain systems are appropriate when liners are used or when local soil infiltration rates are less than 0.3 inches per hour. For the preliminary design, it is sufficient to consider whether or not a subdrain will be required and to note that the minimum slope of a subdrain is 0.5%. Subdrains may serve as discharge points from overflow structures to limit the amount of buried infrastructure necessary for the rain garden construction.

2.1.2.g Total Depth

The total depth of a rain garden is the depth from the freeboard elevation to the bottom of the excavation. For rain gardens that do not include a subdrain or underground overflow structure within the boundary of the garden, the total depth can be calculated with the following relationship.

$$D_r = \frac{P_d + F_d}{12} + E_d$$ *Equation 2.3*

D_r = Total Depth of Rain Garden without Subdrain (feet)
P_d = Depth of Ponded Water (inches), 8 inches maximum
F_d = Freeboard (inches), 2 inches minimum
E_d = Depth of the Engineered Soils (feet), 2.5 feet maximum

For rain gardens that do include a subdrain or underground overflow structure within the boundary of the rain garden, the total depth can be calculated with the following relationship.

$$D_{rs} = \frac{P_d + F_d}{12} + E_d + S_d + 0.005 * L_r$$ *Equation 2.4*

D_{rs} = Total Depth of Rain Garden with Subdrain (feet)
P_d = Depth of Ponded Water (inches), 8 inches maximum
F_d = Freeboard (inches), 2 inches minimum
E_d = Depth of the Engineered Soils (feet), 2.5 feet maximum
S_d = Depth Required for Subdrain Diameter and Drain Rock (feet), can assume 1.75 during the preliminary design
L_r = Approximate Length of Rain Garden, Along the Axis of the Subdrain (feet)

Note: The equation above is intended to assist designers in the conservative estimation of the depth required for the rain garden at its deepest point. The exact depth is determined during final design.

In order to assist designers in the preliminary design of a rain garden, a blank sample calculation sheet has been developed and is presented as Table 3. The sample calculation sheet includes the preliminary design considerations and equations discussed above and is presented in three steps.

Step 1 – Calculate the Target Infiltration Volume

This step is based on Equation 2.1 presented in Subsection 2.1.2.a above, and requires the independent calculation of the runoff coefficient per the DDG.

Table 3 – Rain Garden Preliminary Design

Site Location: **Evaluated by:**			
Date:			
Step 1: Calculate the Target Infiltration Volume, TIV			**Notes**
Contributing Area, A		(ft^2)	
Target Infiltration Rainfall, P	1.1	(in)	Set Value
Runoff Coefficient, C			Per DDG
TIV = A*P*C/12 =		(ft^3)	Using Equation 2.1
***Step 2: Calculate the Required Rain Garden Footprint Area**			
TIV (from Step 1)		(ft^3)	
Depth of Ponded Water, P_d		(in)	Maximum of 8 inches
Design Infiltration Rate, I_e (or I, see Subsection 2.1.2.c)		(in/hr)	1.0 for engineered soils
$A_r = (TIV*12/P_d)\ (0.26*I_e^{-0.53}) =$		(ft^2)	Using Equation 2.2
Approximate Width, W_r $W_r=A_r/L_r=$		(ft)	
Approximate Length, L_r $L_r=A_r/W_r=$		(ft)	
**** Step 3a: Approximate Rain Garden Depth, without Subdrain**			
P_d (From Step 2)		(in)	
Freeboard Depth, F_d		(in)	Minimum of 2 inches
Depth of Engineered Soils, E_d		(ft)	Minimum of 2.5 feet
$D_r = (P_d+F_d)/12+E_d =$		(ft)	Using Equation 2.3
OR			
***** Step 3b: Approximate Rain Garden Depth, with Subdrain**			
P_d (From Step 2)		(in)	
Freeboard Depth, F_d		(in)	Minimum of 2 inches
Depth of Engineered Soils, E_d		(ft)	Minimum of 2.5 feet
Minimum Subdrain Depth, S_d		(ft)	Assume 1.75 feet
L_r (From Step 3)		(ft)	
$D_{rs} = (P_d+F_d)/12+E_d+S_d+(0.005*L_r) =$		(ft)	Using Equation 2.4
Note: *See Appendix C for guidance on selecting a value for I_e. For unlined rain gardens without subdrains, substitute variable I_e with I, the design infiltration rate for the native soil. **Subdrain and/or underground overflow control system *will not* be used. ***Subdrain and/or underground overflow control system *will be* used.			

Step 2 – Calculate the Rain Garden Footprint
This step involves the application of Equation 2.2 presented in Subsection 2.1.2.c. In this step, the designer must also approximate the length and width values to represent the geometry of the rain garden. The product of these numbers should be approximately equal to the calculated footprint area.

Step 3 – Approximate Garden Depth
There are two equations for approximating the rain garden depth. Step 3a involves the application of Equation 2.3, presented in Subsection 2.1.2.g., to rain gardens that do not include subdrains or underground overflow structures within the rain garden boundaries. Step 3b involves the application of Equation 2.4, presented in Subsection 2.1.2.g., to rain gardens that do include subdrains or underground overflow structures.

Once the site evaluation and preliminary design have been completed, the final design can be conducted.

2.1.3 Final Design – Rain Gardens

In order to develop a final rain garden design based on the results of the preliminary design, there are several basic factors that must be addressed. Addressing these factors requires some basic understanding of engineering and hydraulic principles. At a minimum, each of the factors discussed in the subsections below should be considered during final design.

2.1.3.a Specifying the Engineered Soils

The engineered soils mixture is a critical component in a rain garden design. The recommended soil mixture for rain garden applications is a mixture of 60 to 65% loamy sand mixed with 35 to 40% compost. An alternative recommended soil mixture consists of 20% to 30% topsoil (sandy loam), 50% to 60% coarse sand, and 20% to 30% compost (or peat). The soil mix should be uniform and free of stones, stumps, roots or other similar material greater than 2 inches in diameter. Additional guidance for the specification of engineered soils has been adapted from the Puget Sound Action Team publication titled, *Low Impact Development Technical Guidance Manual for Puget Sound*, and is presented in Appendix C.

2.1.3.b Specifying Rain Garden Plants

Rain garden plants will assist in the storm water treatment process and contribute to the aesthetic value of the garden. It is preferable to use native plants, since they will require less maintenance. If non–native plants are used, they shall not be invasive species (USDA, 2007). There are a wide variety of plants available for use in a rain garden. For large plant orders, coordinate with nurseries early to assure an adequate supply will be available. Generally speaking, the selected plants should be tolerant to a wide variety of moisture and salinity conditions, and should not interfere with utilities in the area. In the selection of rain garden plants, it is also important to consider the potential for attracting wildlife. A list of suitable plants for the Anchorage area is provided in Appendix C. This list is a good starting point for plant materials; see the Additional References for more information.

2.1.3.c Subdrain System Design

Note: Subdrain systems are not always required. However, when site characteristics dictate the use of a subdrain system, they should be designed according to the guidance provided here.

The subdrain in a rain garden performs the important task of removing treated water from the garden soils and transporting it to the storm drain system or outfall. The subdrain system consists of three main components: a subdrain pipe, drain rock, and an aggregate filter blanket. Each of these components is discussed separately below.

The subdrain pipe should be constructed out of slotted polyvinyl chloride (PVC) pipe. The slots should be approximately 0.05 inches wide and 0.25 inches apart. The slots should be arranged in four rows spaced on 45–degree centers, and cover 50% of the circumference of the pipe. The minimum diameter of the drainpipe should be 8 inches and the minimum slope should be 0.5% (PSAT, 2003). The number of subdrains within a rain garden should be adequate to handle the full ponding depth discharge rate of the rain garden according to Manning's equation.

The subdrain pipe is placed on a layer of drain rock that is a minimum of 3 feet wide and 3 inches thick. A 6–inch thick layer of drain rock should be placed above the drainpipe. The recommended gradation for the drain rock is provided below (PSAT, 2003):

Sieve Size	Percent Passing
¾ inch	100
¼ inch	30–60
US No. 8	20–50
US No. 50	3–12
US No. 200	0–1

An aggregate filter blanket diaphragm (pea gravel) will reduce the likelihood of clogging when placed in a 4–inch layer above the drain rock. Pea gravel should be washed and be 0.25 to 0.5 inches in diameter.

2.1.3.d Bottom Grading

In order for the underdrain system to function properly, the bottom of the rain garden must be graded to allow the treated water to flow towards the subdrain. The minimum acceptable bottom slope for providing drainage to the subdrain is 0.5%.

2.1.3.e Specifying the Rain Garden Impermeable Liner

An impermeable liner is not a requirement for all rain gardens. However, liners are required if minimum separation distances from building foundations, road subgrades, or water sources cannot be achieved. Lined rain gardens shall be lined with 30–mil polyethylene plastic with welded joints.

2.1.3.f Overflow Bypass

Overflow bypass structures are important for the proper design of rain gardens. An overflow structure can take many forms. Examples include stand pipes discharging to an underground storm drain network, and broad–crested grassed weirs discharging to grassed ditches. All rain gardens must include some form of overflow bypass sufficient to transmit runoff from a 100–year, 24–hour duration storm event without overtopping the rain garden. Overtopping shall be allowed in cases where discharge due to overtopping is provided a clear, safe, non–destructive path to a conveyance system.

2.1.3.g Pretreatment

Pretreatment for rain gardens can significantly reduce the amount of maintenance associated with sediment deposition. Filter strips, as described in Section 5, are suitable for providing pretreatment. Where site conditions allow, pretreatment devices are recommended for rain gardens receiving runoff from parking areas and other areas known to have high sediment loads.

2.2 *Rain Garden Construction and Maintenance*

2.2.1 Construction Considerations – Rain Gardens

In addition to the minimum construction considerations discussed in Subsection 1.5, consideration should be given to the placement of engineered soils. Onsite mixing and/or placement of engineered soils should not be performed when the soil or ground is saturated. The engineered soils should be placed and graded by excavators and/or backhoes operating adjacent to the rain garden. If machinery must operate in the rain garden for excavation, lightweight, low ground contact pressure equipment should be used. The engineered soils should be placed in 12–inch lifts. Compaction of engineered soils should be allowed to occur through natural settlement over time rather than through mechanical means. To speed settling, each lift can be watered to the saturation point. Water should be applied by either a spraying or sprinkling apparatus (PSAT, 2003).

The minimum considerations presented in this manual do not include some typical engineering considerations such as resolving utility conflicts, and are not a substitute for sound engineering judgment.

2.2.2 Maintenance Considerations – Rain Gardens

In order to function properly over long periods of time, rain gardens must be maintained properly and regularly. The following are general considerations that should be addressed when developing a maintenance agreement as required by the DDG.

2.2.2.a Watering

Because the plants selected for rain garden applications are to be suitable for a wide range of soil moisture conditions, watering will generally not be required after the plants are well established. However, during the first 2 to 3 years, watering will be required to nurture the young plants. Watering may also be required during prolonged dry periods after plants are established (PSAT, 2003)

2.2.2.b Plant Material

Depending on the aesthetic requirements of the rain garden, occasional pruning and removal of dead plants may be necessary. Periodic weeding will be necessary for the first 2 to 3 years, until the plants are well established (PSAT, 2003). As the garden matures, it may be necessary to prune, thin, or split plants to avoid an overgrown appearance and maintain plant health.

2.2.2.c Mulch

If mulch is used in a rain garden, it should be replaced annually if heavy metal deposition or heavy sedimentation is likely (e.g., if runoff comes from parking lots and roads). If heavy metal deposition and/or sedimentation is not a major concern, the mulch should be amended at least once every 2 years to maintain a 2 to 3–inch depth (PSAT, 2003). If mulch is used, allow for additional depth to account for the thickness of the mulch layer.

2.2.2.d Soil

In rain gardens where heavy metals deposition is likely, it is recommended that the engineered soil be removed and replaced once every 20 years. Replacing soil in rain gardens will provide a prolonged service life.

2.2.2.e Inspection and Trash Removal

Rain gardens should be inspected following large rain events. If ponded water persists for more than 24 hours after a rain event, the first six inches of soil may need to be removed and replaced. This task must be performed carefully to limit damage to established plants. Because of the aesthetic value of rain gardens, trash should be regularly removed.

2.2.3 Rain Garden Conceptual Design Example

A conceptual design example for a rain garden is provided in Appendix D of this manual.

3. Infiltration Trenches

An infiltration trench is a rectangular excavation lined with a geotextile filter fabric and filled with coarse stone aggregate. These trenches serve as underground infiltration reservoirs. Storm water runoff directed to these trenches infiltrates into the surrounding soils from the bottom and sides of the trench. Infiltration trenches require pretreatment of storm water runoff to remove large sediments. Pretreatment for infiltration trenches is typically accomplished with the use of filter strips. Trench depths generally range between 2.5 and 10 feet. They can be covered with grating, stone, gabions, sand, or a grassed area with surface inlets. A conceptual drawing of an infiltration trench is provided in Figure 2.

An infiltration trench is a good choice to treat and infiltrate runoff from impervious parking lots, high– and low–density housing developments, and recreation areas. Infiltration trenches can be difficult to use in high–density urban applications due to the amount of area they require for pretreatment, and the potential hazard they pose to adjacent foundations. Infiltration trenches are intended to remove fine suspended solids and other pollutants such as copper, lead, zinc, phosphorous, nitrogen, and bacteria (ARC, 2003).

In order for infiltration trenches to be effective, they must be located in areas where the local soil is appropriate for infiltration and they must be designed accordingly. The process for developing an appropriate infiltration trench design based on local site constraints is presented in the following sections.

3.1 *The Infiltration Trench Design Process*

The infiltration trench design process involves preliminary site evaluation, preliminary and final design, and the basic site evaluation considerations discussed in Subsection 1.4.

Figure 2 – Infiltration Trench Conceptual Drawing

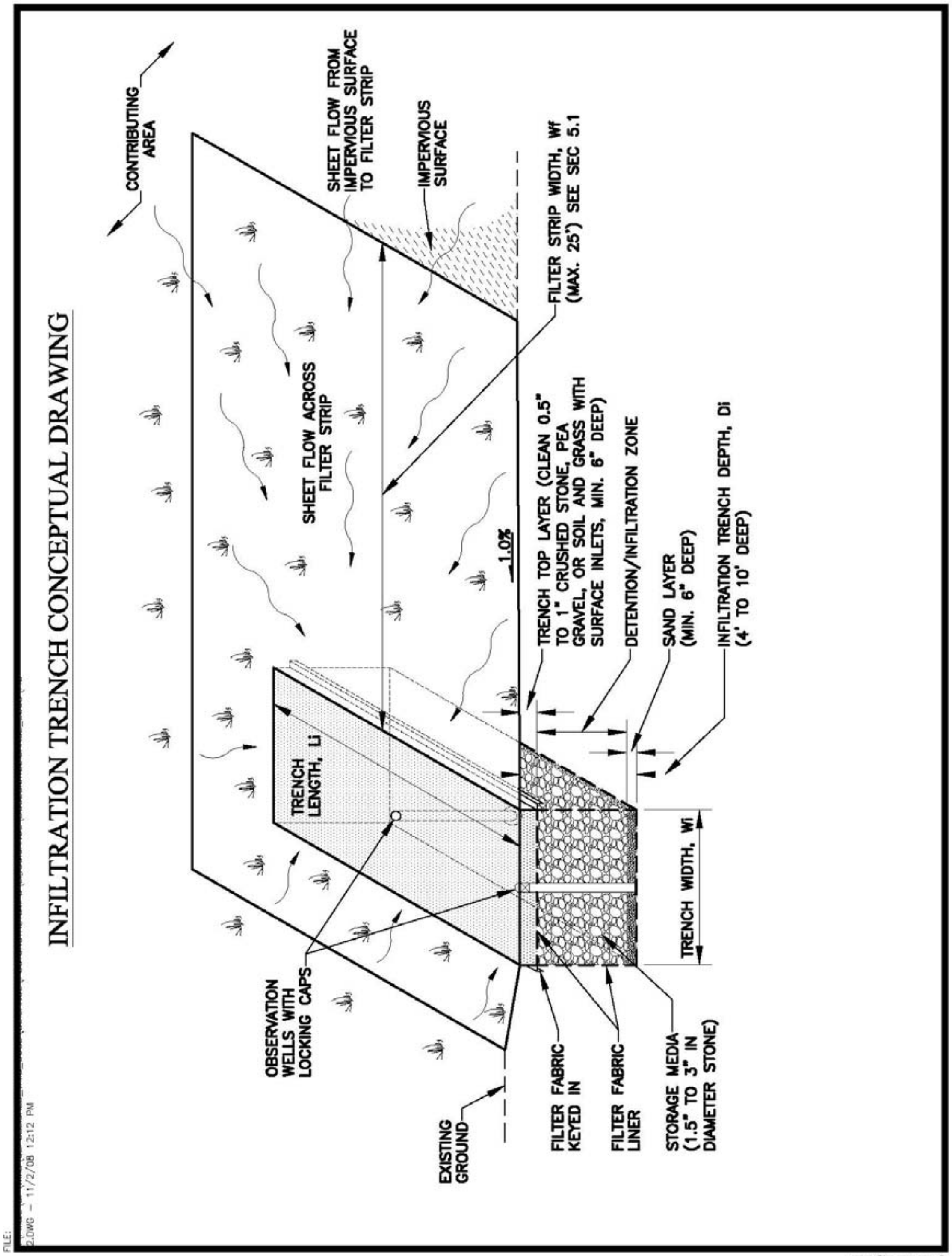


FIGURE 2

3.1.1 Preliminary Site Evaluation – Infiltration Trench

The following subsections present the minimum site–specific factors, in addition to those discussed in Subsection 1.4, that are to be considered when evaluating a site for the potential use of an infiltration trench to treat storm water runoff. The minimum considerations presented below do not include some typical engineering considerations such as resolving utility conflicts, and are not a substitute for sound engineering judgment.

3.1.1.a Runoff Source

Infiltration trenches are intended to treat runoff from urban and suburban drainage areas where pollutant loads are related primarily to parking lot and road surface runoff. Infiltration trenches are not appropriate to receive runoff from industrial facilities where runoff is likely to contain industrial pollutants.

3.1.1.b Contributing Area

In the past, infiltration trenches have been designed to accommodate large drainage areas. However, long term monitoring suggests that large–scale infiltration is not feasible. The main factor being that infiltration of storm water from a large area into a relatively small area does not reflect the natural hydrologic cycle and generally leads to problems such as groundwater mounding, soil clogging, and soil compaction. It is recommended that the contributing area to an infiltration trench be limited to 3 acres or less (modified from MDEP, 1997).

3.1.1.c Slope of Available Area for Infiltration Trench

Infiltration trenches are generally difficult to construct on steep sites because the bottom and top surfaces of the trench must be completely level. The design of filter strips to provide pretreatment to runoff is also more problematic on steep sites. For these reasons, the maximum recommended slope of a site being considered for use of an infiltration trench is 5 % (MDEP, 1997).

3.1.1.d Available Area

Due primarily to pretreatment requirements, the area that is required for an infiltration trench can be as much as 18 to 35% of the total contributing area. The most efficient sites are ones in which the contributing area dimensions are nearly square and the infiltration trench can be constructed along one side of the square. Infiltration trenches can be designed to receive runoff from sites with length to width ratios as low as 3:1 with moderate increases in the percentage of the relative area required for the trench. During the site evaluation process, it can be assumed that the area required for the infiltration trench and filter strip(s) is 35% of the total contributing area.

3.1.1.e Down Gradient Slope

The slope of adjacent properties that are down gradient of the site is important to consider to limit the possibility of seepage from the subgrade to the ground surface at lower elevations. For this reason, infiltration trenches should not be used when the average slope of an adjacent down gradient property is 12% or greater (MOA, 2007c).

In order to assist designers in the evaluation of sites for use of an infiltration trench, a checklist of each of the above considerations, as well as those discussed in Subsection 1.4, is provided in Table 4. A site must meet all of the requirements discussed in these subsections to be a candidate for the use of an infiltration trench.

3.1.2 Preliminary Design Considerations – Infiltration Trench

If the preliminary site evaluation indicates that the site is a good candidate for the use of an infiltration trench to treat storm water, the preliminary design can be carried out to establish the approximate dimensions of the trench and pretreatment area. Knowing the required dimensions of the infiltration trench will allow for further evaluation of whether or not there is adequate space within the site to accommodate the trench and pretreatment area. There are several important considerations to be made when performing a preliminary design of an infiltration trench. Descriptions of the recommended preliminary design considerations are provided in the subsections below.

Table 4 – Infiltration Trench – Preliminary Site Evaluation Checklist

Site Location:		**Evaluated by:**		
Date:				
Considerations	**Requirement/Recommendation**	**Site Conditions/Notes**	**Pass/Fail**	**Data Source**
Soil Infiltration	Measured soil infiltration rate must be between 0.3 and 8 in/hr.			
Proximity to Class A and B Wells	Trench must be located at least 200 feet from Class A and B wells.			
Proximity to Class C Well	Trench must be located at least 100 feet from Class C wells.			
Proximity to Surface Waters	Trench should be located at least 100 feet from surface waters.			
Depth to Seasonal High Groundwater Level	Must be 4 feet or more below the bottom of the trench.			
Depth To Bedrock	Bedrock must be 3 feet or more below the bottom of the trench.			
Proximity to Building Foundations	Trench must be located outside of the zone of influence or at least 20 feet from building foundations.			
Proximity to Road Subgrades	Trench must be located at least 20 feet from road subgrades.			
Runoff Source	Infiltration trench is not to receive runoff containing industrial pollutants.			
Contributing Area	The contributing area must be less than 3 acres.			
Available Area Slope	Available area slope must be less than or equal to 5%.			
Available Area	The area available for treatment must be at least 18% of the total catchment area.			
Down Gradient Slope	Average slope of adjacent down gradient property must be less than 12%.			

3.1.2.a Target Treatment Volume

The target treatment volume will ultimately determine the area of the infiltration trench. The target treatment volume is referred to in this manual as the Target Infiltration volume. This volume is a function of the contributing area, runoff coefficient, and target precipitation. The equation relating the three variables, presented for the first time in Subsection 2.1.2.a, is presented again below.

$$TIV = \frac{A * P * C}{12} \qquad \textit{Equation 2.1}$$

TIV = Target Infiltration Volume (feet^3)
A = Contributing Area (feet^2)
P = Target Precipitation (inches), 1.1 for the 1–Year, 24–Hour Storm
C = Runoff Coefficient per the DDG

3.1.2.b Void Ratio

The function of an infiltration trench is reliant on not only the infiltration rate of the surrounding soil but also on the trench's ability to temporarily retain water. The storm water is retained within the void spaces of the storage media. The ratio of the volume of the space between individual particles of the storage media over the volume of the storage media particles is known as the void ratio. Infiltration trench storage media should consist of clean aggregate ranging from 1.5 to 3 inches in diameter. For the sake of calculation in this manual, assume a void ratio of 0.4.

3.1.2.c Retention Time

The retention time associated with an infiltration trench is the amount of time it takes for the full trench to discharge to the surrounding soil. In order to provide adequate treatment, the acceptable range for retention time is 24 to 48 hours.

3.1.2.d Trench Depth

The trench depth is the depth of the trench from the top surface to the bottom of the excavated area. Trench depth is a function of the design infiltration rate, the storage media void space, and the retention time. The trench depth should be between 4 and 10 feet. A minimum depth of 4 feet allows for the bottom of the trench to be at or below the frost line. Shallower depths may be permitted in non–frost susceptible soils. The equation for determining trench depth is provided below (modified from MOA, 2004).

$$D_i = \frac{I * t}{n_s * 12} + 1 \qquad \textit{Equation 3.1}$$

D_i = Trench Depth (feet), must be 4 to 10 feet
I = Design Infiltration Rate (inches/hour), between 0.3 and 1 inch/hour
t = Retention Time (hours), 24 to 48 hours
n_s = Storage Media Void Ratio, 0.4 typical for 1.5 to 3–inch stones

The additional one foot added to the equation above is to allow for the use of a 6–inch layer of sand in the bottom of the trench and a 6–inch top layer. The sand acts to distribute flow and to reduce localized compaction when placing the storage media during construction.

3.1.2.e Trench Footprint

The trench footprint is the plan view area of the trench and is a function of the design infiltration rate, the retention time, and the target infiltration volume. The equation for determining the trench footprint is provided below (modified from MOA, 2004).

$$A_i = \frac{TIV * 0.66}{n_s * (D_i - 1)} \qquad \textit{Equation 3.2}$$

A_i = Trench Footprint (feet^2)
TIV = Target Infiltration Volume (feet^3)
n_s = Storage Media Void Ratio, 0.4 typical for 3–inch stones
D_i = Trench Depth (feet), between 4 and 10 feet

3.1.2.f Trench Width

The width of a trench can be adjusted to meet site constraints as long as the necessary footprint area is maintained. The minimum suggested length to width ratio to be applied to an infiltration trench design is 3:1. The maximum allowable trench width, parallel to flow, is 25 feet.

3.1.3 Pretreatment

Infiltration trenches require pretreatment to remove large particulates. Grass filter strips are generally used to provide pretreatment for runoff entering an infiltration trench although other pretreatment devices may be used including vegetated swales, ponds, etc. At the preliminary design stage, the designer may assume a 20–foot filter strip width. For additional information on sizing filter strips for pretreatment, refer to Subsection 5.1 of this manual.

In order to assist designers in the preliminary design on an infiltration trench, a sample calculation sheet has been developed and is included in Table 5. The calculation sheet covers the above considerations and equations in six steps.

<u>Step 1 – Calculate the Target Infiltration Volume</u>
This step is based on Equation 2.1 presented in Subsection 3.1.2.a above, and requires the independent calculation of the runoff coefficient per the DDG.

<u>Step 2 – Calculate the Depth of the Trench</u>
This step is based on Equation 3.1 presented in Subsection 3.1.2.d above. The depth can be adjusted by adjusting the drawdown time. However, it should be noted that reductions in depth will result in increases in area.

<u>Step 3 – Calculate the Footprint of the Trench</u>
This step is based on Equation 3.2 presented in Subsection 3.1.2.e above.

<u>Step 4 – Establish the Trench Length and Width</u>
In this step, the designer may choose to set either the trench length or width to meet particular site requirements. Note that the maximum allowable trench width is 25 feet and the maximum recommended length to width ratio is 3:1.

<u>Step 5 – Account for Pretreatment</u>
This step involves determining the total width of the infiltration trench and associated filter strips. Note that if the site only drains to one side of an infiltration trench, only a single filter strip on that side is necessary.

<u>Step 6 – Required Length and Width for Trench and Filter Strip</u>
This step involves summarizing the preliminary design values for length and width established in Steps 4 and 5.

Table 5 – Infiltration Trench Preliminary Design

Site Location:	**Evaluated by:**		
Date:			
Step 1: Calculate the Target Infiltration Volume			**Notes**
Contributing Area, A		(ft^2)	
Target Infiltration Rainfall, P		(in)	Set Value
Runoff Coefficient, C			Per DDG
TIV = A*P*C/12 =		(ft^3)	Using Equation 2.1
Step 2: Calculate the Depth of the Trench			Must be between 4 and 10 feet
Void Ratio, n_s			0.4 is Typical of 1.5 to 3 in. Stone
Design Infiltration Rate, I		(in/hr)	Based on site investigation (Subsection 1.4.1 and DDG)
Retention Time, t		(hr)	Must be 24 to 48 hours
$D_i = (I*t)/(n_s*12) + 1 =$		(ft)	Using Equation 3.1
Step 3: Calculate the Footprint of the Trench			
TIV (from Step 1)		(ft^3)	
n_s (from Step 2)			
D_i (from Step 2)		(ft)	
$A_i = (TIV *0.66)/(n_s*(D_i – 1)) =$		(ft^2)	Using Equation 3.2
Step 4: Establish the Trench Length and Width			Minimum Recommended Ratio is 3L:1W
Set Trench Length, L_i		(ft)	
Or			
Set Trench Width, W_i		(ft)	Maximum Width is 25 feet
Then Calculate Either			
$W_i=A_i/L_i$		(ft)	Maximum Width is 25 feet
Or			
$L_i=A_i/W_i$		(ft)	
Record Final L_i and W_i Values			
$L_i=$		(ft)	
$W_i=$		(ft)	
Step 5: Account for Pretreatment			
Filter Strip Width, W_f		(ft)	Minimum Recommended Width is 20 feet
If Receiving Flow From Both Sides			
Total Width (W_{if1}), $W_{if1} = W_i + 2*W_f =$		(ft)	
Or, If Receiving Flow From One Side			
Total Width (W_{if2}), $W_{if2}=W_i +W_f =$		(ft)	
Step 6: Required Length and Width for Trench and Filter Strip			
L_i (from Step 4) =		(ft)	
Appropriate Total Width (from Step 5) =		(ft)	

Once the site evaluation and preliminary design have been completed, the final design can be performed.

3.1.4 Final Design Considerations – Infiltration Trench

In order to develop a final infiltration trench design based on the results of the preliminary design, there are several basic factors that must be addressed. Addressing these factors requires some basic understanding of engineering and hydraulic principles. At a minimum, each of the factors discussed in the subsections below should be considered during final design.

3.1.4.a Filter Fabric

Filter fabric selection and placement are important to both the effectiveness and the service life of an infiltration trench. Filter fabric should be selected that matches the infiltrative capacity of the soil in the trench to prevent clogging and piping. The fabric should be placed so that it lines the bottom and sides of the trench. Overlap between separate pieces of fabric should be a minimum of one foot. Filter fabric should also be placed below the top layer of the infiltration trench to reduce maintenance costs, since the top fabric can be cleaned or replaced much more easily than the fabric lining the bottom and sides when fine particles clog the trench.

3.1.4.b Overflow Structure

Overflow structures are important for the proper design of infiltration trenches. An overflow structure can take many forms. Examples include stand pipes discharging to an underground storm drain network, and broad crested weirs discharging to grassed ditches. No matter what kind of overflow structure is selected, it must be capable of safely transmitting runoff from the 100–year, 24–hour duration storm event so that the infiltration trench does not overtop. Overtopping may be allowed in cases where discharge due to overtopping is provided an unobstructed, safe, and non–destructive path to a conveyance system.

Any portion of an overflow structure that lies within the subgrade of an infiltration trench will reduce the volume of storm water that can be held by the trench. The trench footprint must be adjusted accordingly to account for the lost storage volume.

3.1.4.c Top Layer

Infiltration trenches can be covered with a variety of different materials. The top layer is intended to provide cover for the first layer of filter fabric and to provide a level surface that can be easily traversed. An additional benefit of the top layer is improvement of aesthetics. The top layer of an infiltration trench should consist of a minimum of 6 inches of one of the following: clean 0.5 to 1–inch crushed stone, pea gravel, or soil and grass. Note, that if a grass cover is used, sufficient surface inlets into the infiltration trench must also be installed. Due to the need for periodic maintenance, infiltration trenches should not be covered with concrete or asphalt.

3.1.4.d Bottom Layer

The bottom layer of an infiltration trench consists of 6 inches of clean sand. The purpose of the bottom layer is to evenly distribute flows along the bottom of the trench and to protect the underlying soil from localized compaction during placement of the storage media.

3.1.4.e Grading

Site grading is one of the most critical factors in the final design of an infiltration trench. The site must be graded so that runoff is directed to the infiltration trench evenly across the surface of the filter strips. The site must also be graded so that both the top surface and the bottom of the infiltration trench are completely level.

3.1.4.f Observation Well

An observation well is to be installed in each infiltration trench. An additional observation well shall be installed for every 50 linear feet of infiltration trench. Observation wells allow drawdown times to be monitored within the trench, and will allow maintenance crews to identify when the trench has become clogged and is in need of repair. The wells should be placed to the full depth of the trench and be secured to a footing plate. The observation well should be a minimum of 6 inches in diameter and have a waterproof locking cap at the surface.

The perforated portion of the observation well shall be between the top and bottom layers of filter fabric. Where the observation well passes through the top layer of filter fabric, the filter fabric shall be sealed around the un–perforated section of the well. This will limit the intrusion of sediments collected by the upper filter fabric into the lower portion of the well, where they are more difficult to remove.

The above list does not include every possible final design consideration. However, for most infiltration trench designs, each of the above design considerations will be necessary. Additional engineering considerations, such as the depth and location of utilities within and adjacent to the site, will be required depending on the site specific conditions.

3.2 *Infiltration Trench Construction and Maintenance*

3.2.1 Construction Considerations – Infiltration Trench

In addition to the minimum general considerations, discussed in Subsection 1.5, the construction of an infiltration trench requires care in the placement of the storage media. Storage media should be placed without causing compaction of the subsoil. This can be accomplished by placing the storage media in 6–inch lifts. The storage media should not be compacted.

The minimum considerations presented in this manual do not include some typical engineering considerations such as resolving utility conflicts, and are not a substitute for sound engineering judgment.

3.2.2 Maintenance Considerations – Infiltration Trench

In order to function properly over long periods of time, infiltration trenches must be maintained properly and regularly. The following are general considerations that should be addressed when developing a maintenance agreement as required by the DDG.

3.2.2.a Watering and Weeding

If a top layer of grass (with inlets) is used, periodic watering will be required in the first year to help the grass become established. Watering may also be required during prolonged dry periods. Weeding should be performed as necessary to maintain a healthy grassed top layer.

3.2.2.b Filter Fabric

The top layer of filter fabric in an infiltration trench will require periodic cleaning or replacement. The observation well(s) can be used to establish which portion of the filter fabric is in need of replacement. If standing water persists in the infiltration trench longer than the designed retention time, the observation well(s) should be checked. If the observation wells are empty, then the top layer of filter fabric will need to be cleaned or replaced to remove accumulated sediments. If the observation wells are full of standing water, then the storage media will need to be removed and washed, and the layer of filter fabric along the trench sides and bottom will need to be cleaned or replaced.

3.2.2.c Routine Post–Storm Inspection

Infiltration trenches and filter strips should be inspected after large rain events. The filter strips and the top layer of the infiltration trench should be inspected for evidence of erosion (which is unlikely in properly designed systems). Any visible trash accumulated on top of the infiltration trench or on the filter strip should be removed.

3.2.3 Infiltration Trench Conceptual Design Example

A conceptual design example for an infiltration trench is provided in Appendix E of this manual.

4. Soak–Away Pits

A soak–away pit is a small excavated, subterranean chamber lined with filter fabric on all sides, and filled with coarse stone aggregate that serves as an underground infiltration reservoir. Storm water runoff directed to soak–away pits infiltrates into the surrounding soils through the bottom and, in some cases, the sides of the pit. Soak–away pit depths generally range between 3 and 10 feet, and widths generally range from 4 to 8 feet. Soak–away pits are intended to remove fine suspended solids and other pollutants such as copper, lead, zinc, phosphorous, nitrogen, and bacteria (ARC, 2003). A soak–away pit is a good choice to treat and infiltrate runoff from rooftop downspouts. A section through a conceptual soak–away pit is presented in Figure 3.

In order for soak–away pits to be effective, they must be located in areas where the local soil is appropriate for infiltration. The process of developing an appropriate soak–away pit design based on local site constraints is presented in the following sections.

4.1 *The Soak–Away Pit Design Process*

The soak–away pit design process involves preliminary site evaluation, preliminary and final design, and the basic site evaluation considerations discussed in Subsection 1.4.

Figure 3 – Soak–Away Pit Conceptual Section

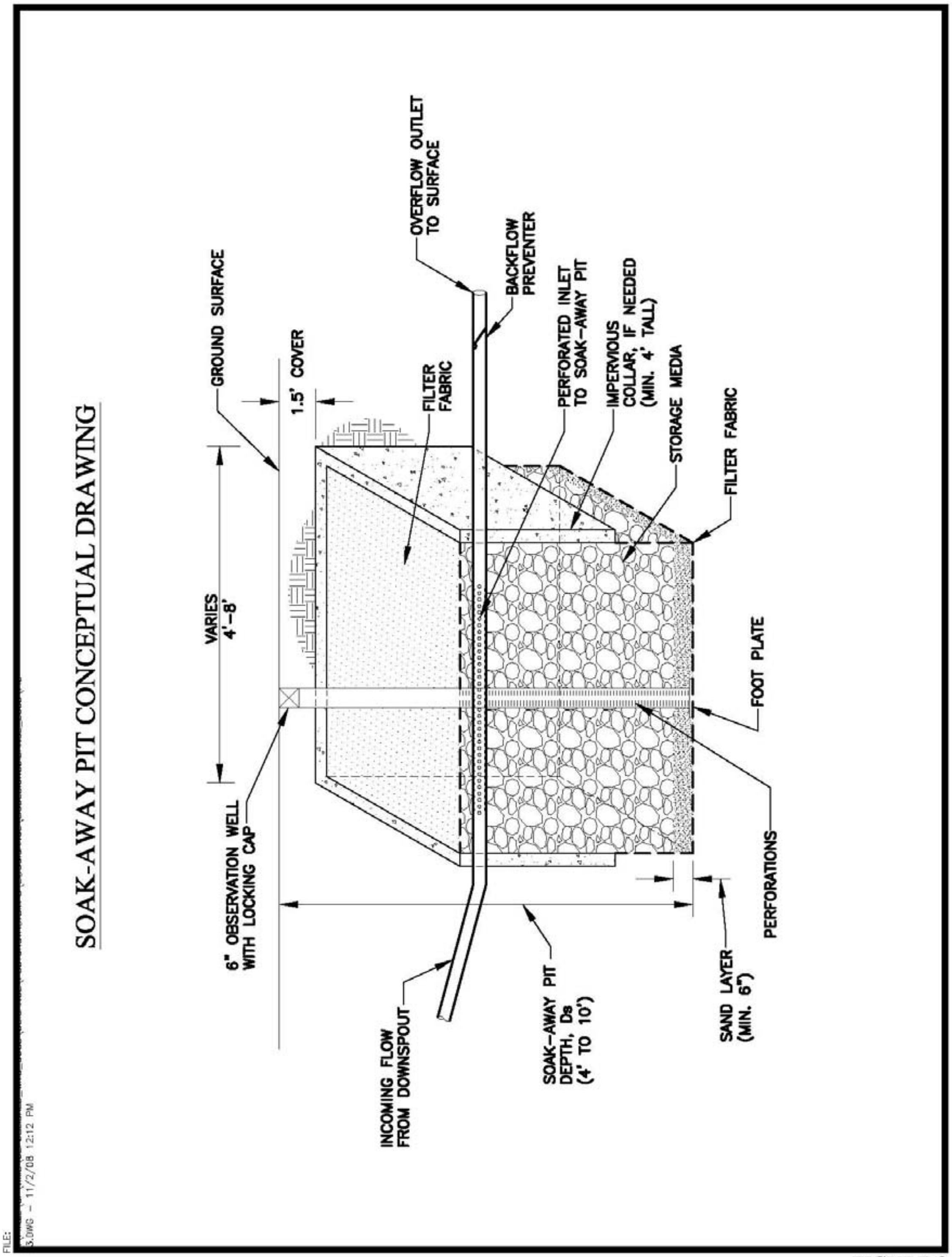


FIGURE 3

Preliminary Site Evaluation – Soak–Away Pits

The following subsections present the minimum site–specific factors, in addition to those discussed in Subsection 1.4, that are to be considered when evaluating a site for the potential use of a soak–away pit to treat storm water runoff. The minimum considerations presented below do not include some typical engineering considerations such as resolving utility conflicts, and are not a substitute for sound engineering judgment.

4.1.1.a Runoff Source

Soak–away pits are normally intended to treat runoff from rooftops. Pretreatment devices are not required due to the low sediment concentrations in rooftop runoff. Soak–away pits may be used in some instances to treat runoff from sources other than rooftops if appropriate pretreatment is used.

4.1.1.b Contributing Area

Soak–away pits are relatively small LID elements when compared to rain gardens and infiltration trenches. Consequently, the maximum allowable contributing area is also relatively small and should not exceed 1,900 feet^2.

4.1.1.c Slope of Available Area for Soak–Away Pit

Unlike rain gardens and infiltration trenches, soak–away pits can be constructed on relatively steep sites. Soak–away pits may be constructed on sites with slopes up to 12%.

4.1.1.d Available Area

Soak–away pits require relatively little area. However, deep soak–away pits will require a considerable area to construct if it is necessary to lay back the walls of the excavation during construction. Generally, a soak–away pit will occupy 4% of the total contributing area, after construction. While the exact area required for a soak–away pit can only be established through the design process, an estimate of 4% of the total contributing area is a good starting point to use during the site evaluation process.

4.1.1.e Down Gradient Slope

The slope of adjacent properties that are down gradient of the site is important to consider to limit the possibility of seepage from the subgrade to the ground surface at lower elevations. For this reason, soak–away pits should not be used when the average slope of an adjacent down gradient property is greater than 12% (MOA, 2007c).

4.1.1.f Separation Distances from Adjacent Soak–Away Pits

If multiple soak–away pits are to be used to treat a large area, it is important to consider the separation distance between individual soak–away pits. The minimum recommended separation distance between soak–away pits is 20 feet. This consideration does not apply to soak–away pits that incorporate an impervious collar.

In order to assist designers in the evaluation of sites for use of a soak–away pit, a checklist of each of the above considerations, as well as those discussed in Subsection 1.4, is provided in Table 6. A site must meet all of the requirements discussed in these subsections to be a candidate for the use of a soak–away pit.

4.1.2 Preliminary Design Considerations – Soak–Away Pits

If the preliminary site evaluation indicates that the site is a suitable candidate for the use of a soak–away pit to treat storm water runoff, the preliminary design can be carried out to establish the approximate dimensions of the pit. Knowing the required dimensions of the soak–away pit will allow for further evaluation of whether or not there is adequate space within the site to accommodate the pit. There are several important considerations to be made when performing the preliminary design of a soak–away pit. Descriptions of the recommended preliminary design considerations are provided in the subsections below.

4.1.2.a Target Treatment Volume

One of the most fundamental considerations in the design of a soak–away pit is the volume of runoff that the pit will need to accommodate. The target treatment volume is referred to in this manual as the Target Infiltration volume. This volume is a function of the contributing area, runoff coefficient, and target precipitation. The equation relating the three variables, presented for the first time in Subsection 2.1.2.a, is presented again below.

$$TIV = \frac{A * P * C}{12}$$ *Equation 2.1*

TIV = Target Infiltration Volume ($feet^3$)
A = Contributing Area ($feet^2$)
P = Target Precipitation (inches), 1.1 for the 1–Year, 24–Hour Storm
C = Runoff Coefficient per the DDG

4.1.2.b Void Ratio

The main function of a soak–away pit is infiltration, which is not only reliant on the design infiltration rate for the surrounding soil but also on the pit's ability to temporarily retain water. The storm water is retained within the void spaces of the storage media. The ratio of the volume of the space between individual particles of the storage media over the volume of the storage media particles is known as the void ratio. Soak–away pit storage media should consist of clean aggregate ranging from 1.5 to 3 inches in diameter. For the sake of calculation in this manual, assume a void ratio of 0.4.

Table 6 – Example Soak–Away Pit – Preliminary Site Evaluation Checklist

Site Location:		**Evaluated by:**		
Date:				
Considerations	**Requirement/Recommendation**	**Site Conditions/Notes**	**Pass/Fail**	**Data Source**
Soil Infiltration	Measured soil infiltration rate below the soak–away pit must be between 0.3 and 8 inches/hour.			
Proximity to Class A and B Wells	The soak–away pit must be separated at least 200 feet from Class A and B wells.			
Proximity to Class C Wells	The soak–away pit must be separated at least 100 feet from Class C wells.			
Proximity to Surface Waters	The soak–away pit should be separated at least 100 feet from surface waters.			
Depth to Seasonal High Groundwater Level	Groundwater must be 4 feet or more below the bottom of the pit.			
Depth To Bedrock	Bedrock must be 3 feet or more below the bottom of the pit.			
Proximity to Building Foundations*	The pit must be located outside of the zone of influence or at least 20 feet from building foundations.			
Proximity to Road Subgrades*	The pit must be located outside of the zone of influence or at least 20 feet from road subgrades.			
Runoff Source	Soak–away pit is not to receive runoff containing industrial pollutants.			
Contributing Area	The contributing area must be less than 1,900 feet^2.			
Slope of Available Area	The available area slope must be less than or equal to 12%.			
Available Area	The area available for treatment must be at least 4% of the total catchment area.			
Down Gradient Slope	Average slope of adjacent down gradient property must be less than 12%.			
Horizontal Separation Distance from Adjacent Soak–Away Pits*	Soak–away pits must be separated by a distance of 20 feet.			
Note: * These criteria do not apply to soak–away pits with impervious collars.				

4.1.2.c Retention Time

The retention time associated with a soak–away pit is the amount of time it takes for the full pit to discharge to the surrounding soil. To provide adequate treatment, the acceptable range for retention time is 24 to 72 hours. The retention time may be adjusted to adjust the required pit depth (see Subsection 4.1.2.d).

4.1.2.d Soak–Away Pit Depth

Pit depth is the depth of the pit from the surface to the bottom of the excavation. Pit depth is a function of the design infiltration rate, the storage media void ratio, and the retention time. Soak–away pit depth should not fall outside the range of 4 to 10 feet. A minimum dept of 4 feet allows for the bottom of the trench to be at or below the frost line. Shallower depths may be permitted in non–frost susceptible soils. The equation for determining pit depth is provided below (modified from MOA, 2004).

$$D_s = \frac{I * t}{n_s * 12} + 2$$ *Equation 4.1*

D_s = Soak–Away Pit Depth (feet), from 4 to 10 feet
I = Design Infiltration Rate (inches/hour), between 0.3 and 1 inches/hour
t = Retention Time (hours), from 24 to 72 hours
n_s = Storage Media Void Ratio, 0.4 typical for 1.5 to 3–inch stones

The additional two feet added to the equation above is to allow for the use of a 6–inch layer of sand in the bottom of the pit and a 1.5–foot layer over the top of the pit for cover. The sand in the bottom of the pit acts to distribute flow and to reduce localized compaction during the placement of the storage media during construction.

4.1.2.e Soak–Away Pit Footprint

The pit footprint is the plan view area of the pit, and is a function of the design infiltration rate, the retention time, and the target infiltration volume. The maximum allowable pit footprint area is 64 feet^2. The equation for determining the pit footprint is provided below (modified from MOA, 2004).

$$A_s = \frac{TIV * 0.66}{n_s * (D_s - 2)}$$ *Equation 4.2*

A_s = Soak–Away Pit Footprint (feet^2), 64 feet^{2}maximum
TIV = Target Infiltration Volume (feet^3)
n_s = Storage Media Void Ratio, 0.4 typical for 1.5 to 3–inch stones

D_s = Soak–Away Pit Depth (feet), from 4 to 10 feet

In order to assist designers in the preliminary design of a soak–away pit, a sample calculation sheet has been developed and is included in Table 7. The calculation sheet covers the above considerations and equations in three steps.

Step 1 – Calculate the Target Infiltration Volume
This step is based on Equation 2.1 presented for TIV in Subsection 4.1.2.a above, and requires the independent calculation of the runoff coefficient per the DDG.

Step 2 – Calculate the Depth of the Pit
This step is based on Equation 4.1 presented in Subsection 4.1.2.d above. The depth of the pit can be adjusted by changing the retention time to a value between 24 and 72 hours.

Step 3 – Calculate the Soak–Away Pit Footprint
This step is based on Equation 4.2 presented in Subsection 4.1.2.e above. The footprint of the pit must be limited to 64 square feet. If a trench configuration cannot be established to accommodate this requirement, then alternative treatment options, such as infiltration trenches, should be explored.

Once the site evaluation and preliminary design have been completed, the final design can be performed.

4.1.3 Final Design Considerations – Soak–Away Pits

In order to develop a final soak–away pit design based on the results of the preliminary design, there are several basic factors that must be addressed. Addressing these factors requires some basic understanding of engineering and hydraulic principles. At a minimum, each of the factors discussed in the subsections below should be considered during final design.

4.1.3.a Inlet

Runoff enters a soak–away pit through a perforated pipe running through the top of the storage media. The perforated pipe must be at least 4 inches in diameter or have a cross–sectional area no smaller than the cross–sectional area of the connected rooftop downspout. The size and spacing of perforations should be adequate to accommodate the peak runoff from the Target Infiltration design storm. If suitable prefabricated materials cannot be obtained, perforations can be created by drilling holes with a diameter no more than 1/4 the diameter of the inlet pipe.

Table 7 – Example Soak–Away Pit Preliminary Design

Site Location:	**Evaluated by:**		
Date:			
Step 1: Calculate the Target Infiltration Volume			**Notes**
Contributing Area, A		(ft^2)	Should Be Less Than 1,900 $feet^2$
Target Infiltration Rainfall, P		(in)	1–Year, 24–Hour Rainfall Depth
Runoff Coefficient, C			Calculated per DDG
TIV = A*P*C/12 =		(ft^3)	Using Equation 2.1
Step 2: Calculate the Depth of the Pit			Must be between 4 and 10 feet
Void Ratio, n_s			0.4 is typical of 1.5 to 3 in stone
Design Infiltration Rate, I		(in/hr)	Based on site investigation (Subsection 1.4.1 and DDG)
Retention Time, t		(hr)	Must be between 24 to 72 hours
$D_s = (I*t)/(n_s*12) + 2 =$		(ft)	Using Equation 4.1
Step 3: Calculate the Soak–Away Pit Footprint			Must be less than 64 $feet^2$
TIV (from Step 1)		(ft^3)	
n_s (from Step 2)			
D_s (from Step 2)		(ft)	
$A_s = (TIV*0.66)/(n_s*(D_s–2)) =$		(ft^2)	Using Equation 4.2

4.1.3.b Impervious Collar

Soak–away pits placed within the zone of influence or closer than 20 feet to road subgrades and building foundations will require the use of an impervious collar. One choice for an impervious collar is a prefabricated open–ended casing such as those commonly used in manhole construction. These prefabricated structures are commonly available in circular and square geometries and are typically constructed of reinforced concrete. Impervious collars are to be installed to a depth of 4 feet below the top of the storage media.

4.1.3.c Filter Fabric

Filter fabric selection and placement are important to both the effectiveness and the service life of a soak–away pit. Filter fabric that is similar to the infiltrative capacity of the soil surrounding the pit shall be selected to prevent clogging and piping. The fabric shall be placed on all sides of the soak–away pit, with a minimum of one foot of overlap between separate pieces of fabric.

4.1.3.d Overflow Structures

Overflow structures are important for the proper design of soak–away pits. Systems should incorporate an overflow structure into the roof downspout system such that the roof downspout will drain to the surface when the soak–away pit is completely full of storm water. Soak–away pits may incorporate an overland flow path to a storm water collection system such that when the pit is full, flows will be directed to the collection system. All overflow structures shall be designed to safely convey runoff from the 100–year, 24–hour storm event.

4.1.3.e Bottom Layer

The bottom layer of a soak–away pit consists of 6 inches of clean sand. The purpose of the bottom layer is to evenly distribute flows along the bottom of the trench and to protect the underlying soil from localized compaction during placement of the storage media.

4.1.3.f Grading

The bottom of the soak–away pit must be completely level to promote infiltration evenly across the bottom.

4.1.3.g Observation Wells

An observation well is to be installed in each soak–away pit. The well allows drawdown times to be monitored within the pit. The observation well will allow maintenance crews to identify when the pit has become clogged and is in need of repair. Wells should be placed to the full depth of the soak–away pit, and be secured to a footing plate. The observation well should be a minimum of 6 inches in diameter, and have a waterproof locking cap at the surface.

The perforated portion of the observation well should be restricted to the area within the storage media. Where the observation well passes through the filter fabric lining the top of the soak–away pit, the fabric should be sealed around the un–perforated section of the observation well.

The above list of final design considerations does not include every possible final design consideration. However, for most soak–away pit designs, each of the above design considerations will be necessary. Additional engineering considerations, such as the depth and location of utilities within and adjacent to the site, will be required depending on the site–specific conditions.

4.2 *Soak–Away Pit Construction and Maintenance*

4.2.1 Construction Considerations – Soak–Away Pits

In addition to the minimum general considerations discussed in Subsection 1.5, the construction of a soak–away pit requires care in the placement of the storage media. Storage media should be placed without causing compaction of the subsoil. This can be accomplished by placing the storage media in 6–inch lifts. The storage media should not be compacted.

The minimum considerations presented in this manual do not include some typical engineering considerations such as resolving utility conflicts, and are not a substitute for sound engineering judgment.

4.2.2 Maintenance Considerations – Soak–Away Pits

In order to function properly over long periods of time, soak–away pits must be maintained properly and regularly. The following are general considerations that should be addressed when developing a maintenance agreement, as required by the DDG.

4.2.2.a Routine Post–Storm Inspection

Soak–away pits should be inspected after large rain events. Soak–away pits can be inspected via the observation well. Standing water should not persist in the soak–away pit any longer than the designed retention time. Any accumulated trash should be removed.

4.2.2.b Filter Fabric

Standing water can indicate that the storage media needs to be removed and cleaned, or that the filter fabric needs to be replaced or cleaned. This is uncommon, since soak–away pits receive rooftop runoff with low sediment concentrations. If standing water persists in a soak–away pit, the storage media needs to be removed and cleaned, and the layer of filter fabric needs to be cleaned or replaced.

4.2.3 Soak–Away Pit Conceptual Design Example

A conceptual design example for a soak–away pit is provided in Appendix F of this manual.

5. Filter Strips

Filter strips are gently sloped, vegetated areas designed to decelerate and filter sheet flow runoff. Existing areas of dense, healthy vegetation that are capable of dispersing runoff and have experienced relatively little site disturbance or soil compaction often provide the most desirable areas for use as filter strips. These LID elements primarily treat total suspended solids (TSS), but they can also reduce concentrations of hydrocarbons, heavy metals, and nutrients. Filter strips remove pollutants via sedimentation, filtration, absorption, infiltration, biological uptake, and microbial activity. Depending on site characteristics such as soil type, vegetative cover, slope, and available area, filter strips can provide a modest reduction in runoff volume due to infiltration. In addition to their value as storm water treatment devices, filter strips can serve as attractive landscaping features that may incorporate a variety of trees, shrubs, and native vegetation. The simplest and often most effective filter strips are those that incorporate undisturbed existing vegetation.

The size and character of contributing drainage areas largely dictate the size and location of filter strips, since filter strips perform effectively only under sheet flow conditions, and flows tend to concentrate and have higher velocities over large or impervious drainage areas. A conceptual drawing of a filter strip is presented in Figure 4.

The advantages of filter strips include removal of sediment and insoluble contaminants from runoff, and increased infiltration of soluble nutrients and pesticides. The tall, dense vegetation of filter strips can provide a visual barrier between roads and recreation sites. Filter strips work particularly well in residential areas, providing open spaces for recreation and maintaining riparian zones along streams, which can reduce erosion and enhance animal habitats and aquatic life. In general, filter strips are simple and inexpensive to install, and have relatively few maintenance requirements. In order for filter strips to be effective, they must be properly graded to limit erosive velocities.

Figure 4 – Filter Strip Conceptual Plan and Profile

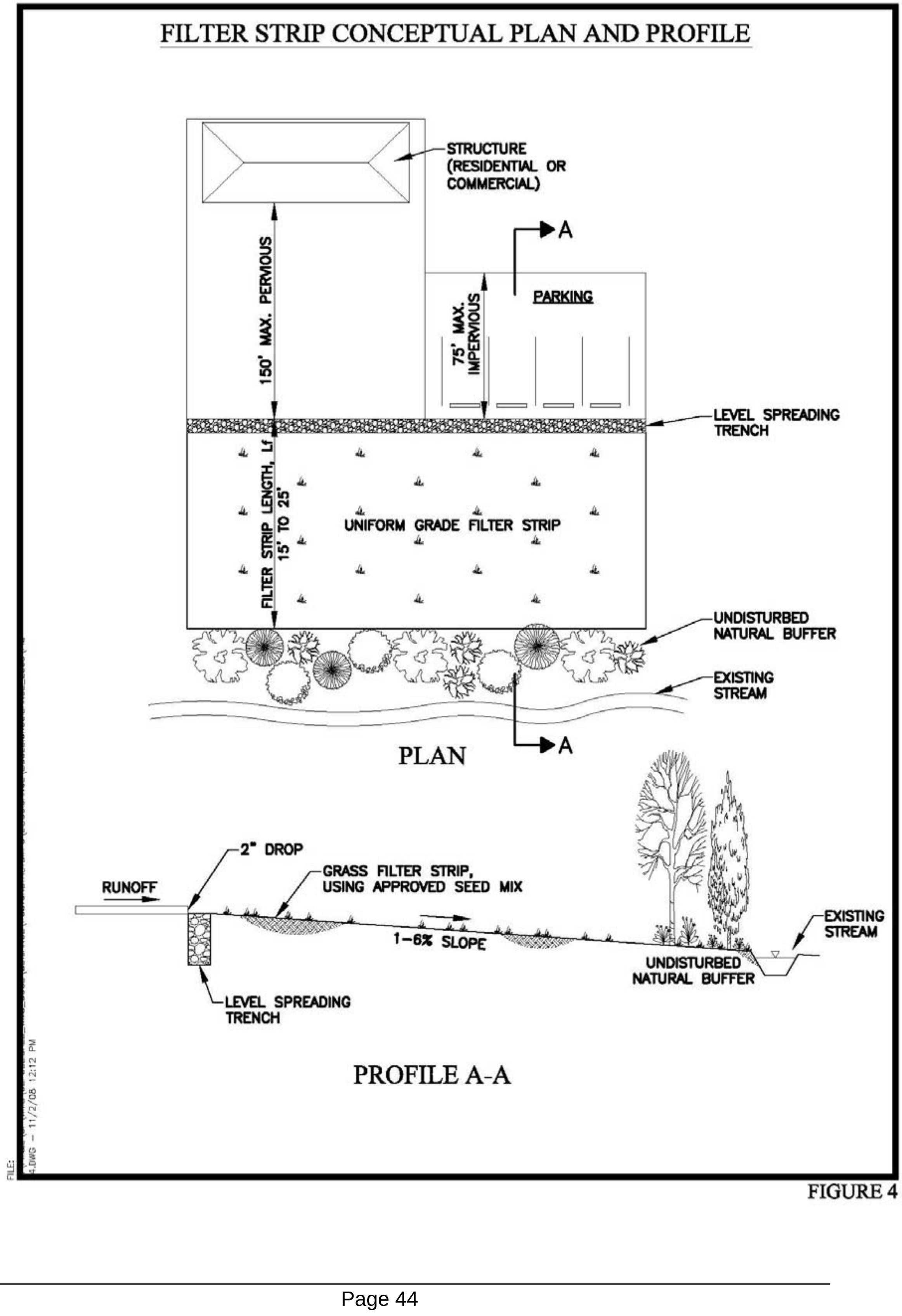


FIGURE 4

5.1 *Filter Strips for Pretreatment*

Filter strips are commonly used for pretreatment in association with other LID elements such as rain gardens and infiltration trenches. Table 8 presents design guidance for slopes and lengths (parallel to flow) of pretreatment filter strips based on the slopes, dimensions, and surface characteristics of the contributing drainage areas.

Table 8 – Pretreatment Filter Strip Design Guidance

Parameter	**Land Cover in Contributing Areas**							
	Impervious Areas				**Pervious Areas**			
Maximum Inflow Approach Length (ft)	35		75		75		100	
Filter Strip Slope (Maximum = 6%)	≤ 2%	> 2%	≤ 2%	> 2%	≤ 2%	> 2%	≤ 2%	> 2%
Minimum Filter Strip Length (ft)	10	15	20	25	10	12	15	18

(MOA, 2004)

5.2 *The Filter Strip Design Process*

The filter strip design process involves preliminary site evaluation, preliminary and final design. The following subsections present the minimum site–specific factors that are to be considered when evaluating a site for the potential use of a filter strip as primary LID elements discharging to storm water conveyance systems, natural areas, or receiving waters. These sections include a site evaluation checklist and preliminary design calculation table to guide readers through design processes for filter strips.

5.2.1 Preliminary Site Evaluation – Filter Strips

The minimum preliminary site evaluation considerations presented below do not include some typical engineering considerations such as resolving utility conflicts and are not a substitute for sound engineering judgment.

5.2.1.a Runoff Source

Filter strips are intended to treat runoff from urban and suburban drainage areas where pollutant loads come from residential, parking, and road surface runoff. Filter strips are not appropriate to receive runoff from industrial facilities or from areas where runoff is likely to contain industrial pollutants.

5.2.1.b Contributing Area

Filter strips are suitable to treat small drainage areas, generally one acre or less in size. It is possible to treat runoff from large areas if multiple filter strips are used. For effective performance, runoff must enter the filter strip as sheet flow. Runoff tends to concentrate within 75 feet along impervious surfaces and within 150 feet along pervious surfaces. Longer flow paths upstream of filter strips are acceptable, but require special consideration to ensure design flows are spread evenly across the surface of the filter strips.

5.2.1.c Slope of the Contributing Area and Filter Strip

The contributing drainage area slopes should be less than 10% for effective performance. Steeper slopes require additional energy dissipation to promote the dispersion of storm water evenly across the length of the filter strips and to prevent erosion. Slopes parallel to the flow path across filter strips should be between 1 and 6%.

5.2.1.d Available Area

For a given site, filter strip length, parallel to the direction of flow, is dependent on slope, vegetative cover, and soil type. Generally, filter strips should extend a minimum of 15 feet in the direction of flow, with 25 feet preferred if space is available. Filter strip width, perpendicular to the direction of flow, should be equal to the width of the contributing drainage area. When filter strips are the primary LID element providing storm water treatment, the ratio of contributing area to filter strip area should not exceed 6:1.

To assist designers in the evaluation of sites for use of a filter strip, a checklist of each of the above considerations is provided in Table 9. A site must meet all of the requirements discussed in the subsections above to be a candidate for the use of a filter strip.

5.2.2 Preliminary Design – Filter Strips

If the preliminary site evaluation indicates that a site is a good candidate for the use of filter strips to treat storm water, the preliminary design can proceed to establish approximate filter strip dimensions. Determining the dimensions of filter strips during preliminary design is an iterative process. There are several important considerations to be made when performing the preliminary design of a filter strip. Descriptions of the recommended preliminary design considerations are provided in the subsections below.

5.2.2.a Filter Strip Slope

Filter strip slopes should generally range from 1% to 6% for effective performance. Slopes at the top and toe of filter strips should be as flat as possible to encourage sheet flow and prevent erosion. The maximum allowable lateral slope (perpendicular to the direction of flow) for filter strips should not exceed 1%.

Table 9 – Filter Strips – Preliminary Site Evaluation Checklist

Site Location:		**Evaluated by:**		
Date:				
Considerations	**Requirement/Recommendation**	**Site Conditions/Notes**	**Pass/Fail**	**Data Source**
Runoff Source	The filter strip is not to receive runoff containing industrial pollutants.			
Contributing Area	The contributing area must be less than 1 acre.			
Slope of the Contributing Area	Slope of the contributing area must be less than 10%.			
Available Area	The available area for the filter strip shall generally extend the full width of the contributing area and allow for a length (parallel to flow) of 15 to 25 feet. The ratio of total contributing area to the total available area must not exceed 6:1.			

5.2.2.b Filter Strip Flow Depths

Flow depths on filter strip surfaces should not exceed 0.5 inches. At depths greater than 0.5 inches, treatment through infiltration is reduced as deeper flows tend to push filter strip grasses parallel to the ground.

5.2.2.c Maximum Discharge Loading

The maximum discharge load represents the maximum flow rate that can cross the threshold of a filter strip without compromising the filter strip performance. The maximum discharge loading refers to the flow entering the filter strip. The calculation of maximum discharge loading per foot width along the filter strip is based on Manning's equation, as shown below.

$$q = \frac{1.49}{n} * \left(\frac{Y}{12}\right)^{\frac{5}{3}} * S^{\frac{1}{2}}$$

Equation 5.1

q = Volumetric Discharge per Foot Width (feet^3/second–foot)
Y = Maximum Allowable Depth of Flow (inches), 0.5
S = Slope of Filter Strip (feet/foot), between 1% and 6%
n = Manning's "n" Roughness Coefficient, Equal to 0.2 for mowed grass and 0.25 for unmowed grass

5.2.2.d Maximum Allowable Design Velocity

The maximum allowable design velocity is the minimum allowable velocity along the filter strip under normal design conditions. The maximum allowable velocity for filter strips is 0.9 feet per second. This is based on the calculated volumetric discharge per foot width and the design flow depth. The maximum allowable design flow depth is 0.5 inches. The design velocity can be calculated using the following formula.

$$V = \frac{q}{Y/12}$$

Equation 5.2

V = Velocity (feet/second), 0.9 feet^3/second maximum
q = Volumetric Discharge per Foot Width (feet^3/second–foot)
Y = Maximum Allowable Depth of Flow (inches), 0.5 inches maximum

5.2.2.e Minimum Allowable Filter Strip Width

The minimum width (W_{fp}) of a filter strip, which is the dimension perpendicular to flow, is a function of flow rate entering and exiting the filter strip, according to equation shown below.

$$W_{fp} = \frac{A_a * C * 0.5}{q}$$

Equation 5.3

W_{fp} = Width of Filter Strip Perpendicular to Flow Path (feet)
A_a = Area (acres)
C = Runoff Coefficient per the DDG
q = Volumetric Discharge per Foot Width (feet^3/second-foot)

5.2.2.f Filter Strip Length

Filter strip length is the dimension parallel to flow. Filter strip length should be calculated for a travel time of 5 to 9 minutes according to the Soil Conservation Service (SCS) Technical Release 55 (TR–55) travel time equation (SCS, 1986) shown below.

$$L_f = \frac{T_t^{1.25} * P^{0.625} * (S * 100)^{0.5}}{3.34 * n}$$

Equation 5.4

L_f = Length of Filter Strip Parallel to Flow Path (feet), 15 to 25 feet
T_t = Travel Time through Filter Strip (minutes), 5 minutes minimum
P = Precipitation (inches) (SCS parameter used to calibrate this equation); 1.3 for the 2–Year, 24–Hour Storm
S = Slope of Filter Strip (ft/ft), 0.01 to 0.06 ft/ft

n = Manning's "n" Roughness Coefficient, Equal to 0.2 for mowed grass and 0.25 unmowed grass

To assist designers in the preliminary design of a filter strip, a sample calculation sheet has been developed and is presented as Table 10. The calculation sheet covers the above considerations and equations in 4 steps.

Step 1 – Calculate the Maximum Discharge Loading
This step is based on guidance provided in Subsection 5.2.2.a and Equation 5.1 presented in Subsection 5.2.2.c above.

Step 2 – Check Velocity
This step is based on Equation 5.2 and guidance provided in Subsection 5.2.2.d.

Step 3 – Calculate the Minimum Allowable Filter Strip Width
This step is based on Equation 5.3 and guidance provided in Subsection 5.2.2.e above.

Step 4 – Calculate the Minimum Allowable Filter Strip Length
This step is based on Equation 5.4 and guidance provided in Subsection 5.2.2.f above.

Once the site evaluation and preliminary design have been completed the final design can be conducted.

5.2.3 Final Design – Filter Strips

To develop a final filter strip design based on the results of the preliminary design, there are several basic factors that must be addressed. Addressing these factors requires some basic understanding of engineering and hydraulic principles. At a minimum, each of the factors discussed in the subsections below should be considered during final design.

Table 10 – Filter Strip Preliminary Design

Site Location: **Evaluated by:**			
Date:			
Step 1: Calculate the Maximum Discharge Loading, q			**Notes**
Maximum Allowable Depth of flow, Y		(in)	Maximum is 0.5 inches
Slope of Filter Strip, S		(ft/ft)	Between 0.01 and 0.06
Manning's "n"			
$q=(1.49/n)*(Y/12)^{(5/3)}*S^{(1/2)}$		(ft^3/sec–ft)	Using Equation 5.1
Step 2: Check Velocity, V			Maximum Allowable is 0.9 ft/sec
q (from Step 1)		(ft^3/sec–ft)	
Y (from Step 1)		(in)	
V=q/(Y/12)		(ft/sec)	Using Equation 5.2
Step 3: Calculate the Minimum Allowable Filter Strip Width, W_{fp}			
q (from Step 1)		(ft^3/sec–ft)	
Contributing Area, A_a		(acres)	
Runoff Coefficient, C			Per DDG
$W_{fp} = (A_a*C*0.5)/q$		(ft)	Using Equation 5.3
Step 4: Calculate the Minimum Allowable Filter Strip Length, L_f			
Travel Time Through Filter Strip, T_t		(min)	Between 5 and 9
Calibration Precipitation, P		(in)	1.3 inches
S (from Step 1)		(ft/ft)	
n (from Step 1)			
$L_f=(T_t^{1.25}*P^{0.625}*S*100)^{0.5})/3.34*n$		(ft)	Using Equation 5.4

5.2.3.a Overall Site Integration

Site designs should incorporate filter strips as elements in the overall site plan. Filter strips can outfall to a variety of features, such as natural buffer areas, vegetated swales, curb and gutter systems, or natural drainage features.

5.2.3.b Filter Strip Cover

Filter strip cover may consist of existing vegetation, hearty native vegetation, planted turf grasses, or a mixture of grasses and shrub vegetation. Optimal vegetation arrangements incorporate plants with dense growth patterns, fibrous root systems for stability, and adaptability to local soil and climatic conditions. The MOA has developed three different types of seed mixtures suited for a variety of applications, including filter strips. These seed mixtures are summarized in Chapter 2 of the MOA *Design Criteria Manual*. Filter strips can also incorporate vegetation including sedges and flowers.

5.2.3.c Level Spreading Devices

Level spreading devices installed upstream of filter strips produce uniform sheet flow conditions along the entire leading edge of the filter strip, and help prevent concentration of flows that create erosive conditions. Level spreaders have a number of different configurations with one common function – to spread concentrated flow into sheet flow upstream of filter strips. The following examples describe common features and applications of two types of level spreading devices.

<u>Level Spreading Trench</u>

This device consists of a gravel–filled trench installed along the entire leading edge of a filter strip. Gravel can range in size from pea gravel, as specified by ASTM D 448, to shoulder ballast for roadways. Level spreading trenches typically have widths of 12 inches and depths of 24 to 36 inches, and they typically use nonwoven geotextile linings. A 1–inch to 2–inch drop between the adjacent impervious surface and the edge of the trench inhibits the formation of an initial deposition barrier. In addition to acting as a level spreader, these trenches also act as pretreatment devices, allowing sediment to settle out before reaching the filter strip.

<u>Natural Berms</u>

Shaping and grading of the area immediately upslope of a filter strip into a berm can also promote uniform sheet flow conditions. This method has a more natural appearance, though the berms can fail more readily than other devices due to irregularities in berm elevation and density of vegetation that may grow over time.

5.3 *Filter Strip Construction and Maintenance*

5.3.1 Construction Considerations – Filter Strips

The following subsections summarize the minimum considerations to be made during construction to enhance the effectiveness and function of filter strips. These construction considerations are not all necessarily applicable when using existing undisturbed areas as filter strips.

5.3.1.a Filter Strip Installation

Before beginning construction, install temporary erosion and sediment control measures and ensure that upgradient sites have stabilized slopes. Install the filter strips during a time of year when successful establishment of vegetation can occur with little or no irrigation, and use temporary irrigation during dry periods. Clear and grub the site as necessary for filter strips that incorporate planted rather than native vegetation. During installation, disturb as little existing vegetation as possible and avoid soil compaction.

5.3.1.b Grading and Level Spreader Installation

Accurate grading must occur during the construction of filter strips, because even small departures from design slopes can affect sheet flow conditions and decrease filter strip effectiveness. Use the lightest, least disruptive equipment when rough grading slopes to avoid excessive compaction and land disturbance. Following the rough grading, install level spreading devices at the upgradient edges of filter strips. If using a gravel trench, do not compact the subgrade and follow the construction sequence for infiltration trenches.

5.3.1.c Vegetation Establishment

Seeding should be performed immediately after grading. Simultaneously stabilize seeded filter strips with temporary techniques such as erosion control matting or blankets. Maintain erosion control for seeded filter strips for at least 75 days following the first storm event of the season.

5.3.2 Maintenance Considerations – Filter Strips

The application of regular maintenance procedures enables filter strips to function properly over long periods of time. The following subsections outline suggestions for consideration when developing a maintenance plan and schedule as required by the DDG.

5.3.2.a Soil

In areas where heavy metals deposition is likely, it is recommended that soils should be removed and replaced once every 20 years. Replacing soil in filter strips is likely to provide a prolonged service life. When replacing soil in filter strips, refer to recommendations for engineered soils in rain gardens provided in Appendix C of this manual.

5.3.2.b Watering and Weeding

Periodic watering is required in the first year to help grass become established. Watering may also be required during prolonged dry periods. Weeding should be performed as necessary to maintain a healthy grassed top layer.

5.3.2.c Routine Post–Storm Inspection

Filter strips should be inspected after large rain events and should be inspected for evidence of erosion, which is not likely in properly designed systems. Any visible trash accumulated-on the filter strips should be removed.

5.3.2.d Vegetation Maintenance

Basic maintenance of filter strips involves normal landscaping maintenance activities such as mowing, trimming, removal of invasive species, and replanting when necessary. Filter strips receiving large amounts of sediment may require periodic regrading and reseeding of their upslope edges. If a high volume of sediment builds up, creating concentrated flows and channels, filter strips may require reworking or replanting. Grass should be maintained at a length of 3 to 8 inches. Allowing grass to grow taller can cause thinning, which compromises the effectiveness of the vegetative cover. The removal of clippings and regular maintenance promotes vegetation growth and pollutant uptake.

5.3.3 Filter Strip Conceptual Design Example

A conceptual design example for a filter strip is provided in Appendix G of this manual.

6. Additional LID Elements for Consideration

6.1 *Storm Water Discharge to Wetlands*

One of Anchorage's many unique features is the collection of small and large urban wetlands present within the city. These urban wetlands often represent small remaining portions of what had been larger wetlands that were present before the development of the city. The urban wetlands provide natural habitat for native animals, recreational opportunities for residents, and contribute to the natural character of the city. These wetlands can provide another service to the city in the form of storm water retention and treatment.

Many of the concepts that LID elements aim to incorporate, such as filtration and pollutant uptake, are natural functions of wetlands. The use of a wetland to provide storm water treatment is a natural alternative to the construction of an LID element within, or adjacent to a site. There are a number of factors to be weighed when considering the discharge of storm water to wetlands. These factors include pretreatment, which typically involves the removal of large sediments and floatables, and wetland capacity, which is the wetland's ability to receive and treat storm water without harm to wetland ecosystems.

In 2002, Watershed Management Services (WMS) developed guidance for storm water treatment in wetlands. The guidance document, titled: *Anchorage Storm Water Treatment in Wetlands: 2002 Guidance* is available through WMS at http://wms.geonorth.com/.

6.2 *Constructed Wetlands*

Constructed wetlands are designed and constructed to temporarily store storm water runoff in shallow pools that support conditions suitable for the growth of wetland plants. They can provide value in terms of natural aesthetics, wildlife habitat, erosion control, and pollutant removal. Because constructed wetlands are artificial, they typically do not have the full range of ecological functions of natural wetlands. Constructed wetlands require large tributary drainage areas or perennial baseflow to assure adequate water to sustain wetland vegetation during dry periods. They can be generally classified as either constructed wetland basins or constructed wetland channels.

A constructed wetland basin is a shallow retention pond that requires a perennial baseflow to permit the growth of rushes, willows, cattails, and reeds to slow runoff and allow time for sedimentation, filtering, and biological uptake. Flood control storage can be provided in addition to the design storm volume, and the system can be designed to meet both flood control and retention requirements under the DDG.

A constructed wetland channel is a shallow conveyance feature that takes advantage of dense natural vegetation to slow down runoff, and allow time for sedimentation and storm water treatment through biological uptake and other mechanisms. Constructed wetland channels can be located downstream of storm water detention facilities (water quality and/or flood control) where a large portion of any residual sediments can be removed. Considerations for constructed wetland basins and channels are summarized in Table 11.

Table 11 – Considerations for Constructed Wetlands

Consideration	Constructed Wetlands Basin	Constructed Wetlands Channel
Drainage Area	25 acres or greater	5 acres or less
Ideal Application	Rural or residential	Rural or residential
Infiltration Rate	Soils with high infiltration rates (greater than 2 inches per hour) may require a liner.	
Depth to Water Table	1 to 4 feet	At or less than 1 foot

(modified from MOA, 2004)

The primary design and maintenance considerations for constructed wetlands provided in the bulleted list below have been adapted from the 2004 MOA document titled *Low Impact Development in Anchorage: Concepts and Criteria, Review Copy*.

- The need for a continuous baseflow to ensure viable wetland vegetation growth. This should be determined using a water budget analysis to show that the net inflow of water is sufficient to meet all the projected losses (such as evaporation, evapotranspiration, and seepage) for each season of operation.

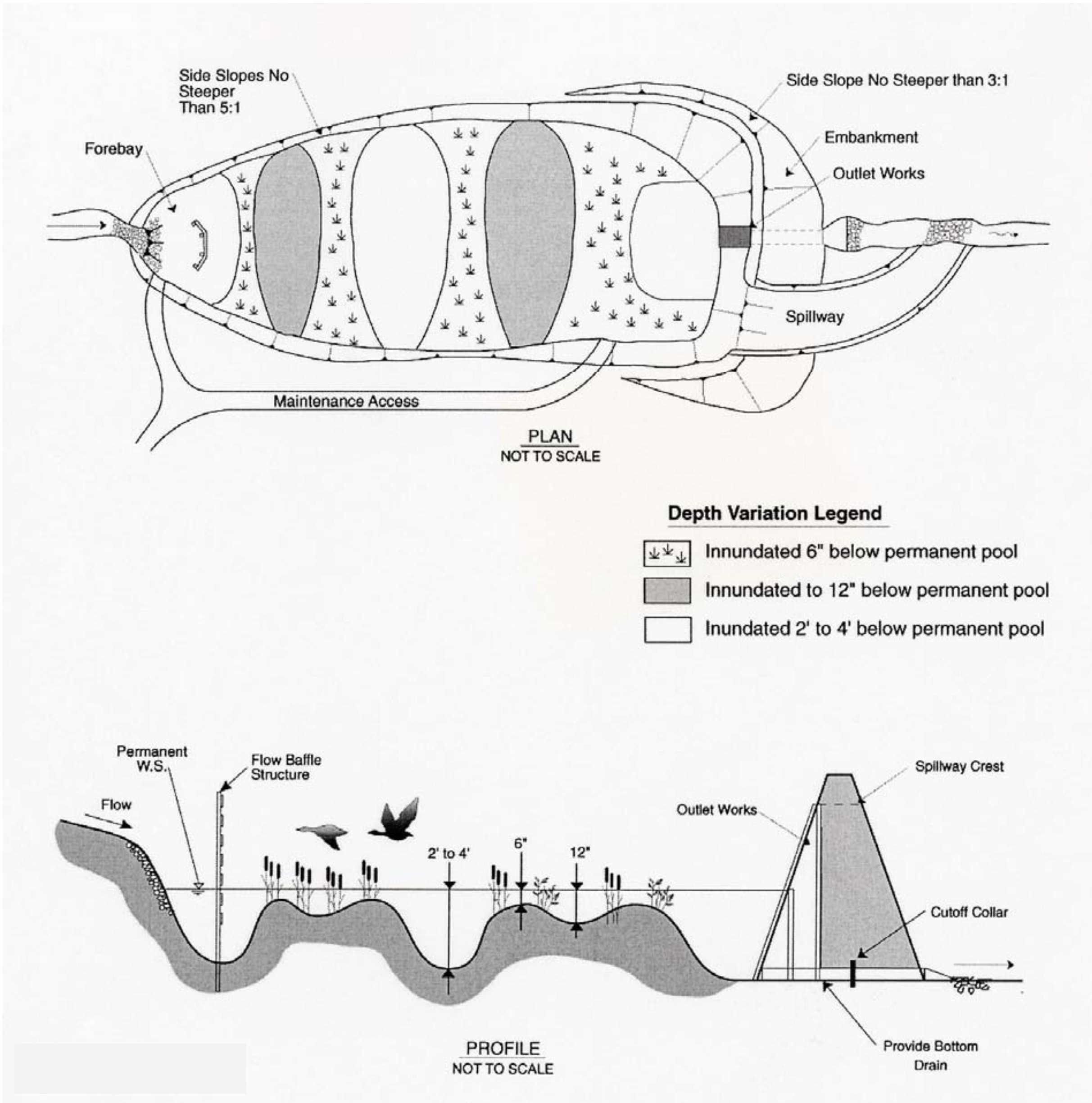
- In order to maintain healthy wetland growth, the surcharge depth above the average water surface should not exceed 2 feet.

- Along with routine vegetation and good housekeeping maintenance, periodic sediment removal is required when sediment accumulations become too large and affect storm water treatment performance. Periodic sediment removal ensures proper distribution of growth zones and water movement within the wetland.

Figure 5 shows a conceptual constructed wetland basin, and Figure 6 illustrates a conceptual constructed wetland channel.

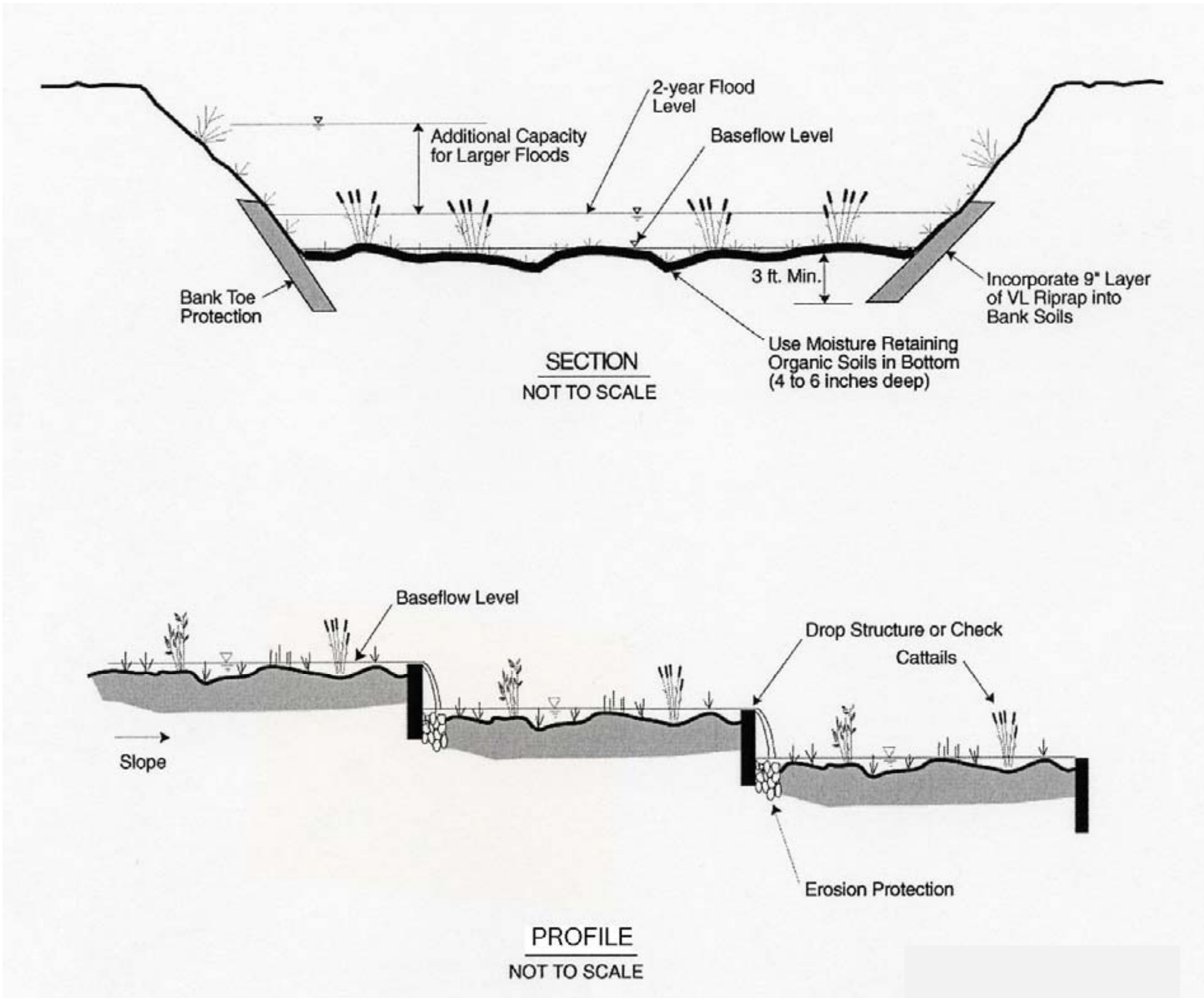
Constructed wetland design requires a thorough understanding of basic hydrologic and hydraulic principles, and wetland mechanics. The basin design steps presented in Table 12 are provided to assist storm water professionals in the design of constructed wetland basins to meet extended detention requirements presented in the DDG. Table 13 is provided to assist in the design of constructed wetland channels to meet DDG water quality requirements. Note that the steps presented in both tables are also applicable when designing constructed wetlands to meet DDG wetland retention requirements as long as the correct hydrologic inputs are used.

Figure 5 – Conceptual Constructed Wetland Basin



(MOA, 2004)

Figure 6 – Conceptual Constructed Wetland Channel



(MOA, 2004)

Table 12 – Design Steps for a Constructed Wetland Basin

<table>
<tr><th>Step</th><th colspan="3">Design</th></tr>
<tr><td>1. Basin Surcharge Storage Volume</td><td colspan="3">Provide a surcharge storage volume equal to the post-development project runoff in excess of the pre-development runoff volume for the 1–year, 24–hour storm. Level pool routing may be applied to reduce volume requirements provided that retention times remain between 12 and 24 hours.</td></tr>
<tr><td rowspan="5">2. Wetland Pond Depth and Volume</td><td colspan="3">The volume of the permanent wetland pool should be no less than 75% of the design storm volume from Step 1. Proper distribution of wetland habitat is needed to establish a diverse ecology. Distribute pond area in accordance with the following:</td></tr>
<tr><td>Component</td><td>Pool Surface Area</td><td>Water Design Depth</td></tr>
<tr><td>Forebay, outlet and free water surface area</td><td>30% to 50%</td><td>2 to 4 feet deep</td></tr>
<tr><td>Wetland zones with emergent vegetation</td><td>50% to 70%</td><td>6 to 12 inches deep*</td></tr>
<tr><td colspan="3">*One–third to one–half of this zone should be 6 inches deep.</td></tr>
<tr><td>3. Depth of Surcharge</td><td colspan="3">The surcharge depth of the design storm volume above the average water surface should not exceed 2 feet.</td></tr>
<tr><td>4. Outlet Works</td><td colspan="3">Use an outlet that is capable of releasing the design storm volume in a 12– to 24–hour period.</td></tr>
<tr><td>5. Trash Rack</td><td colspan="3">Provide a trash rack of sufficient size to prevent clogging of the primary outlet. Size the rack so as not to interfere with the hydraulic capacity of the outlet. Use one–half of the total outlet area to calculate the trash rack’s size.</td></tr>
<tr><td>6. Basin Use</td><td colspan="3">Determine if flood storage or other uses will be provided for above the surcharge depth. Design for combined uses when they are to be provided for.</td></tr>
<tr><td>7. Basin Shape</td><td colspan="3">Shape the pond with a gradual expansion from the inlet and a gradual contraction to the outlet, thereby limiting short–circuiting. Try to achieve a basin length to width ratio between 2:1 to 4:1. It may be necessary to modify the inlet and outlet works through the use of pipes, swales, or channels, to accomplish this. Always maximize the distance between the inlet and outlet.</td></tr>
<tr><td>8. Basin Side Slopes</td><td colspan="3">Basin side slopes are to be gentle and stable to facilitate maintenance and access. Side slopes should be no steeper than 4:1, and should preferably be 5:1, or flatter.</td></tr>
<tr><td>9. Base Flow</td><td colspan="3">A net influx of water must be available throughout the year that exceeds all of the losses.</td></tr>
<tr><td>10. Inlet/Outlet Protection</td><td colspan="3">Provide a means to dissipate flow energy entering the basin to limit sediment resuspension. Outlets should be placed in an outlet bay that is at least 3 feet deep. The outlet should be protected from clogging by a skimmer shield that starts at the average water depth and extends above the maximum capture volume depth.</td></tr>
<tr><td>11. Forebay Design</td><td colspan="3">Provide the opportunity for larger particles to settle out in an area where the bottom has a solid driving surface to accommodate heavy equipment used to remove sediment. The volume of the forebay should be 5% to 10% of the design surcharge volume.</td></tr>
<tr><td>12. Vegetation</td><td colspan="3">Cattails, sedges, reeds, and wetland grasses should be planted in the bottom of the wetland. Berms and side–sloping areas should be planted with native turf–forming grasses. Initial establishment of wetland vegetation requires control of the water depth. After planting wetland species, water depths should be limited to 3 to 4 inches to allow establishment of wetland plants, after which the pool should be allowed to fill to its operating level.</td></tr>
<tr><td>13. Maintenance Access</td><td colspan="3">Provide vehicle access to the forebay and outlet area for maintenance and removal of bottom sediments. Maximum grades should not exceed 10%, and a stabilized, all–weather driving surface needs to be provided. Provide a concrete, or grouted boulder–lined bottom and side–slopes in the forebay area to define sediment removal limits and permit the operation of heavy equipment.</td></tr>
</table>

(modified from MOA, 2004)

Table 13 – Design Steps for Constructed Wetlands Channel

Step	Design
1. Design Flow Rate	Determine the 1–year, 24–hour peak flow rate in the wetland channel without reducing it for any upstream ponding or flood routing effects.
2. Channel Geometry	Design the channel's geometry to pass the design 1–year, 24–hour flow rate with a maximum velocity of 2 feet per second with a water depth between 2 to 4 feet. The channel cross–section should be trapezoidal with side slopes of 4:1 (horizontal/vertical) or flatter. Bottom widths should be no less than 8 feet.
3. Longitudinal Slope	Set the longitudinal slope using Manning's equation and a Manning's roughness coefficient of n=0.03, for the 1–year, 24–hour flow rate. If the desired longitudinal slope cannot be attained with existing terrain, grade control checks, or small drop structures must be incorporated to provide the desired slope.
4. Final Channel Capacity	Calculate the final channel capacity for the 1–year, 24–hour flow rate using a Manning's roughness coefficient of n=0.08, and the same geometry and slope used when initially designing the channel with n=0.03. Adjustment of the channel capacity may be done by increasing the bottom width of the channel. Minimum bottom width should be 8 feet.
5. Drop Structures	Drop structures should be designed to eliminate the potential for scour.
6. Vegetation	Vegetate the channel bottom and side slopes to provide solid entrapment and biological nutrient uptake. Cover the channel bottom with loamy soils upon which cattails, sedges, and reeds can be established. Side slopes should be planted with native or irrigated turf grasses.
7. Maintenance Access	Provide access for maintenance vehicles along the channel length. Provide a solid driving surface with a maximum grade of 10% for maintenance vehicles.

(modified from MOA, 2004)

6.3 *Pervious Pavements*

One approach to lowering the overall imperviousness of an area, while retaining necessary surfaces for fire lanes, shoulders, sidewalks, etc., is the use of porous pavement technologies. Some porous pavement technologies are not applicable in areas where sanding is common. However, other types of porous pavement can be used when adequate underdrainage, such as a sand or gravel bed, is provided. Porous pavement types suitable for application in Anchorage are discussed below.

<u>Open–Graded Aggregate</u> – This is unbound aggregate, single–sized, angular, durable, and clean of fine particles so that dust is not generated.

<u>Open–Jointed Paving Blocks or Interlocking Concrete Pavements</u> – These are modular paving units that allow infiltration between individual units. They are typically built over an open–graded or rapid–draining crushed stone base, with less than 3% fines passing the No. 200 sieve (see Figure 7). Perforated drainage pipes can provide drainage in heavy overflow conditions, or provide secondary drainage if the base loses some of its capacity over time. For installations where slow–draining subgrade soils are present, perforated pipes can drain excess runoff and alleviate potential for frost heaving.

Figure 7 – Open–Jointed Paving Block

(MOA, 2004)

Concrete Grids – These are perforated concrete units installed over a compacted soil subgrade, which overlies a dense–graded base of compacted crushed stone, which in turn overlies a 1 to 1–1/2 inch thick bedding sand (see Figure 8). The openings in the grids are filled with either topsoil and grass or aggregate.

Figure 8 – Concrete Grid

(MOA, 2004)

Plastic Lattices (Geocells) – These are interlocking, high–strength blocks made from plastic materials. They provide vehicular and pedestrian load support over grass areas while protecting the grass from the harmful effects of traffic. The system is comprised of base support soil beneath the lattice unit, which is then filled with selected topsoil, and seeded with selected vegetation.

The benefits of porous pavement technologies include the following:

- Porous pavements provide a pervious, load–bearing surface with minimal increases in imperviousness.

- Application of pervious pavement technologies can reduce site runoff and limit the degree of complexity required for storm drain design and analysis under the DDG.
- In some cases, construction costs of porous pavements can be less than conventional pavements.
- Pavers can be installed with heating coils to promote ice and snow melt.
- Soil–enhanced turf systems are advantageous for sports and recreation fields because they resist compaction, promote infiltration, and provide a soft playing surface.

Though porous pavement technologies have a number of potential applications and benefits, there are some limitations that bear consideration. These limitations include the following:

- Sand and salt in snowmelt runoff can cause clogging of porous pavements. However, studies suggest that permeable surfaces can be used successfully, especially if they are installed properly (backfilled with clean gravel), and maintained through semi–annual vacuum cleaning.
- Construction costs of porous pavements can be higher in some cases than conventional pavement, depending on the application, and maintenance costs are usually higher.
- Most porous pavements limit wheelchair access and do not meet Americans with Disabilities Act standards, thus limiting their applicability in foot traffic areas.

Some design considerations for porous pavement are listed below.

- Assessment of site soil infiltration capacity is required to assure proper functioning of the porous pavement, which should not be installed on clayey soils or in areas of high groundwater.
- Where existing subsoil drainage is poor, install subdrains.
- Plant with drought tolerant turf grass (such as fescue) rather than less drought tolerant strains such as bluegrass.

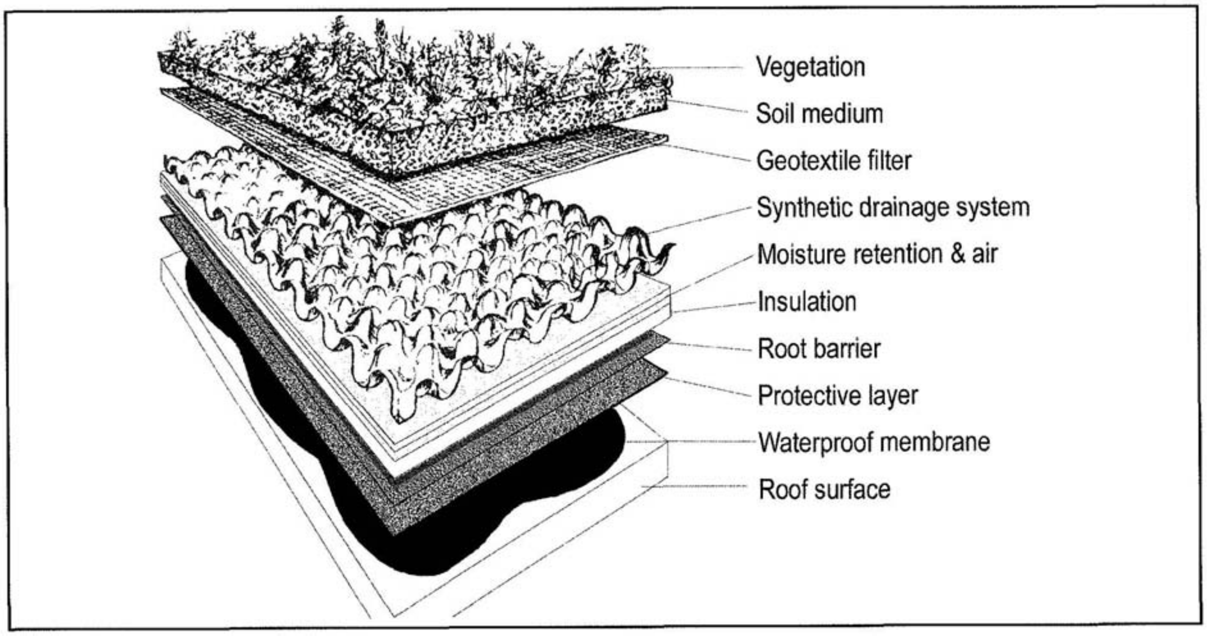
6.4 *Green Roof Technologies*

6.4.1 Green Rooftops

Green rooftops are weather– and moisture–proof roofing systems covered with live vegetation that can be installed on buildings such as warehouses, garages, office buildings, and industrial facilities. Green rooftops are capable of mimicking many of the hydrologic processes associated with vegetated terrain. Some of the rain that falls on green rooftops is captured on the foliage of the vegetation and absorbed into the root zone, encouraging evapotranspiration and reducing rooftop runoff volumes. The portion of the rainfall that becomes runoff is released slowly, reducing the peak runoff flow rate for the site.

Green rooftops can be built in a variety of ways, but generally include a waterproof membrane, a protective layer, a root barrier, insulation, a moisture retention layer, a drainage system, geotextile filter fabric, soil medium, and vegetation. Figure 9 presents a general example of these component layers.

Figure 9 – Green Roof Component Layers



(BEC, 2007)

Green rooftops can be more extensive than the example shown above, and employ soils deeper than 1 foot, large tree and shrub root systems, and structures to support human use such as park benches and walking trails. These green rooftops are generally more expensive to design, construct, and maintain than conventional roofing systems. The discussions presented below are specific to simple green rooftops, such as that shown in Figure 9, with soil depths ranging from 2 to 6 inches deep.

6.4.2 Advantages

Green rooftops have several advantages over conventional rooftops. The advantages discussed below were taken from the following two sources: *Low Impact Development in Anchorage: Concepts and Criteria, Review Copy, 2004,* and *Minnesota Urban Small Sites BMP Manual: Storm Water Best Management Practices for Cold Climates, 2001.*

<u>Runoff Peak and Volume Reduction</u> – Unlike traditional roofing materials, such as tar or shingles, green roof systems detain, filter, and slowly release storm water, reducing the peak flows and overall volume of runoff. If widely implemented, green rooftops have the potential to reduce storm water runoff and nonpoint source pollution problems in urban and suburban environments. A study conducted in Chicago in the summer of 2003 indicates that runoff volumes from vegetated rooftops may be as low as a quarter of that from conventional rooftops for low–intensity storms, such as the 2–year 6–hour storm in Anchorage (MOA, 2004).

Reduction of the Urban Heat Island Effect – Conventional rooftops absorb heat, and have been reported to reach temperatures has high as 175º Fahrenheit (F). The radiation of this heat to the surrounding air can cause temperatures in large cities to be as much as 6º to 10º F higher than surrounding suburban and rural areas. This phenomenon is known as the "urban heat island effect." Green rooftops help reduce the urban heat island effect since they trap and absorb much less heat than conventional rooftops, lower air temperatures through plant transpiration and evaporative cooling.

Improvement of Air Quality – Urban heat island temperatures exacerbate air pollution, contributing to the formation of smog and ozone. Warm air rising from conventional rooftops can circulate fine particulate matter and further degrade air quality. These increases in air pollution increase the risk of health complications, and reduce the quality of life for those who work and live in cities. Green rooftops indirectly help alleviate these air pollution problems by reducing air temperatures. Additionally, plants on rooftops can contribute directly to enhanced air quality by trapping and absorbing nitrous oxides, volatile organic compounds, particulate matter, and by providing oxygen.

Energy Conservation – The additional insulation provided by green rooftop materials can reduce the amount of energy required to maintain warm interior temperatures in the winter. In the summer, rooftop plants located near intakes for air conditioning systems will transpire, lowering the temperature of incoming air and reducing energy required to cool the building's air supply.

Longer Service Life – Green rooftop manufacturers/installers claim that their products will last at least 40 years, versus the 10 to 15 year lifespan of a conventional roof (BEC, 2007). This reduces replacement costs and the amount of materials needed for roofs.

Avian Wildlife Habitat – Vegetation on green rooftops provides wildlife habitat for birds and other species.

Improved Urban Aesthetics – Green rooftops provide more attractive views from other buildings than do traditional roofing materials.

Meeting DDG Runoff Requirements – Green rooftops can be used in urban and suburban applications to reduce rainfall runoff volumes and peaks to be less than the threshold values for small simple sites (0.22 cfs/acre for the 1–year, 24–hour–event or 0.41 cfs/acre for the 10–year, 24–hour event). This will allow the site to qualify as a "simple small project" rather than a "complex small project," eliminating requirements for extended detention and downstream impact analyses under the DDG.

6.4.3 Disadvantages

There are also some disadvantages to green rooftops over conventional rooftops. For instance, because some rainfall is detained on the roof, any leaks in the waterproof membrane may result in significant damage to the interior of the building. Green rooftops can also be expensive to design and construct, especially when retrofitting existing buildings. Those constructed on steeply sloped rooftops (slopes greater than 9%) require special consideration for erosion control. Once constructed, exposure to extreme sun and wind conditions can present challenges for plant survival, and the rooftop will require more maintenance than is required for conventional rooftops. Finally, snow loads may limit applications to rooftops with high load–bearing capacities.

6.4.4 Design Considerations

The following design considerations are provided to promote the use of green rooftops in the Anchorage area. The list of design considerations below is not exhaustive and is not intended to supplant sound engineering judgment.

Load Bearing Capacity – The load–bearing capacity of the underlying roof deck is a critical consideration in designing a green rooftop. This means considering both dead load (the total weight of roof materials including soil and plants, along with snow) and live load (loads due to wind, maintenance personnel, etc.). Generally, green rooftops weighing more than 17 pounds per square foot in a saturated condition require consultation with a structural engineer (BEC, 2007).

Wind Uplift – Wind uplift codes are not yet in place for green rooftops. Uplift pressures tend to be higher at roof corners and perimeters, therefore, it is recommended that these areas be designated as "vegetation–free zones."

Roof Slopes – Flat roofs (or those with a slope less than 1.5%) can generally be designed without any provisions for cross members to hold the component layers in place. However, rooftops with steeper slopes (up to 9 %) require the addition of cross members. Buildings with roofs steeper than 9% are not recommended for green rooftop applications.

Shade Conditions – With all rooftops, sun and shade conditions must be considered and appropriate plant species used. Deeply shaded rooftops may not be suitable for vegetation.

Waterproofing – The waterproofing layers of a green rooftop include a waterproof membrane, a protective layer, and a root barrier layer. Waterproof membranes come in two basic varieties – monolithic and single ply. Monolithic membrane, a rubberized asphalt applied as a hot liquid, is generally thought to provide superior waterproofing and require less maintenance. In retrofit applications, existing roofing material will need to be removed to allow installation directly on the roof surface. Single ply membrane is available in rubber or plastic. Generally, this membrane is installed over a vapor barrier and insulating layer. Commercial low–slope roof membranes often used in Anchorage are listed below. Each of these membranes comes in a variety of thicknesses, ultraviolet susceptibility, colors, etc.

- Single ply ethylene propylene diene monomer (EPDM) membrane
- Single ply polyvinyl–chloride (PVC) membrane
- Single ply thermoplastic olefin (TPO) membrane
- Monolithic multi–ply hot asphalt mineral surfaced built–up–roof (MCBUR) membrane

The protective layer and the root barrier layer provide protection for the membrane. Monolithic membranes and single ply membranes require different protective layers. Monolithic membranes require a modified bituminous protective sheet and the single ply membranes require a protective sheet of high density polyethylene.

To prevent both the membrane and the protective layer from root penetration, a root barrier is necessary. These barriers may be either physical or chemical in nature. For most applications, that use shallow rooted plants, a thin physical layer is usually sufficient.

Insulation – Roof insulation with an insulating value of approximately R30 is typically used on commercial buildings in Anchorage. Roof insulation with an insulating value of approximately R38 is currently required on Anchorage residential roofs. An Anchorage practice of using just one layer of insulation is becoming less common. Two layers of insulation on commercial low slope buildings are highly recommended. A building energy model, typically created by mechanical engineers, is useful to determine the necessary rooftop insulation thickness.

Moisture Retention and Drainage – The drainage system, often consisting of recycled–polyethylene elements resembling egg crates, creates a series of small depressions that retain rain water for plant uptake during dry periods, and allow drainage of surplus water. The depth of the drainage layer varies, depending on the level of runoff management desired, and roof–deck load–bearing capacity.

Soils – Soils for green rooftops are lighter weight than typical soil mixes. They generally consist of 75% mineral material and 25% organic material. Soils must be carefully formulated to meet the oxygen, nutrient, and moisture needs of plants, and to have the appropriate pH level (BEC, 2007).

Plants – The range of plant species suitable for use on green rooftops is limited by the extremes of the rooftop microclimate, including high wind, drought and low winter temperatures due to lack of ambient heat (normally retained in the ground). As a result, tundra species are well suited to rooftop applications.

6.4.4 Resources for Additional Information

The following websites contain additional information on green rooftop applications:

- www.greenroof.org
- www.greenroofs.com

7. Glossary of Selected Terms

Freeboard – The vertical distance between the level water surface and the lowest point along the top of a structure, such as a berm, that impounds or restrains the water.

Soak–Away Pit – The term soak–away pit is used in this document to describe the LID element commonly referred to as a dry well. The term soak–away pit however refers specifically to a dry well that does not qualify as a Class V injection well according to EPA regulations.

Zone of Influence – The zone of influence refers to the area of the surrounding subgrade that is critical to proper function and support of the overlying and/or adjacent foundation or road subgrade. Generally, the zone of influence can be defined as the area bounded within a 3–dimensional surface extending at a 1:1 slope down and away from the outer edge of a foundation or road subgrade.

Catchment Area – In this document, catchment area refers to the total area contributing storm water runoff to a particular LID element.

Impervious Collar – In this document the phrase "impervious collar" refers to an impervious barrier constructed around the walls of a soak–away pit. The intent of the impervious collar is to provide a clear hydraulic divide between the wall of the soak–away pit and adjacent structures. The impervious collar may be constructed out of a variety of materials as long as this intent is met.

Cleanout – A cleanout is an access point in a buried storm drain conveyance to allow periodic removal of any collected sediment or debris.

Keyed In – The phrase "keyed in" refers to the condition in which the top edge of a geotextile (impervious or pervious) is folded into the surrounding soil to keep the material from slipping downward over time.

Foot Plate – A foot plate is a plate that can be round or rectangular, is in plan view, and is fixed to the bottom of an observation well. The intent of the foot plate is to provide a foundation for the observation well and prevent any vertical movement. Generally, foot plates should be either plastic or metal with the shortest dimension in plan view being twice the length of the diameter of the observation well.

Hydrologic Soil Group D – Soils with a very low rate of water transmission (less than 0.06 in/hr) (NRCS, 2007).

Runoff Coefficient – Rational Method Runoff Coefficient calculated according to guidance contained in the Municipality of Anchorage *Drainage Design Guidelines*.

Subdrain –A system of underground perforated pipes which are used to collect water that has infiltrated through the soil in a rain garden and transmit it to an underground conveyance.

Underground Conveyance – This term refers to a system of underground storm drain pipes which convey storm water, such as pipes within the existing municipal separate storm sewer system.

8. Annotated Bibliography and Additional References

Atlanta Regional Commission. 2001. *Georgia Storm Water Management Manual, Volume 2*. Atlanta, Georgia. August.

The aim of this manual is to provide an effective tool for local governments and the development community to reduce both storm water quality and quantity impacts, and to protect downstream areas and receiving waters. The first volume of this manual covers storm water policy and the second volume covers technical design. Volume two of this manual contains guidance on storm water management planning, storm water hydrology, structural storm water controls, and storm water drainage system design. This manual includes descriptions and design guidance for bioretention, infiltration trenches, filter strips, and underground sand filters, among others. The manual also includes general design examples.

Auckland Regional Council (ARC). 2003. *Technical Publication # 10: Storm Water Treatment Devices Drainage Design Guidelines*. Auckland, New Zealand. May.

This manual was primarily developed to outline and demonstrate the Auckland Regional Council's preferred design approach for structural storm water management devices. The secondary objectives of this manual include informing readers of the environmental effects of LID management providing a resource guideline for designers. The manual covers storm water management device selection. It also provides design, construction and maintenance guidance for treatment ponds, treatment wetlands, filtration designs, infiltration designs, swale designs, and filter strip designs, among others.

Caraco, Deb and Richard Claytor. 1997. *Storm Water BMP Design Supplement for Cold Climates*. Prepared for the U.S. Environmental Protection Agency Region 5, Office of Wetlands, Oceans, and Watersheds. Prepared by the Center for Watershed Protection. Ellicott City, MD. December.

This document was developed by the Center for Watershed Protection for the EPA to address special design concerns with BMP application in cold climates. The guidance provided in this document is based in part on telephone and write–in surveys of storm water professionals in cold climate regions. The document includes discussions of cold climate design challenges such as pipe freezing, frost heave, short growing seasons, and snow management. The document also includes a short discussion of pertinent hydrologic calculations. Further, the document includes design modifications for storm water treatment in wetlands, infiltration, filtration, and open channel storm water treatment elements. General design examples for these treatment elements are also provided.

Massachusetts Department of Environmental Protection (MDEP). 1997. *Storm water Management, Volume Two: Storm Water Technical Handbook*. March.

The intent of this manual is to provide designers in the state of Massachusetts with general guidance for the design of structural and non–structural storm water treatment elements. The manual includes a general discussion of hydrology as it relates to storm water management. Site planning is briefly addressed and general guidance for the design of detention basins, wet ponds, constructed wetlands, infiltration basins, infiltration trenches, dry wells (soak–away pits), and sand filters, among others.

Minneapolis Metropolitan Council (MMC). 2001. *Minnesota Urban Small Sites BMP Manual: Storm Water Best Management Practices for Cold Climates*. Metropolitan Council, City of Minneapolis, City of St. Paul. July.

This manual was developed to provide best management practices (BMP) tools for the Twin Cities municipalities and watershed management organizations. The BMP tools presented in this manual are specifically aimed at addressing development and redevelopment projects in the Twin Cities area. The manual covers the proper selection of BMP tools and provides extended descriptions and design discussions for a number of BMP tools. The manual includes discussions of impervious road reduction, housekeeping, construction practices, sediment control, and infiltration systems, among others.

Municipality of Anchorage (MOA). 2004. *Low Impact Development in Anchorage: Concepts and Criteria, Review Copy*. Municipality of Anchorage, Watershed Management Services. February.

This document was developed primarily for internal use within the Municipality of Anchorage. The document includes a discussion of the climatic and geologic factors that affect the utility of LID elements in the Anchorage area. The document provides discussion of the design concepts for LID elements including reduced paved parking, vegetated rooftops, infiltration basins, infiltration trenches, and porous pavement. The document also provides general design guidance for infiltration trenches.

Prince George's County. 2000. *Low–Impact Development Design Strategies: An Integrated Design Approach*. Prince George's County, Maryland Department of Environmental Resources, Programs and Planning Division. January.

Prince George's County is credited with the development of several LID elements and with the development of guidance for LID elements that can be applied on the National level. This manual provides general design guidance for a number of LID elements including bioretention, dry wells (soak–away pits), filter strips, infiltration trenches, and level spreaders. The manual also includes guidance on LID site planning, hydrologic analysis, and public outreach. This manual provides generalized design equations as well as guidance on the acceptable ranges for variables within the equations.

Puget Sound Action Team (PSAT). 2003. *Natural Approaches to Storm Water Management: Low Impact Development in Puget Sound*.

This document was developed during the early stages of the period when planners, developers, engineers and others around Puget Sound were transitioning to the LID *approach to storm water management. This manual includes examples of the successful use of LID elements including bioretention, permeable pavement, rooftop rainwater harvesting, and green roofs. The manual also provides examples of ordinances and regulations that have been developed to encourage the use of LID elements in development.*

Additional References

Barr Engineering Company (BEC). 2007. Minnesota *Urban Small Sites BMP Manual*. Prepared for the Metropolitan Council. July.

Daugherty, Robert L. and Joseph B. Franzini. 1977. *Fluid Mechanics with Engineering Applications*. McGraw–Hill Book Company. New York.

Minnesota Pollution Control Agency (MN PCA). 2008. *Minnesota Stormwater Manual*. January. www.pca.state.mn.us/water/stormwater/stormwater-manual.html

Municipality of Anchorage (MOA). 2007a. *Drainage Design Guidelines, Version 4.08*. Municipality of Anchorage, Project Management and Engineering.

MOA. 2007b. *Design Criteria Manual, Chapter 2, Drainage*. Municipality of Anchorage, Project Management and Engineering.

MOA. 2007c. *Rain Gardens: A How To Manual for Homeowners in the Municipality of Anchorage.* Municipality of Anchorage, Project Management and Engineering.

Pratt, Verna. 1989. *Field Guide to Alaskan Wildflowers: A Roadside Guide.* Alaska Drafts, Inc., Anchorage, AK, 136 pages (ISBN: 0–9623192–0–1: 16th Printing 2007).

Soil Conservation Service (SCS). 1986. Technical Release TR–55 Manual.

United States Department of Agriculture (USDA). 2007. *Selected Invasive Plants of Alaska.* Forest Service Alaska Region R10–TP–130B. www.fs.fed.us

United States Department of Agriculture. Natural Resources Conservation Service. (NRCS) 2007. *National Engineering Handbook Part 630 Hydrology Chapter 7 Hydrologic Soil Groups.* May. http://policy.nrcs.usda.gov/

United States Environmental Protection Agency (EPA). 2007. *Reducing Stormwater Costs through Low Impact Development (LID) Strategies and Practices*. EPA 841-F-07-006. December. www.epa.gov/nps/lid

Wisconsin Department of Natural Resources (WI DNR) 2004. Storm Water Post-Construction Technical Standards *Site Evaluation for Stormwater Infiltration (1002)* http://www.dnr.state.wi.us/runoff/stormwater/techstds.htm

Wurbs, Ralph A. and Wesley P. James. 2002. *Water Resources Engineering*. Prentice Hall. Upper Saddle River, NJ.

Appendix A
Class V Injection Well Memorandum

UNITED STATES ENVIRONMENTAL PROTECTION AGENCY
WASHINGTON, D.C. 20460

JUN 13 2008

OFFICE OF WATER

MEMORANDUM

SUBJECT: Clarification on which stormwater infiltration practices/technologies have the potential to be regulated as "Class V" wells by the Underground Injection Control Program

TO: Water Division Directors, Regions 1-10

FROM: Linda Boornazian, Director
Water Permits Division (MC 4203M)

Steve Heare, Director
Drinking Water Protection Division (MC 4606M)

Over the past several years stormwater infiltration has become an increasingly effective tool in the management of stormwater runoff. Although primary stormwater management responsibilities within EPA fall under the Clean Water Act (CWA), the infiltration of stormwater is, in some cases, regulated under the Safe Drinking Water Act (SDWA) with the goal of protecting underground sources of drinking water (USDWs). Surface and ground water protection requires effective integration between the overlapping programs. This memorandum is a step forward in that effort and is meant to provide clarification on stormwater implementation and green infrastructure, in particular under the CWA, which is consistent with the requirements of the SDWA's Underground Injection Control (UIC) Program.

In April 2007, EPA entered into a collaborative partnership with four national groups (the Association of State and Interstate Water Pollution Control Administrators, the Low Impact Development Center, the National Association of Clean Water Agencies, and the Natural Resources Defense Council) to promote green infrastructure as a cost-effective, sustainable, and environmentally friendly approach to stormwater management. The primary goals of this collaborative effort are to reduce runoff volumes and sewer overflow events through the use of green infrastructure wet weather management practices.

Within the context of this collaborative partnership, green infrastructure includes a suite of management practices that use soils and vegetation for infiltration, treatment, and evapotranspiration of stormwater. Rain gardens, vegetated swales, riparian buffers and porous pavements are all common examples of green infrastructure techniques that capture and treat stormwater runoff close to its source. Green infrastructure management practices typically do not include commercially manufactured or proprietary infiltration devices or other infiltration practices such as simple drywells, which do not provide for pre-treatment prior to infiltration.

The partnership is promoting green infrastructure as an effective approach to stormwater management because these practices are associated with a number of environmental benefits. In addition to reducing and delaying runoff volumes, green infrastructure approaches can also reduce pollutant levels in stormwater, enhance ground water recharge, protect surface water from stormwater runoff, increase carbon sequestration, mitigate urban heat islands, and increase wildlife habitat.

Given the multiple benefits that green infrastructure can provide, EPA and its partners have increased efforts to incorporate green infrastructure techniques into stormwater management strategies nationwide. In recent years, public support for these practices has gradually increased. For more information on green infrastructure, please visit www.epa.gov/npdes/greeninfrastructure.

There are cases where stormwater infiltration practices are regulated as Class V wells under the UIC program, and State and local stormwater managers report that some developers are hesitant to incorporate green infrastructure practices because they fear regulatory approvals will slow the process and increase costs. EPA believes those fears are unfounded and notes that most green infrastructure practices do not meet the Class V well definition and can be installed without regulatory oversight by the UIC Program. However, EPA remains committed to the protection of USDWs and emphasizes the need for UIC program compliance (per 40 CFR 144).

To provide clarification on which stormwater infiltration techniques meet EPA's UIC Class V well definition, EPA's Office of Water has developed the attached "Class V Well Identification Guide." State or Regional stormwater and nonpoint source control programs, developers, and other interested parties are requested to contact the State or Regional UIC Program Director with primary authority for the UIC Class V program when considering the use of practices that have been identified, or potentially identified, as Class V wells. UIC program managers should consider the proximity to sensitive ground water areas when looking at the suitability of stormwater infiltration practices. Depending on local conditions, infiltration without pretreatment may not be appropriate in areas where ground waters are a source of drinking water or other areas identified by federal, state, or local governments as sensitive ground water areas, such as aquifers overlain with thin, porous soils.

Please share this memo and the attached guide with your State and Regional stormwater, nonpoint source control, UIC and other ground water managers, as well as with appropriate green infrastructure contacts. These programs are encouraged to coordinate on stormwater management efforts when sensitive ground water issues arise.

Attachment

Underground Injection Control (UIC) Program Class V Well Identification Guide

This reference guide can be used to determine which stormwater infiltration practices/technologies have the potential to be regulated as "Class V" wells. Class V wells are wells that are not included in Classes I through IV. Typically, Class V wells are shallow wells used to place a variety of fluids directly below the land surface. By definition, a well is "any bored, drilled, driven shaft, or dug hole that is deeper than its widest surface dimension, or an improved sinkhole, or a subsurface fluid distribution system" and an "injection well" is a "well" into which "fluids" are being injected (40 CFR §144.3). Federal regulations (40 CFR §144.83) require all owners/operators of Class V wells to submit information to the appropriate regulatory authorities including the following:

1. Facility name and location
2. Name and address of legal contact
3. Ownership of property
4. Nature and type of injection well(s)
5. Operating status of injection well(s)

For more information on Class V well requirements, please visit http://www.epa.gov/safewater/uic/class5/comply_minrequirements.html. For more information on green infrastructure, please visit http://www.epa.gov/npdes/greeninfrastructure.

The stormwater infiltration practices/technologies in rows A through I below are generally not considered to be wells as defined in 40 CFR §144.3 because typically they are not subsurface fluid distribution systems or holes deeper than their widest surface dimensions. If these practices/technologies are designed in an atypical manner to include subsurface fluid distribution systems and/or holes deeper than their widest surface dimensions, then they may be subject to the Class V UIC regulations. The stormwater infiltration practices/technologies in rows J through K however, depending upon their design and construction probably would be subject to UIC regulations.

	Infiltration Practice/Technology	Description	Is this Practice/Technology Generally Considered a Class V Well?
A	Rain Gardens & Bioretention Areas	Rain gardens and bioretention areas are landscaping features adapted to provide on-site infiltration and treatment of stormwater runoff using soils and vegetation. They are commonly located within small pockets of residential land where surface runoff is directed into shallow, landscaped depressions; or in landscaped areas around buildings; or, in more urbanized settings, to parking lot islands and green street applications.	No.
B	Vegetated Swales	Swales (e.g., grassed channels, dry swales, wet swales, or bioswales) are vegetated, open-channel management practices designed specifically to treat and attenuate stormwater runoff. As stormwater runoff flows along these channels, vegetation slows the water to allow sedimentation, filtering through a subsoil matrix, and/or infiltration into the underlying soils.	No.
C	Pocket Wetlands & Stormwater Wetlands	Pocket/Stormwater wetlands are structural practices similar to wet ponds that incorporate wetland plants into the design. As stormwater runoff flows through the wetland, pollutant removal is achieved through settling and biological uptake. Several design variations of the stormwater wetland exist, each design differing in the relative amounts of shallow and deep water, and dry storage above the wetland.	No.
D	Vegetated Landscaping	Self-Explanatory.	No.
E	Vegetated Buffers	Vegetated buffers are areas of natural or established vegetation maintained to protect the water quality of neighboring areas. Buffer zones slow stormwater runoff, provide an area where runoff can infiltrate the soil, contribute to ground water recharge, and filter sediment. Slowing runoff also helps to prevent soil and stream bank erosion.	No

	Infiltration Practice/Technology	Description	Is this Practice/Technology Generally Considered a Class V Well?
F	Tree Boxes & Planter Boxes	Tree boxes and planter boxes are generally found in the right-of-ways alongside city streets. These areas provide permeable areas where stormwater can infiltrate. The sizes of these boxes can vary considerably.	No.
G	Permeable Pavement	Permeable pavement is a porous or pervious pavement surface, often built with an underlying stone reservoir that temporarily stores surface runoff before it infiltrates into the subsoil. Permeable pavement is an environmentally preferable alternative to traditional pavement that allows stormwater to infiltrate into the subsoil. There are various types of permeable surfaces, including permeable asphalt, permeable concrete and even grass or permeable pavers.	No.
H	Reforestation	Reforestation can be used throughout a community to reestablish forested cover on a cleared site, establish a forested buffer to filter pollutants and reduce flood hazards along stream corridors, provide shade and improve aesthetics in neighborhoods or parks, and improve the appearance and pedestrian comfort along roadsides and in parking lots.	No.
I	Downspout Disconnection	A practice where downspouts are redirected from sewer inlets to permeable surfaces where runoff can infiltrate.	In certain circumstances, for example, when downspout runoff is directed towards vegetated/pervious areas or is captured in cisterns or rain-barrels for reuse, these practices generally would not be considered Class V wells.
J	Infiltration Trenches	An infiltration trench is a rock-filled trench designed to receive and infiltrate stormwater runoff. Runoff may or may not pass through one or more pretreatment measures, such as a swale, prior to entering the trench. Within the trench, runoff is stored in the void space between the stones and gradually infiltrates into the soil matrix. There are a number of different design variations.	In certain circumstances, for example, if an infiltration trench is "deeper than its widest surface dimension," or includes an assemblage of perforated pipes, drain tiles, or other similar mechanisms intended to distribute fluids below the surface of the ground, it would probably be considered a Class V injection well.

	Infiltration Practice/Technology	Description	Is this Practice/Technology Generally Considered a Class V Well?
K	Commercially Manufactured Stormwater Infiltration Devices	Includes a variety of pre-cast or pre-built proprietary subsurface detention vaults, chambers or other devices designed to capture and infiltrate stormwater runoff.	These devices are generally considered Class V wells since their designs often meet the Class V definition of subsurface fluid distribution system.
L	Drywells, Seepage Pits, Improved Sinkholes.	Includes any bored, drilled, driven, or dug shaft or naturally occurring hole where stormwater is infiltrated.	These devices are generally considered Class V wells if stormwater is directed to any bored, drilled, driven shaft, or dug hole that is deeper than its widest surface dimension, or has a subsurface fluid distribution system.

Appendix B
Equations

Equation 2.1: Target Treatment Volumes for Rain Gardens

$$TIV = \frac{A * P\ \ * C}{12} \qquad \textit{Equation 2.1}$$

TIV = Target Infiltration Volume (feet^3)
A = Contributing Area (feet^2)
P = Target Precipitation (inches)
C = Runoff Coefficient per the DDG

The equation selected to define the target treatment volume for rain gardens, infiltration trenches, and soak–away pits is the Target Infiltration volume equation. This equation uses the rational runoff coefficient in combination with terms for area and target precipitation. Target precipitation values used by other states and municipalities vary but generally range from 0.5 to 1 inch of rainfall. The LID manual defines the Target Infiltration volume based on the total 1–year, 24–hour event. The value of 12 in the divisor is a conversion constant (inches to feet). The term Target Infiltration Volume, as used in this manual, is analogous to water quality volume. Additional information on this equation using the term water quality volume can be found at www.stormwatercenter.net.

Equation 2.2: Rain Garden Footprint

$$A_r = \left(\frac{12 * WQ_v}{P_d}\right) * \left(0.26 * I_e^{-0.53}\right) \qquad \textit{Equation 2.2}$$

A_r = Rain Garden Footprint (feet^2)
TIV = Target Infiltration Volume (feet^3)
P_d = Depth of Ponded Water (inches)
I_e = Infiltration Rate of Engineered Soils (inches/hour)

The equation for approximating the required footprint for a rain garden was developed specifically for use in the MOA LID manual. This equation is described in two major terms:

Term 1: $\left(\frac{12 * TIV}{P_d}\right)$

Term 2: $\left(0.26 * I_e^{-0.53}\right)$

In Term 1 of Equation 2.2, the target infiltration volume is divided by the ponded depth (converted into feet by the constant 12) to obtain the approximate area required to contain the Target Infiltration volume at the design depth. Note that this area does not include the additional space required to accommodate sloped excavation from existing ground to the surface of the rain garden.

Term 2 is a reduction factor developed to account for the fact that infiltration is occurring throughout the rain event, and thus, the Target Infiltration volume is never stored entirely at the surface. This term assumes constant infiltration. The term was developed by determining appropriate reduction factors (0.10 to 0.90, unitless) for a range of infiltration rates from 0.3 to 1 inches/hour. Routing and infiltration modeling computations were carried out to identify reduction factors based on two conditions: the reduction in rain garden size could not result in ponding over the designed ponded depth, and the reduction in size could not result in ponded water at the rain garden surface more than 24 hours after the design event. Calculated reduction factors were plotted against their corresponding infiltration rates and the equation presented as Term 2 was developed to describe the plotted relationship.

Note that the reduction factor was calculated according to a 1–year, 24–hour rainfall hyetograph computed with a 1–hour time step. The routing computations were then made on 6–minute intervals using straight line interpolation between ordinates on the 1–hour time step, 1–year, 24–hour rainfall hyetograph.

Equation 2.3: Total Depth for Rain Gardens without Subdrains

$$D_r = \frac{P_d + F_d}{12} + E_d \qquad \textit{Equation 2.3}$$

D_r = Total Depth of Rain Garden Without Subdrain (feet)
P_d = Depth of Ponded Water (inches)
F_d = Freeboard (inches)
E_d = Depth of the Engineered Soils (feet)

The equation for estimating the excavation depth required for a rain garden with no subdrain system has been developed specifically for use in this manual. This equation involves adding the ponded depth to the freeboard (both converted to feet by the constant 12) to the depth of engineered soils.

Equation 2.4: Total Depth for Rain Gardens with Subdrains

$$D_{rs} = \frac{P_d + F_d}{12} + E_d + S_d + 0.005 * L_r \qquad \textit{Equation 2.4}$$

D_{rs} = Total Depth of Rain Garden with Subdrain (feet)
P_d = Depth of Ponded Water (inches)
F_d = Freeboard (inches)
E_d = Depth of the Engineered Soils (feet)
S_d = Depth Required for Subdrain Diameter and Drain Rock (feet)
L_r = Approximate Length of Rain Garden Along the Axis of the Subdrain (feet)

The equation for estimating the excavation depth required for a rain garden with a subdrain system has been developed specifically for use in the MOA LID Manual. This equation is similar to Equation 2.3 but adds terms to account for excavation necessary for the subdrain pipe and rock as well as the depth necessary to accommodate the slope of the subdrain (0.005 ft/ft).

Equation 3.1: Trench Depth

$$D_i = \frac{I * t}{n_s * 12} + 1 \qquad \textit{Equation 3.1}$$

D_i = Trench Depth (feet)
I = Design Infiltration Rate (inches/hour)
t = Retention Time (hours)
n_s = Storage Media Void Ratio

The equation for evaluating trench depth is a slight modification of the trench depth equation presented in the document, *Low Impact Development in Anchorage: Concepts and Criteria*, *Review Copy*, 2004. The equation has been modified to include the conversion factor 12, to eliminate the infiltration rate safety factor, and to include an additional 1 foot to account for the use of a 6–inch layer of sand in the bottom of the trench and a 6–inch cover layer.

Equation 3.2: Trench Footprint

$$A_i = \frac{TIV * 0.66}{n_s * (D_i - 1)} \qquad \textit{Equation 3.2}$$

A_i = Trench Footprint ($feet^2$)
TIV = Target Infiltration Volume ($feet^3$)
n_s = Storage Media Void Ratio
D_i = Trench Depth (feet)

The equation for estimating the trench footprint is a modification of the trench area equation presented in the document, *Low Impact Development in Anchorage: Concepts and Criteria*, *Review Copy*, 2004. The equation has been modified to include the subtraction of 1 foot from the D_i variable because no significant storage will occur within the pea gravel and sand layer at the bottom, and to include the reduction factor 0.72. The reduction factor of 0.72 was developed in a similar fashion as the reduction term in Equation 2.2, to account for the fact that infiltration is occurring thought the rain event, and thus, the target infiltration volume is never stored entirely within the trench.

Unlike Equation 2.2, the reduction factor is expressed as a constant rather than as an exponential relationship with infiltration. This is due to the fact that the local infiltration rate also determines the depth, and thus, the storage capacity of the trench. Trial designs and numerical modeling indicated that for design infiltration rates between 0.3 and 1, a constant reduction factor of 0.72 can be used to reduce the trench footprint area without causing the trench to fill above the design depth or retain water for more than 24 to 48 hours following the design event.

Equation 4.1: Soak–Away Pit Depth

$$D_s = \frac{I * t}{n_s * 12} + 2 \qquad \textit{Equation 4.1}$$

D_s = Soak–Away Pit Depth (feet)
I = Design Infiltration Rate (inches/hour)
t = Retention Time (hours)
n_s = Storage Media Void Ratio

The equation for evaluating soak–away pit depth is a slight modification of the trench depth equation presented in the document, *Low Impact Development in Anchorage: Concepts and Criteria, Review Copy,* 2004. The equation has been modified to include the conversion factor 12, to eliminate the infiltration rate safety factor, and to include an additional 2 feet to account for the use of a 6–inch layer of sand in the bottom of the pit and 1.5–feet over the top of the pit for cover.

Equation 4.2: Soak–Away Pit Footprint

$$A_s = \frac{TIV * 0.66}{n_s * (D_s - 2)} \qquad \textit{Equation 4.2}$$

A_s= Soak–Away Pit Footprint (feet^2)
TIV = Target infiltration Volume (feet^3)
n_s = Storage Media Void Ratio
D_s = Soak–Away Pit Depth (feet)

The equation for estimating soak–away pit depth is similar to Equation 3.2. Equation 4.2 is modified to include the subtraction of 2 feet from the depth term because no significant storage will occur in the 6–inch layer of sand in the bottom of the well and 1.5 feet of cover or inlet construction over the pit.

Equation 5.1: Filter Strip Maximum Discharge Loading

$$q = \frac{1.49}{n} * \left(\frac{Y}{12}\right)^{\frac{5}{3}} * S^{\frac{1}{2}}$$

Equation 5.1

q = Volumetric Discharge per Foot Width (feet^3/second–foot)
Y = Allowable Depth of Flow (inches)
S = Slope of Filter Strip (feet/foot)
n = Manning’s “n” Roughness Coefficient

Equation 5.1, for estimating the maximum discharge loading for filter strips, is the unit width sheet flow variation of Manning's Equation with English units.

Equation 5.2: Maximum Allowable Design Velocity

$$V = \frac{q}{Y/12}$$

Equation 5.2

V = Velocity (feet/second)
q = Volumetric Discharge per Foot Width (feet^3/second–foot)
Y = Maximum Allowable Depth of Flow (inches)

Equation 5.3: Minimum Allowable Filter Strip Width

$$W_{fp} = \frac{A_a * C * 0.5}{q}$$

Equation 5.3

W_{fp} = Width of Filter Strip Perpendicular to Flow Path (feet)
A_a = Area (acres)
C = Runoff Coefficient per the DDG
q = Volumetric Discharge per Foot Width (feet^3/second–foot)

The equation for estimating filter strip width was modified from an equation provided in the *Georgia Stormwater Management Manual*. Equation 5.3 is an approximation of the average peak runoff from a 1–year, 24–hour storm divided by the maximum discharge loading. The value of 0.5 represents the 10–minute peak of a 1–year, 24–hour storm event.

Equation 5.4: Filter Strip Length

$$L_f = \frac{T_t^{1.25} * P^{0.625} * (S * 100)^{0.5}}{3.34 * n}$$

Equation 5.4

L_f = Length of Filter Strip Parallel to Flow Path (feet)
T_t = Travel Time through Filter Strip (minutes)
P = Target Precipitation (inches)
S = Slope of Filter Strip (ft/ft)
n = Manning's "n" Roughness Coefficient

The equation for estimating filter strip length was modified from the *Georgia Stormwater Management Manual (Atlanta Regional Commission, 2001)*. The equation is based on the SCS TR–55 travel time equation presented as Equation 3–3 in the 1986 TR–55 manual (SCS, 1986). The equation has been rearranged to solve for length and include appropriate unit conversions.

Equation D.1: Weir Equation for Flow into Standpipe or Riser

$$Q = N_s * G * C_w * P_s * H^{3/2}$$

Equation D.1

Q = Flow Rate, (feet^3/second)
N_s = Number of Outfall Structures
G = Grate Reduction Factor
C_w = Weir Coefficient
P_s = Perimeter of the Stand Pipe (feet)
H = Head (feet)

Equation D.1 is a variation of the weir equation used to illustrate the design of the overflow structure for rain gardens. The original equation is presented in *Fluid Mechanics with Engineering Applications*, by Robert L. Daugherty and Joseph B. Franzini, 1977. The terms N_s and G have been included to account for the number of stand pipes and the presence of grates on the stand pipes. For the design of LID elements in this manual, the weir coefficient can be assumed as 3.3. Grate reduction factors are available from various grate manufacturers. For preliminary planning purposes, a value of 0.5 may be used.

Equation F.1: Orifice Equation for Discharge through Circular Perforations Under Head

$$Q = N_o * A_o * C_d * \sqrt{2 * g * H}$$

Equation F.1

Q = Flow Rate, (feet^3/second)
N_o = Number of Orifices
A_o = Orifice Opening Area (feet^2)
C_d = Coefficient of Discharge
g = Gravitational Constant (feet/second^2)
H = Head (feet)

Equation F.1 is a variation of the orifice equation used for estimating flow through soak–away pit inlets. The original equation is presented in *Water Resources Engineering* by Ralph A. Wurbs and Wesley P. James, 2002. The term N_o has been included to account for the number of orifices along the inlet pipe. The coefficient of discharge will be 0.62 for all inlets that conform to the design recommendations in this manual. Other published values may be used if inlet design modifications are necessary that result in some condition other than a sharp edge entrance.

Appendix C
Additional Specifications for Rain Gardens

Table C.1 – Vegetation Suitable for Rain Gardens in Anchorage

Information Obtained from *Rain Gardens: A Manual for Homeowners in the Municipality of Anchorage*

This list is periodically updated. The most current list can be obtained at www.anchorageraingardens.com

Latin Name	Common Name	Bloom Time	Bloom Color	Height	Spacing
SHRUBS					
Aronia melanocarpa	Black Chokeberry	na	na	3–5 feet	4 feet
*Cornus sericea	Red–twig Dogwood	Early Summer	white	5–8 feet	5 feet
Cornus sericea ‘flaviramea’	Yellow–twig Dogwood	Early Summer	white	5–8 feet	5 feet
*Viburnum edule	Highbush Cranberry	Spring	white	4–8 feet	3 feet
**Willow	Willow	Spring	varies		
Myrica gale	Sweet Gale		white	3–4 feet	3 feet
PERENNIALS					
**Aquilegia	Columbine	All Summer	varies	6–36 in	12 in
*Aruncus dioicus	Goat’s Beard	Early Summer	ivory white	5 feet	24 in
**Aconitum delphinifolium	Monkshood		purple	3–4 feet	2 feet
*Dodecatheon pulchellum	Shooting Star	Late spring	pink	12 in	12 in
*Geranium erianthum	Wild Geranium	Late spring	blue/violet	24–36 in	18–24 in
*Oplopanax horridus	Devil’s Club	Spring	white	3–10 feet	2–3 feet
*Athyrium felix–femina	Lady Fern			30–36 in	24–30 in
*Frittilaria camschatcensis	Chocolate Lily	Spring	purple/brown	18 in	5–6 in
**Dodecatheon	Shooting Star	Spring	violet	12–18 in	12 in
*Dryopteris dilitata	Wood Fern			30–36 in	24–30 in
*Geranium erianthum	Cranesbill.Geranium	Spring/Summer	purple/white	12–24 in	12 in
Hemerocallis ‘stella.de.oro’	Stella de Oro Daylily	All Summer	yellow	12 in	12 in
*Iris setosa	Alaska Wild Iris	Early Summer	purple/white	18–30 in	18 in
Iris psuadacoris	Iris	Early Summer	yellow	18–30 in	18 in
*Matteuccia struthiopteris	Ostrich Fern			36–48 in	24–30 in
*Mertensia	Bluebells	Spring	blue/purple	18–30 in	18 in
*Myosotis alpestris	Forget–Me–Not	Spring/Summer	blue/pink	4–12 in	12 in
**Polemonium	Jacob’s Ladder	Summer	blue	12–36 in	12 in
Thalictrum	Meadow Rue	Summer	pink/white/purple	36–48 in	18 in
Trollius	Globeflower	Spring/Summer	yellow/orange	24–36 in	12 in
Filipendula	Meadowsweet	Late summer	white/pink	18–60 in	24–36 in
Ligularia (x2) – stenocephala/przewalskii	The Rocket/Ligularia	Late Summer	yellow/gold	36–60 in	24 in
GRASSES AND SEDGES					
*Carex gmelini	Native Sedge			36 in	30 in
Miscanthus sinesis	Red Flame Grass	Late Summer	Silver/white	3–4 feet	24 in
**Elymus mollis	Wild Rye			1–3 feet	12 in
*Deschampsia cespitosa	Tufted Hair Grass	Late Summer	Gold/Silver/Purple/Green	2–3 feet	1–2 feet

*Indicates Native Plant Species ** Indicates Native or Non–Native Plant Species na – not applicable

Additional Guidance for the Specification of Engineered Soils for Rain Gardens

Note: The following *guidance for specifying an engineered soil for a rain garden has been adapted from the Puget Sound Action Team, Low Impact Development Technical Guidance Manual for Puget Sound. The information provided in Subsection 2.1.3.a is a starting point for soil mix specifications. The information below provides additional guidance.*

The following bulleted list is intended to assist designers in specifying an engineered soil mix for use in a rain garden. Soil specifications may vary slightly depending on site characteristics and related design considerations.

- The final soil mix (including compost and soil) should have a long–term hydraulic conductivity of approximately 1.0 inch/hour according to ASTM Designation D 2434 (Standard Test Method for Permeability of Granular Soils) at 80% compaction per ASTM Designation D 1557 (Standard Test Methods for Laboratory Compaction Characteristics of Soil Using Modified Effort). Note that infiltration rate and hydraulic conductivity are assumed to be approximately the same in a uniform mix soil.

- The final soil mixture should have a minimum organic content of 10% by dry weight per ASTM Designation D 2974 (Standard Test Method for Moisture, Ash and Organic Matter of Peat and Other Organic Soils).

- The final soil mixture should be tested by an independent laboratory prior to installation for fertility, micronutrient analysis, and organic material content. Soil amendments per laboratory recommendations (if any) should be uniformly incorporated for optimum plant establishment and early growth.

- The clay content of the final soil mix should be less than 5%.

- The pH for the soil mix should be between 5.5 and 7.0. If the pH falls outside of the acceptable range, it may be modified with lime to increase the pH or iron sulfate plus sulfur to lower the pH. The lime or iron sulfate must be mixed uniformly into the soil prior to use in the rain garden.

- Soil mix should be uniform and free of stones, stumps, roots, or other similar material greater than 2 inches in diameter.

Unless laboratory analysis indicates otherwise, engineered soils are to be assigned a design infiltration rate of 1.0 inches/hour during design efforts. This value is consistent with a moderately high saturated hydraulic conductivity (MOA, 2007a).

Appendix D
Rain Garden Design Example

D. Design Example – Rain Garden

This section presents the design process for a rain garden to treat runoff from the parking area of the site described below.

<u>Site Description</u> – *A 1.8–acre lot in midtown Anchorage is to be redeveloped. The existing lot contains an old warehouse and a large parking area. The redeveloped lot will include a three–story office building a landscaped garden and a parking area. The new parking lot will contain approximately 0.75 acres of paved surface. Rain gardens have been identified as a good alternative for treating runoff from the parking area, since a rain garden can be designed to serve as the required site landscaping as well. The preliminary site design has included an area within the center of the parking facility to place the rain garden.*

In the following subsections a preliminary site evaluation and a preliminary design are presented for the design of a rain garden for this site. Following these sections, a final design is discussed and a conceptual drawing of the final design is presented.

D.1 Example Preliminary Site Evaluation – Rain Garden

To conduct the preliminary site evaluation, the preliminary site evaluation checklist (Table 2) provided in Section 2 has been used. To fill out the preliminary site evaluation checklist, the following reference materials were required:

- The draft preliminary site plans,
- Anchorage Water and Wastewater Utility Maps,
- Local topographic maps, and
- The site geotechnical report.

Prior to conducting the preliminary site evaluation, it was noted that due to the close proximity of the rain garden to the parking lot subgrade, the use of a 30–mil polyethylene liner is required. This information was incorporated into the preliminary site evaluation.

The completed preliminary site evaluation checklist is presented as Table D.1. The information presented in Table D.1 indicates that the site is likely suitable for the use of a rain garden to treat parking lot runoff. However, review of the geotechnical report indicates that the groundwater table is located at a depth of 9 feet below grade. Based on the site evaluation, it was not certain that it would be possible to maintain the minimum separation distance between the bottom of the lined rain garden and the groundwater table (2 feet for lined rain gardens). The groundwater table would limit the total depth of the rain garden to no more than 7 feet. This has been noted and is to be addressed during the preliminary design.

Table D.1 – Rain Garden – Preliminary Site Evaluation Checklist

Site Location: 1112 W 100th Street **Evaluated by:** William H. Seward

Date: 8/24/2012

Consid-erations	Applies to Lined Rain Garden?	Applies to Rain Gardens with Subdrains?	Requirement/ Recommendation	Site Conditions /Notes	Pass /Fail	Data Source
Soil Infiltration	Y	N	Measured soil infiltration rate must be between 0.3 and 8 in/hr.	The lowest soil infiltration rate in the area being considered for the rain garden is 1.0 in/hr.	Pass	Geotechnical Report
Proximity to Class A and B Wells	N	Y	Rain garden must be located at least 200 feet from Class A and B wells.	There are no Class A or B wells within 200 feet of the site.	Pass	AWWU Mapping
Proximity to Class C Well	N	Y	Rain garden must be located at least 100 feet from Class C wells.	There is not a Class C well within 100 feet of the site.	Pass	AWWU Mapping
Proximity to Surface Waters	N	Y	Rain garden should be located at least 100 feet from surface waters.	There are no surface waters within 100 feet of the site.	Pass	Topo Map
Depth to Seasonal High Groundwater Level	Y	Y	4 feet or more below the top of an unlined rain garden and 2 feet or more below the top of a lined rain garden.	Groundwater is 9 feet below the proposed grade near the rain garden.	Investi gate Further	Geotechnical Report
Depth To Bedrock	N	Y	Bedrock must be 3 foot or more below the bottom of a rain garden.	Bedrock was not encountered; drilling went to a depth of 15 feet below grade.	Pass	Geotechnical Report
Proximity to Building Foundations	N	Y	Rain garden must be located outside of the zone of influence or at least 20 feet from building foundations.	The garden will be located approximately 60 feet from the nearest foundation.	Pass	Preliminary Site Plans
Proximity to Road Subgrades	N	Y	Rain garden must be located outside of the zone of influence or at least 20 feet from road subgrades.	Rain garden will be located within parking lot. A liner will need to be used.	Pass	Preliminary Site Plans
Runoff Source	Y	Y	Rain garden is not to receive runoff containing industrial pollutants.	Runoff is from a parking lot.	Pass	Preliminary Site Plans
Contributing Area	Y	Y	The contributing area must be less than 5 acres.	Area contributing to the garden is 0.75 acres.	Pass	Preliminary Site Plans
Available Area Slope	Y	Y	The slope must be less than or equal to 5%.	Proposed site slopes are 0.5%.	Pass	Preliminary Site Plans
Available Area	Y	Y	The area available for treatment must be at least 10% of the total contributing area.	Adequate space is available.	Pass	Preliminary Site Plans
Down Gradient Slope	N	Y	Average slope of adjacent down gradient property must be less than 12%.	The grade of the adjacent downgradient lot is less than 2%.	Pass	Topo Map

D.2 Example Preliminary Design – Rain Garden

During the preliminary design process the minimum design considerations presented in Subsection 2.1.2 must be addressed. In order to conduct the preliminary rain garden design, the preliminary design calculation table (Table 3) presented in Section 2 has been used. The completed preliminary design calculations are presented in Table D.2.

In Step 1 of the preliminary design calculations, the runoff coefficient has been obtained from the DDG. The slope of the parking lot is less than 2% resulting in a runoff coefficient of 0.85. The calculation in Step 1 indicates that the rain garden will need to accommodate a volume of approximately 2,546 $feet^3$ of runoff.

In Step 2 of the preliminary design calculations the maximum ponding depth is selected to limit the amount of required area for the rain garden. Also the minimum horizontal to vertical side slope is used to minimize the area required for the rain garden. The resulting area required to contain the rain garden is 993 $feet^2$. It was determined by the design team that a square garden placed in the center of the parking lot would be preferable. Thus, in Step 2 the dimensions of the rain garden were calculated to be approximately 32 feet by 32 feet.

Because a liner must be incorporated into the design, Step 3b was selected to perform the calculation for the approximate depth of the rain garden. The minimum ponded depth and minimum soil depth were both assumed to limit the total depth of the rain garden. The depth required for the subdrain was assumed to be 1.75 feet. This accounts for a 3–inch layer of drain rock under the subdrain, an 8–inch diameter subdrain, a 6–inch layer of drain rock above the subdrain, and a 4–inch layer of pea gravel above the drain rock. The resulting estimated total depth of the rain garden is 5.2 feet.

Table D.2 – Rain Garden Preliminary Design

Site Location: 1112 W 10th Street		**Evaluated by:** Don Sheldon	
Date: 8/24/2012			
Step 1: Calculate the Target infiltration Volume, TIV			**Notes**
Contributing Area, A	32670	(ft^2)	Approximate Parking Lot Area
Target infiltration Rainfall, P	1.1	(in)	Set Value
Runoff Coefficient, C	0.85		Per DDG
TIV = A*P*C/12 =	2546	(ft^3)	Using Equation 2.1
***Step 2: Calculate the Required Rain Garden Footprint Area**			
TIV (from Step 1)	2546	(ft^3)	
Depth of Ponded Water, P_d	8.0	(in)	Maximum of 8 inches
Design Infiltration Rate, I_e (or I, see Subsection 2.1.2.c)	1.0	(in/hr)	1.0 for engineered soils
$A_r = (TIV*12/P_d)(0.26*I_e^{-0.53}) =$	993	(ft^2)	Using Equation 2.2
Approximate Width, W_r $W_r=A_r/L_r=$	32	(ft)	Assume a Square Garden
Approximate Length, L_r $L_r=A_r/W_r=$	32	(ft)	
****Step 3a: Approximate Rain Garden Depth, without Subdrain**			Not applicable
P_d (From Step 2)		(in)	
Freeboard Depth, F_d		(in)	Minimum of 2 inches
Depth of Engineered Soils, E_d		(ft)	Minimum of 2.5 feet Maximum of 4 feet
$D_r = (P_d+F_d)/12+E_d =$		(ft)	Using Equation 2.3
OR			
*****Step 3b: Approximate Rain Garden Depth**			
P_d (From Step 2)	8.0	(in)	
Freeboard Depth, F_d	2	(in)	Minimum of 2 inches
Depth of Engineered Soils, E_d	2.5	(ft)	Minimum of 2.5 feet
Minimum Subdrain Depth, S_d	1.75	(ft)	
L_r (From Step 3)	32	(ft)	
$D_{rs} = (P_d+F_d)/12+E_d+S_d+(0.005*L_r) =$	5.2	(ft)	Using Equation 2.4

Note: *See Appendix C for guidance on selecting a value for I_e. For unlined rain gardens without subdrains, substitute variable I_e with I, the design infiltration rate for the native soil.

**Subdrain and / or underground overflow control system <u>will not</u> be used.

***Subdrain and / or underground overflow control system <u>will</u> be used.

The results of the preliminary site evaluation and the preliminary design indicate that the site is a suitable candidate for the use of a rain garden to treat storm water runoff from the parking lot. Thus, final design efforts are warranted.

D.3 Example Final Design – Rain Garden

To develop the final design based on the dimensions calculated in the preliminary design, the minimum factors presented in Subsection 2.1.3 were addressed. In real world applications, the final design of a rain garden is likely to include slight adjustments in geometry and will likely include site related engineering considerations specific to the particular project. For the sake of this example, the dimensions calculated in the preliminary design have been directly applied to the final design.

Engineered Soils – The specifications for the engineered soils are based on the requirements presented in Subsection 2.1.3.a, the guidance provided in Appendix C, and the geotechnical investigation for the site. The geotechnical investigation for the site indicates that the native soils are primarily loamy sand. Thus, approximately 60% of the excavated native soil will be set aside and mixed with compost to provide engineered soil for the rain garden.

Rain Garden Plants – The specification for the rain garden plants is based on the guidance provided in Subsection 2.1.3.b and the listing of suggested rain garden plants provided in Appendix C. The interior of the garden is to be planted with Red Twig Dogwood and Willow. The interior of the garden will also be planted with Native Sedge grass.

Subdrain System – The subdrain system has been designed according to the guidance provided in Subsection 2.1.3.c. The subdrain system includes a branched network of 8–inch slotted PVC pipes. The PVC drainpipe sits atop a 3–foot wide bed of drain rock that is 3 inches thick. The drainpipe is overlaid with drain rock to a depth of 6 inches above the pipe. The drain rock is covered with 4 inches of pea gravel to reduce the likelihood of clogging. Note that backflow preventers have been included to keep the rain garden subsoil from becoming saturated when the storm drains become surcharged.

Rain Garden Liner – The rain garden is lined with 30–mil polyethylene plastic with welded joints. The liner is keyed into the sides of the rain garden to prevent it from slipping downward over the course of time.

Overflow Structure – The overflow structure selected for the rain garden consists of two standpipes located along the center axis of the rain garden. The standpipes are connected to an underground storm drain that has been sized for the 100–year, 24–hour storm according to the Rational Method and guidance in the DDG. In this case, a 100–year peak flow rate of 0.7 feet^3/second was estimated based on a time of concentration of 15 minutes, an intensity of 1.1 inches per hour, and a weighted C value of 0.85 inch. The standpipes were initially sized to meet the diameter of the underground conveyance system for the sake of convenience. The standpipe sizes were then checked for the maximum overtopping depth of 2 inches using the following inlet weir equation below.

$$Q = N_s * G * C_w * P_s * H^{3/2}$$

Equation D.1

Q = Flow Rate, ($feet^3$/second)
N_s = Number of Structures, 2
G = Grate Reduction Factor, 0.5
C_w = Weir Coefficient, 3.3
P_s = Perimeter of the Stand Pipe (feet), 7.85
H = Head (feet), 0.167

Thus, when the ponded depth of the rain garden is 2 inches above the top of the standpipes, the standpipes will be conveying 1.8 $feet^3$/second of runoff. This exceeds the peak runoff from the 100–year 24–hour storm. The 30–inch diameter standpipes are adequately sized for flood control according to the DDG.

A conceptual drawing of the rain garden resulting from this design effort is presented in Figure D.1.

This rain garden will significantly reduce the runoff peak that exits the site during large rain events. It also provides treatment for the full TIV.

Figure D.1 – Rain Garden Design Example Page 1

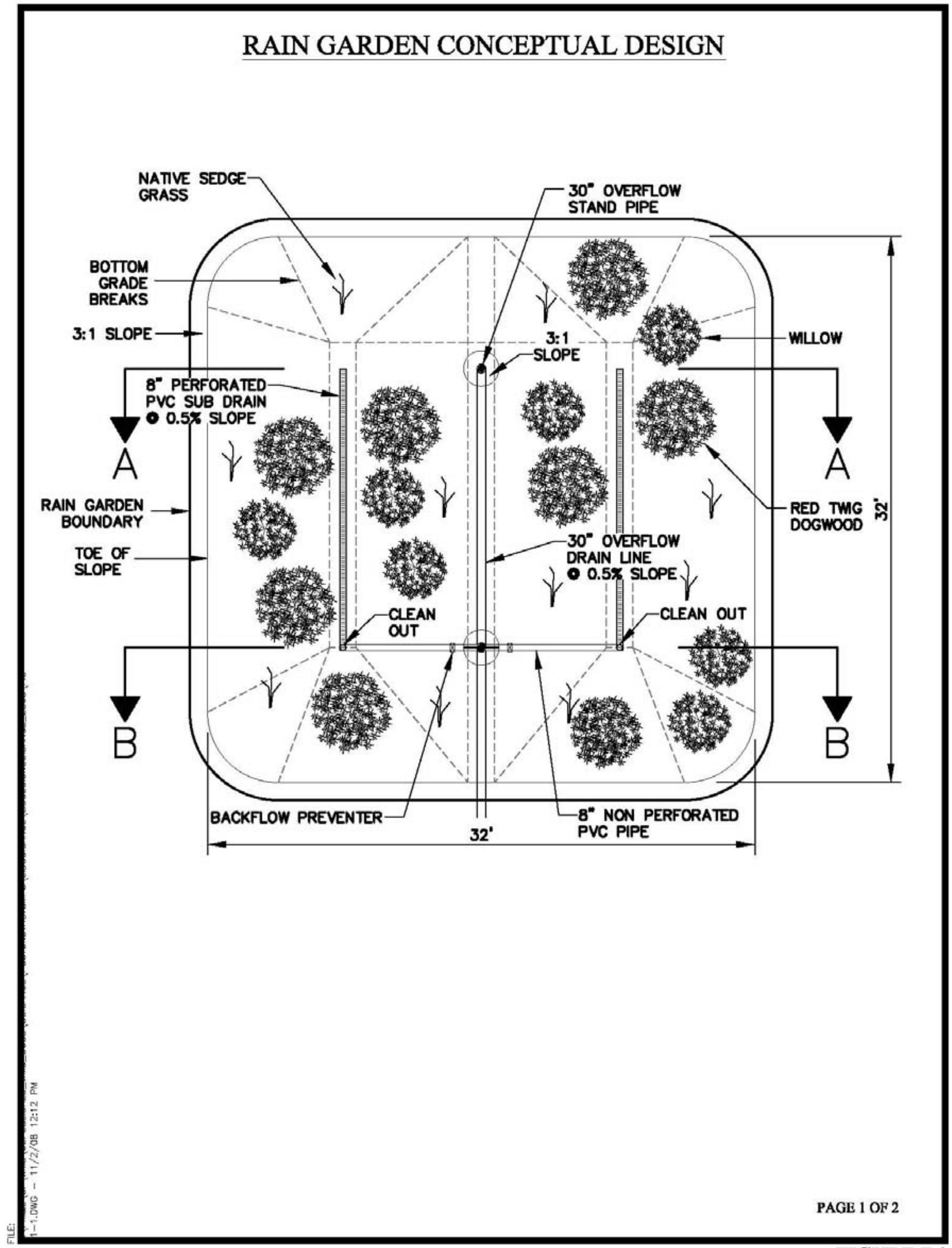


FIGURE D1

Figure D.1 – Rain Garden Design Example Page 2

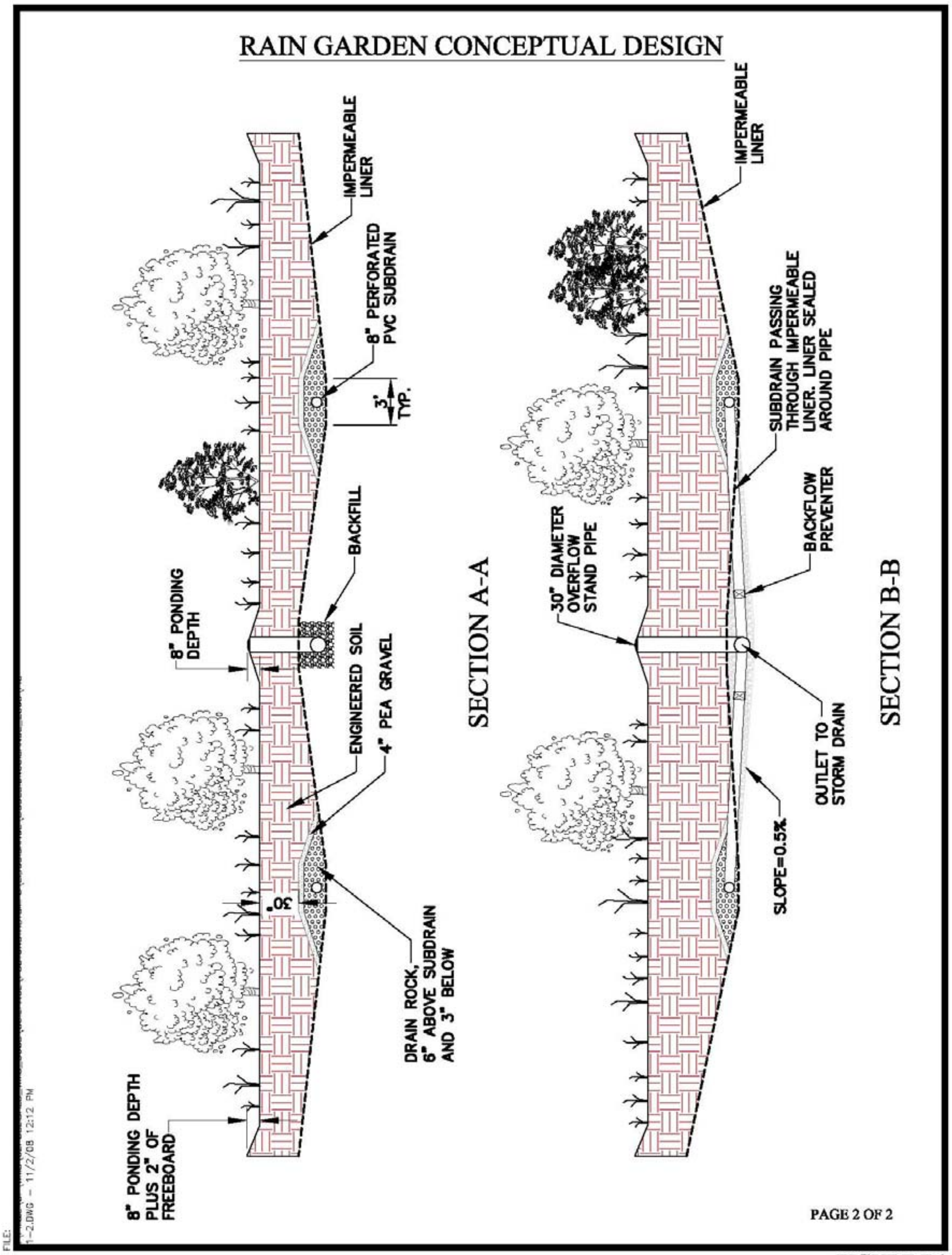


FIGURE D1

Appendix E
Infiltration Trench Design Example

E Design Example – Infiltration Trench

This section presents the design process for an infiltration trench to infiltrate parking lot runoff from a portion of the site described below.

Site Description: *An 8–acre tract of relatively flat land is to be developed into an apartment complex. The complex will include four separate three–story apartment buildings each containing 12 two–bedroom apartments. The development will include a 32 space parking lot for each building as well as open green space to be used for landscaping and recreation. In order to qualify as a "simple large project" under the DDG, the developer would like to incorporate LID elements to infiltrate runoff from the base 1–year, 24–hour storm event. Infiltration trenches have been identified as a potential option to infiltrate runoff from the parking areas. Each parking lot is approximately 65 feet by 140 feet. The available space for an infiltration trench at each parking lot is approximately 40 by 65 feet.*

In the following subsections, a preliminary site evaluation and a preliminary design are presented for an infiltration trench for a single parking lot. Following these sections, a final design is discussed and a conceptual drawing of the final design is presented.

E.1 Example Site Evaluation – Infiltration Trench

In order to conduct the preliminary site evaluation, the preliminary site evaluation checklist (Table 4) provided in Section 3 has been used. To fill out the preliminary site evaluation checklist, the following reference materials were required:

- The draft preliminary site plans,
- Anchorage Water and Wastewater Utility Maps,
- Local topographic maps, and
- The site geotechnical report.

The completed preliminary site evaluation checklist is presented as Table E.1. The information presented in Table E.1 indicates that that the site is likely suitable for the use of an infiltration trench to treat parking lot runoff. However, review of the preliminary site plan indicates that the infiltration trenches will be limited to a length of no more than 65 feet due to the parking lot layout. The preliminary site evaluation also indicates that groundwater is at a depth of 8 feet, thus limiting the allowable depth of an infiltration trench to no more than 4 feet.

Other than these considerations, the site is a good candidate for the use of an infiltration trench to treat runoff from the parking lot. The limitations in the possible trench dimensions have been noted and are addressed during the preliminary design.

Table E.1 – Infiltration Trench – Preliminary Site Evaluation Checklist

Site Location: 1112 W 100th Street **Evaluated by:** Leonhard Seppala

Date: 8/24/2010

Considerations	Requirement/Recommendation	Site Conditions/Notes	Pass/Fail	Data Source
Soil Infiltration Rate	Measured soil infiltration rate must be between 0.3 and 8 in/hr.	The lowest soil percolation rate in the area being considered for the trench is 0.3 in/hr.	Pass	Geotechnical Report
Proximity to Class A and B Wells	Trench must be located at least 200 feet from Class A and B wells.	There are no Class A or B wells within 200 feet of the site.	Pass	AWWU Mapping
Proximity to Class C Well	Trench must be located at least 100 feet from Class C wells.	There is a Class C well within 100 feet of the site. The trench will be located more than 100 feet from the well.	Pass	AWWU Mapping
Proximity to Surface Waters	Trench should be located at least 100 feet from surface waters.	There are no open surface waters within 200 feet of the site.	Pass	Topo Map
Depth to Seasonal High Groundwater Level	Groundwater must be 4 feet or more below the bottom of the trench.	Groundwater is 8 feet below the surface. Need to know how deep trench will be.	Investigate Further	Geotechnical Report
Depth To Bedrock	Bedrock must be 3 feet or more below the bottom of the trench.	Bedrock is at a depth of 10 ft. Need to know how deep the trench will be.	Investigate Further	Geotechnical Report
Proximity to Building Foundations	Trench must be located outside of the zone of influence or at least 20 feet from building foundations.	Trenches will be located more than 20 feet from building foundations	Pass	Draft Preliminary Site Plans
Proximity to Road Subgrades	Trench must be located at least 20 feet from road subgrades.	It is anticipated that there will be adequate room to place the trenches a minimum of 20 feet from road subgrades.	Pass	Draft Preliminary Site Plans
Runoff Source	Infiltration trench is not to receive runoff containing industrial pollutants.	Parking Area	Pass	Draft Preliminary Site Plans
Contributing Area	The contributing area must be less than 3 acres.	The approximate contributing area is 0.2 acres.	Pass	Draft Preliminary Site Plans
Available Area Slope	Available area slope must be less than or equal to 5%.	The average slope of the contributing area is 0.5%.	Pass	Draft Preliminary Site Plans
Available Area	The area available for treatment must be at least 20% of the total catchment area.	Approximately 40% of the total site area will consist of open space for lawns and landscaping.	Pass	Draft Preliminary Site Plans
Down Gradient Slope	Down gradient slope must be less than 12%.	The adjacent properties are also gently sloping.	Pass	Site Visit/Topo Map

E.2 Example Preliminary Design – Infiltration Trench

In order to conduct the preliminary infiltration trench design, the table (Table 5) presented in Section 3 has been used. The completed preliminary design calculations are presented in Table E.2.

In Step 1 of the preliminary design calculations, the runoff coefficient, 0.85, has been obtained from the DDG. The calculation in Step 1 indicates that the infiltration trench will need to accommodate a volume of approximately 709 $feet^3$ of runoff.

In Step 2, the typical void ratio was assumed. A retention time of 48 hours was assumed. The resulting trench depth is 4 feet. This depth will still accommodate the minimum separation distance between the bottom of the infiltration trench and the groundwater table.

In Step 3, the bottom area of the trench is calculated based on values collected and calculated in Steps 1 and 2. The required bottom area of the trench is 390 $feet^2$.

In Step 4, the length and width of the trench is established. The infiltration trench will receive sheet flow from the parking lot along the side that is 35 feet long. Thus, the length of the infiltration trench has been set in Step 4 to be 35 feet. The resulting required width of the infiltration trench (not counting the pretreatment area) is 11.1 feet.

In Step 5, the width required for an infiltration trench receiving runoff from a single side and from both sides is calculated. Note that the infiltration trench will only receive runoff from one side. The resulting width is 31.1 feet.

In Step 6, the length selected in Step 4 is recorded with the width calculated in Step 5 for infiltration trench receiving runoff from one side. These values represent the required area for the infiltration trench.

The results of the preliminary site evaluation and the preliminary design indicate that the site is a suitable candidate for the use of an infiltration trench to treat storm water runoff. Thus, final design efforts are warranted.

Table E.2 – Infiltration Trench Preliminary Design

Site Location: 1112 W 100th Street			**Evaluated by:** Don Sheldon
Date: 8/24/2012			
Step 1: Calculate the Target Infiltration Volume, TIV			**Notes**
Contributing Area, A	9100	(ft^2)	Total Contributing Area
Target Infiltration Rainfall, P	1.1	(in)	Set Value
Runoff Coefficient, C	0.85		Per DDG
TIV = A*P*C/12 =	709	(ft^3)	Using Equation 2.1
Step 2: Calculate the Depth of the Trench, D_i			Must be between 4 and 10 feet
Void Ratio, n_s	0.4		0.4 is Typical of 1.5 to 3 in. Stone
Design Infiltration Rate, I	0.3	(in/hr)	Based on site investigation (Subsection 1.4.1 and DDG)
Retention Time, t	48	(hr)	Must be 24 to 48 hours
$D_i = (I*t)/(n_s*12) + 1 =$	4	(ft)	Using Equation 3.1
Step 3: Calculate the Bottom Footprint of the Trench			
TIV (from Step 1)	709	(ft^3)	
n_s (from Step 2)	0.4		
D_i (from Step 2)	4	(ft)	
$A_i = (TIV *0.66)/(n_s*(D_i – 1)) =$	390	(ft^2)	Using Equation 3.2
Step 4: Establish the Trench Length and Width			Minimum Recommended Ratio is 3L:1W
Set Trench Length, L_i	35.0	(ft)	
Or			
Set Trench Width, W_i		(ft)	Maximum Width is 25 feet
Then Calculate Either			
$W_i=A_i/L_i$	11.1	(ft)	Maximum Width is 25 feet
Or			
$L_i=A_i/W_i$		(ft)	
Record Final L_i and W_i Values			
$L_i=$	35.0	(ft)	
$W_i=$	11.1	(ft)	
Step 5: Account for Pretreatment			
Filter Strip Width, W_f	20.0	(ft)	Minimum Recommended Width is 20 feet
If Receiving Flow From Both Sides			
Total Width (W_{if1}), $W_{if1} = W_i + 2*W_f =$		(ft)	
Or, If Receiving Flow From One Side			
Total Width (W_{if2}), $W_{if2}=W_i + W_f =$	31.1	(ft)	Receiving flow from a single side
Step 6: Required Length and Width for Trench and Filter Strip			
L_t (from Step 4) =	35.0	(ft)	
Appropriate Total Width (from Step 5) =	31.1	(ft)	Receiving flow from a single side

E.3 Example of Final Design – Infiltration Trench

In order to develop the final design based on the dimensions calculated in the preliminary design, the minimum factors presented in Subsection 3.1.4 were addressed. In real world applications, the final design of an infiltration trench is likely to included slight adjustments in geometry as well as site related engineering considerations specific to the particular project.

Filter Fabric – To reduce the likelihood of clogging and piping, a filter fabric has been specified with a flow rate to closely match the surrounding soils' infiltration rate of 0.3 feet/sec. The fabric is placed between the storage media and the trench walls and overlaps by 1–foot long seams. It is placed as a barrier beneath the 6 inches of top material. Filter fabric is placed between the top layer and the storage media. The fabric will be keyed into the sides of the trench walls.

Design of the Overflow Structure – The overflow structure of choice for this particular example is the use of standpipes at the trench boundaries. These structures are connected to a storm drain trunk line that runs down the adjacent road. The standpipes were initially sized to meet the diameter of the underground conveyance system (12 inches) for the sake of convenience. The standpipe sizes were then checked for an overtopping depth of 3 inches as depths greater than 3 inches would result in stormwater spilling beyond the limits of the trench and overflow structures.

$$Q = N_s * G * C_w * P_s * H^{3/2}$$

Equation D.1

Q = Flow Rate, (feet^3/second)
N_s = Number of Structures, 2
G = Grate Reduction Factor, 0.5
C_w = Weir Coefficient, 3.3
P_s = Perimeter of the Stand Pipe (feet), 3.14
H = Head (feet), 0.25

When the ponded depth of the infiltration trench is 3 inches above the top of the standpipes, the standpipes will be conveying 1.3 feet^3/second of runoff. This exceeds the peak runoff from the 100–year 24–hour storm according to a rational calculation. The 12–inch diameter standpipes are therefore adequately sized for flood control according to the DDG.

Top Layer – The material selected for this application is washed pea gravel. The pea gravel will be laid in a 6–inch layer on top of the filter fabric that overlies the storage media.

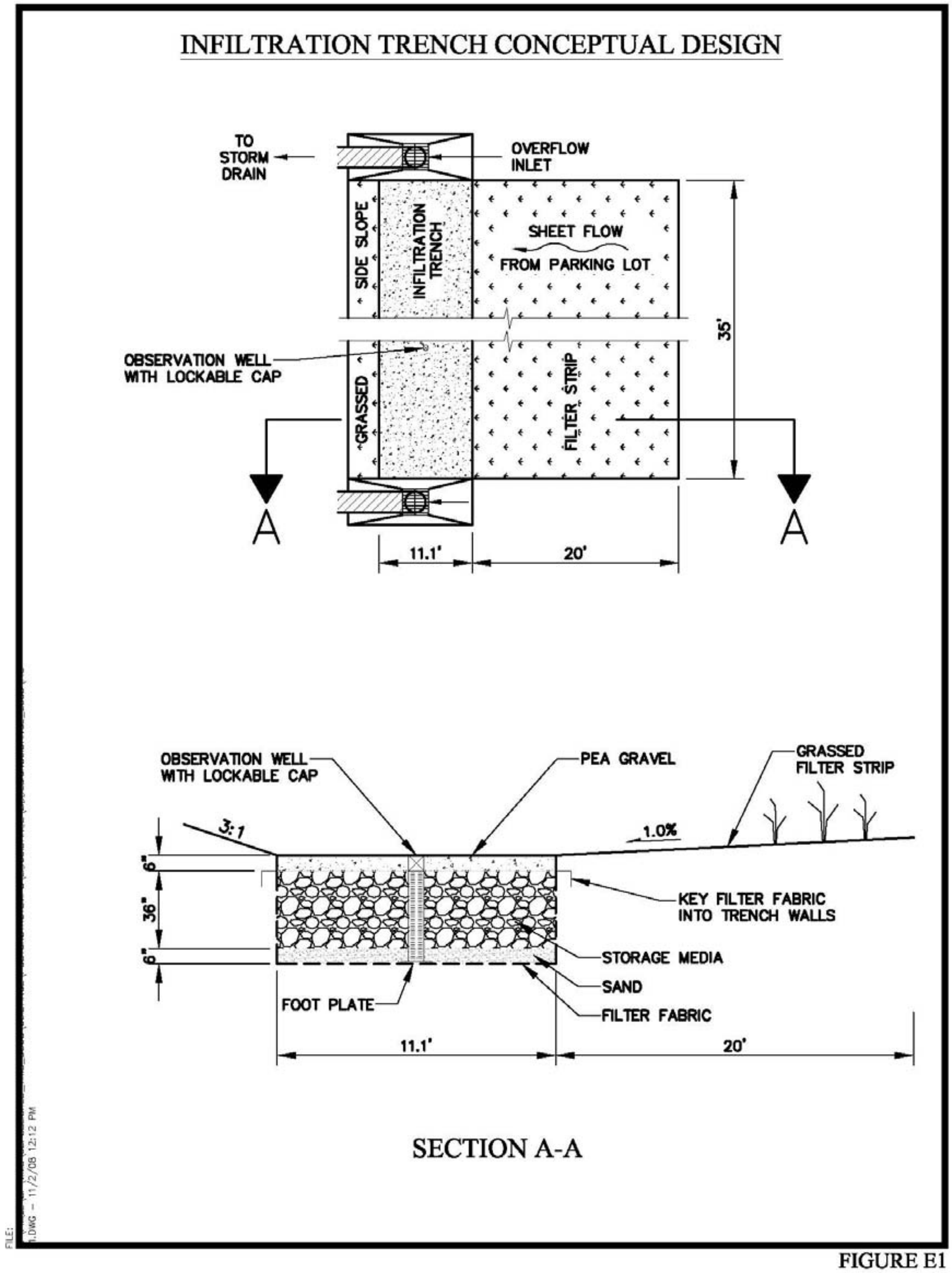
Bottom Layer – The bottom layer consists of washed filter sand.

Grading – The site grading plan has been completed so that the parking lot will sheet drain across the filter strip to the infiltration trench. The trench has been graded to be completely level along the top and bottom.

Observation Well – The infiltration trench includes two 6 inch observation wells that can be seen in Figure E.1.

A drawing of the infiltration trench is presented as Figure E.1. Note that in this design, the area required for the structure is slightly larger than the area estimated using the preliminary design calculations. This is due to the use of overflow inlets on either end of the infiltration trench and the 3:1 (horizontal: vertical) side slope.

Figure E.1 – Infiltration Trench Design Example



Appendix F
Soak–Away Pit Design Example

F. Design Example – Soak–Away Pit

This section presents the design process for a soak–away pit to infiltrate rooftop runoff from a portion of the site described below.

Site Description: *A 4–acre tract of steeply to moderately sloped land is to be developed to include four small condominium buildings. In order to qualify as a "simple small project" under the DDG, the developer would like to incorporate soak–away pits into the site to infiltrate rooftop runoff from the 1–year, 24–hour storm. The rooftops of the four buildings amount to an area of approximately 21,600 feet^2. Each rooftop is to have three individual downspouts each discharging to a single soak–away pit. Each soak–away pit must be designed to accommodate runoff from approximately 1,800 feet^2 of rooftop.*

In the following subsections, a preliminary site evaluation and a preliminary design are presented for the design of a single soak–away pit for this site. Following these sections, a final design is discussed and a conceptual drawing of the final design is presented.

F.1 Example Site Evaluation – Soak–Away Pit

In order to conduct the preliminary site evaluation, the preliminary site evaluation checklist (Table 6) provided in Section 4 has been used. To fill out the preliminary site evaluation checklist, the following reference materials were required:

- The draft preliminary site plans,
- Anchorage Water and Wastewater Utility Maps,
- Local topographic maps, and
- The site geotechnical report.

The completed preliminary site evaluation checklist is presented as Table F.1. The information presented in Table F.1 indicates that the site is likely suitable for the use of a soak–away pit to treat rooftop runoff. However, review of the geotechnical report indicates that bedrock is located at a depth of 10.5 feet. Based on the site evaluation, it was not certain that it would be possible to maintain the minimum separation distance between the bottom of the soak–away pit and the top of the bedrock. The bedrock would limit the depth of the soak–away pit to no more than 7.5 feet. This was noted and addressed during the preliminary design.

Table F.1 – Soak–Away Pit – Preliminary Site Evaluation Checklist

Site Location: 1112 W 100th Street **Evaluated by:** Don Sheldon

Date: 8/24/2007

Considerations	Requirement/Recommendation	Site Conditions/Notes	Pass/Fail	Data Source
Soil Infiltration	Measured soil infiltration rate below the soak–away pit must be between 0.3 and 8 inches/hour.	The lowest soil infiltration rate in the area being considered for a pit is 0.6 inches/hour.	Pass	Geotechnical Report
Separation Distance From Class A and B Wells	The soak–away pit must be separated at least 200 feet from Class A and B wells.	There are no Class A or B wells within 200 feet of the site.	Pass	AWWU Utility Maps
Proximity to Class C Wells	The soak–away pit must be separated at least 100 feet from Class C wells.	There are no Class C Wells within 100 feet of the site.	Pass	AWWU Utility Maps
Proximity to Surface Waters	The soak–away pit should be separated at least 100 feet from surface waters.	There are no surface waters within 100 feet of the site.	Pass	Site Visit/Topo Map
Depth to Seasonal High Groundwater Level	Groundwater must be 4 feet or more below the bottom of the pit.	Groundwater was not located within 10.5 feet below the ground surface.	Investigate Further	Geotechnical Report
Depth To Bedrock	Bedrock must be 3 feet or more below the bottom of the pit.	Bedrock is at a depth of 10.5 feet.	Investigate Further	Geotechnical Report
Proximity to Building Foundations*	The pit must be located outside of the zone of influence or at least 20 feet from building foundations.	The Soak–Away Pit will be located outside of the zone of influence.	Pas	Draft Preliminary Site Plans
Proximity to Road Subgrades*	The pit must be located outside of the zone of influence or at least 20 feet from road subgrades.	The Soak–Away Pit will be located more than 20 feet from road subgrades.	Pass	Draft Preliminary Site Plans
Runoff Source	Soak–away pit is not to receive runoff containing industrial pollutants.	Rooftop runoff.	Pass	Draft Preliminary Site Plans
Contributing Area	The contributing area must be less than 1,900 feet^2.	The approximate contributing area is 1,800 feet^2.	Pass	Draft Preliminary Site Plans
Slope of Available Area	The available area slope must be less than or equal to 12%.	The slope of the site is approximately 12%.	Pass	Draft Preliminary Site Plans
Available Area	The area available for treatment must be at least 4% of the total catchment area.	Approximately 40% of the total site area will consist of open space for lawns and landscaping.	Pass	Draft Preliminary Site Plans
Down Gradient Slope	Down gradient slope must be less than 12%.	The adjacent properties have slopes less than 12%.	Pass	Site Visit/Topo Map
Horizontal Separation Distance from Adjacent Soak–away Pits*	Soak–away pits must be separated by a distance of 20 feet.	The Soak–Away Pit will be located more than 20 feet from each other.	Pass	Draft Preliminary Site Plans

Note: * These criteria do not apply to soak–away pits with impervious collars.

F.2 Example Preliminary Design – Soak–Away Pit

In order to conduct the preliminary soak–away pit design, the preliminary design calculation table (Table 7) presented in Section 4 has been used. The completed preliminary design calculations are presented in Table F.2.

In Step 1 of the preliminary design calculations, the runoff coefficient, 0.87, has been obtained from the DDG. The calculation in Step 1 indicates that the soak–away pit will need to accommodate a volume of approximately 144 $feet^3$ of rooftop runoff.

In an attempt to develop a preliminary design that meets the bedrock separation distance requirement (3 feet), Steps 2 and 3 were performed iteratively. Step 2 was performed in the first iteration with a retention time of 24 hours. Due to the direct relationship between trench depth and retention time, the use of the minimum retention time in the first iteration produces the minimum allowable depth for a soak–away pit for the specified design infiltration rate. The resulting depth was 5 feet. Step 3 was then performed to calculate the required area for the soak–away pit. Based on a retention time of 24 hours, the target infiltration volume, and the design infiltration rate, the area required for the soak–away pit exceeded the maximum allowable area of 64 $feet^2$. Several iterations were then conducted with incremental increases in retention time until the resulting trench area was less than or equal to 64 $feet^2$. The resulting preliminary design indicates that the soak–away pit with a footprint of at least 63 $feet^2$ must be approximately 5.8 feet deep. This corresponds to a retention time of approximately 30 hours. Thus, the results of the preliminary design are all within the required ranges.

The results of the preliminary site evaluation and the preliminary design indicate that the site is a suitable candidate for the use of a soak–away pit to treat storm water runoff. Thus, final design efforts are warranted.

Table F.2 – Example Soak–Away Pit Preliminary Design

Site Location: 1112 W 100th Street		**Evaluated by:** Don Sheldon	
Date: 8/24/2007			
Step 1: Calculate the Target Infiltration Volume, TIV			**Notes**
Contributing Area, A	1800	(ft^2)	Should Be Less Than 1900 $feet^2$
Target Infiltration Rainfall, P	1.1	(in)	2–Year, 24–Hour Rainfall Depth
Runoff Coefficient, C	0.87		Calculated per DDG
TIV = A*P*C/12 =	144	(ft^3)	Using Equation 2.1
Step 2: Calculate the Depth of the Pit			Must be between 4 and 10 feet
Void Ratio, n_s	0.4		0.4 is typical of 1.5 to 3 in stone
Design Infiltration Rate, I	0.6	(in/hr)	Based on site investigation (Subsection 1.4.1 and DDG)
Retention Time, t	30	(hr)	Must be between 24 and 72 hours
$D_s = (I*t)/(n_s*12)+2 =$	5.8	(ft)	Using Equation 4.1
Step 3: Calculate the Soak–Away Pit Footprint			Must be less than 64 $feet^2$
TIV (from Step 1)	144	(ft^3)	
n_s (from Step 2)	0.4		
D_s (from Step 2)	5.8	(ft)	
$A_s = (TIV*0.66)/(n_s*(D_s–2)) =$	63	(ft^2)	Using Equation 4.2

F.3 Example Final Design – Soak–Away Pit

In order to develop the final design based on the dimensions calculated in the preliminary design, the minimum factors presented in Subsection 4.1.3 were addressed. In real world applications, the final design of a soak–away pit is likely to include slight adjustments in geometry and will likely include site related engineering considerations specific to the particular project. For the sake of the example, the dimensions calculated in the preliminary design have been directly applied to the final design.

Inlet – The inlet into a soak–away pit consists of one four–inch diameter perforated PVC pipe. Each pipe was perforated with 144 1–inch diameter holes offset by 1 inch longitudinally and 45 degrees radially around the circumference of the pipe. Each pipe was perforated along a 6–foot section leaving a minimum of 1 foot of separation distance from any perforation and the edge of the soak–away pit. The following formula (the orifice equation) was used to estimate the flow capacity of the perforations.

$$Q = N_o * A_o * C_d * \sqrt{2 * g * H}$$

Equation F.1

Q = Flow rate (feet^3/second)
N_o = Number of Orifices, 144
A_o = Orifice Opening Area (feet^2), 0.0055
C_d = Coefficient of Discharge, 0.62 for sharp edge entrance
g = Gravitational Constant (feet/second^2), 32.2
H = Head (feet), 0.33

The result is 2.3 feet^3/second; this is adequate capacity since it well exceeds the peak flow from the rooftop during a 1–year event as calculated using the Rational Method and guidance in the DDG. In this case, a 1–year peak flow rate of 0.03 feet^3/second was estimated based on a time of concentration of 5 minutes, an intensity of 0.7 inches per hour, and a weighted C value of 0.87.

Filter Fabric – The filter fabric has been approximately matched to the infiltration rate of the surrounding soils.

Overflow Structure – Two overflow structures have been incorporated into the inlet. The downstream overflow structure will allow storm water to discharge from the soak–away pit such that the pit will not experience excessive surcharging. The outlet of this overflow structure is covered with a hinged plate to prevent backflow. The second overflow structure is upstream along the rooftop downspout. This overflow structure is added as a safety measure.

Bottom Layer – A six inch layer of sand was incorporated into the bottom of the soak–away pit to promote even infiltration and to limit the potential of localized compaction of underlying soils during the placement of the storage media.

Grading – The bottom of the pit has been graded completely level.

Observation well – The observation well that has been included in the soak–away pit design is 6 inches in diameter. The observation well is perforated along the full depth of the soak–away pit. The location of the well is offset from the center to avoid conflicts with the soak–away pit inlet.

A conceptual drawing of the soak–away pit resulting from this design effort is presented in Figure F.1.

Figure F.1 – Soak–Away Pit Design Example

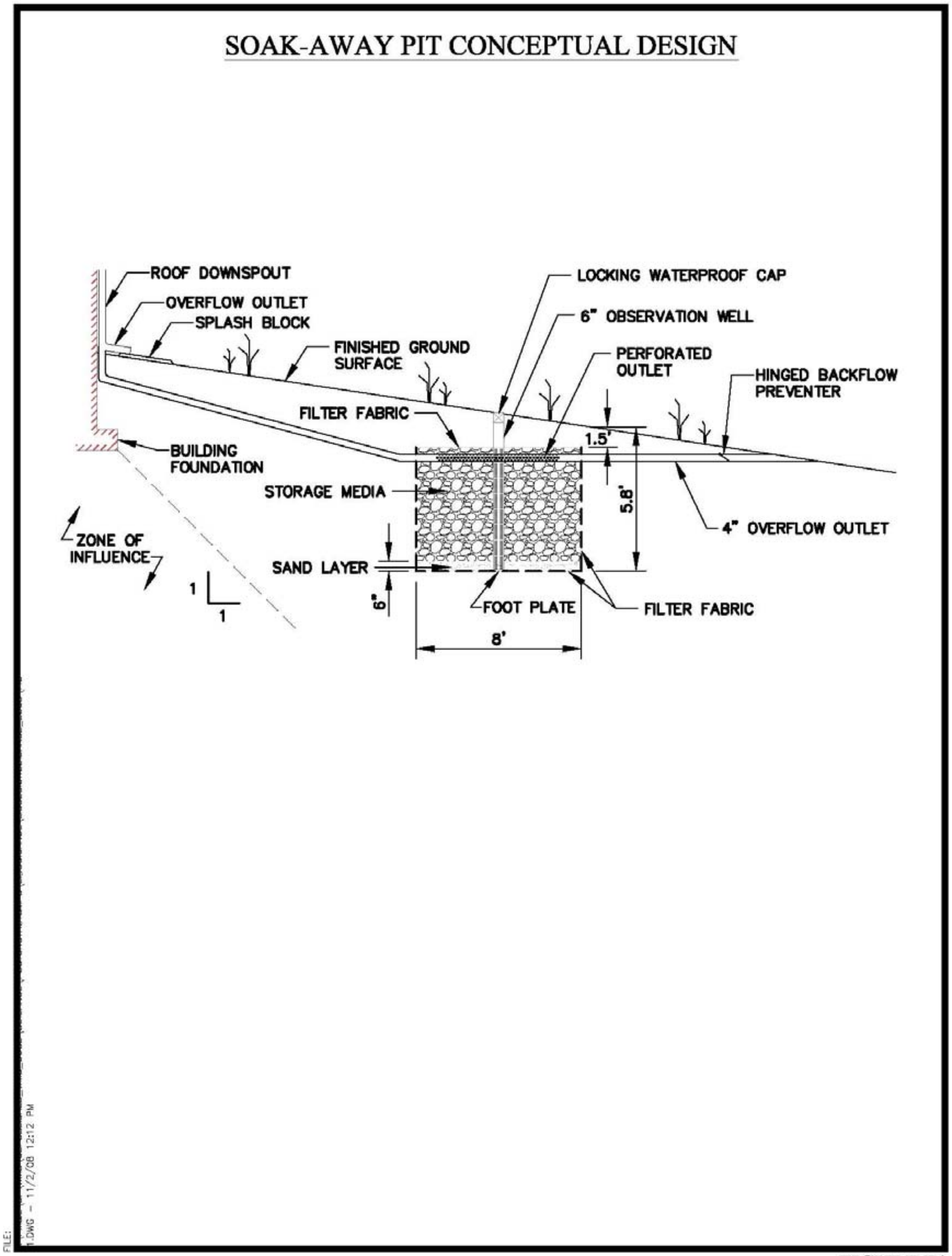


FIGURE F1

Appendix G
Filter Strip Design Examples

G. Filter Strip Design Example

This section presents the design process for a filter strip to infiltrate parking lot runoff from a portion of the site described below.

Site Description: *A small commercial strip development will include a parking area to accommodate 20 vehicles. The site presently drains towards a frontage street into a curb and gutter storm drain system. The area available for the parking lot is 200 feet long by 65 feet wide. In order to meet the water quality protection criteria specified in the DDG, a filter strip is proposed to treat runoff from the parking area prior to discharge to the curb and gutter system along the frontage street. Making the assumption that the filter strip will be approximately 25 feet long (dimension parallel to flow) the parking area will be approximately 200 feet by 40 feet.*

In the following subsection, the preliminary site evaluation and a preliminary design are presented for a filter strip for the parking lot. Following these sections, a final design is discussed and a conceptual drawing of the final design is presented.

G.1 Example Preliminary Site Evaluation – Filter Strips

In order to conduct the preliminary site evaluation, the preliminary site evaluation checklist (Table 9) provided in Section 5 has been used. To fill out the preliminary site evaluation checklist, the draft preliminary site plans were required.

The completed preliminary site evaluation checklist is presented as Table G.1. The information presented in Table G.1 indicates that the site is suitable for the use of a filter strip to treat parking lot runoff.

Table G.1 – Filter Strip – Preliminary Site Evaluation Checklist

Site Location: 1115 W 100th Street **Evaluated by:** Aldo Leopold

Date: 8/24/2010

Considerations	Requirement/Recommendation	Site Conditions / Notes	Pass/Fail	Data Source
Runoff Source	Filer strip is not to receive runoff containing industrial pollutants.	Runoff is from a parking lot.	Pass	Draft Preliminary Site Plans
Contributing Area	The contributing area must be less than 1 acre.	Contributing area is approximately 0.18 acres.	Pass	Draft Preliminary Site Plans
Slope of the Contributing Area	Slope of the contributing area must be less than 10%.	The parking lot will have a slope much less than 10%	Pass	Draft Preliminary Site Plans
Available Area	The available area for the filter strip shall generally extend the full width of the contributing area and allow for a length (parallel to flow) of 15 to 25 feet. The ratio of total contributing area to the total available area must not exceed 6:1.	Site provides adequate space for a filter strip. The available area for the filter strip (200 feet by 25 feet) is more than 1/6th the size of the contributing parking lot (200 feet by 40 feet).	Pass	Draft Preliminary Site Plans

G.2 Example Preliminary Design – Filter Strips

In order to conduct the preliminary filter strip design, the preliminary design calculation table (Table 10) presented in Section 5 has been used. The completed preliminary design calculations are presented in Table G.2.

In Step 1, of the preliminary design calculations, the maximum allowable depth of flow is assumed, the design slope is set to 3 %, and a Manning's "n" of 0.25 is selected for dense grass. The calculation in Step 1 indicates that the filter strip will be able to accommodate 0.005 $feet^3/sec$ runoff for every linear foot of width (the dimension perpendicular to flow).

In Step 2, the velocity along the filter strip is checked by dividing the maximum discharge loading by the design depth. According to the calculations is Step 2, the design velocity is 0.12 feet/second, which is equal to the maximum allowable velocity.

In Step 3, the minimum allowable filter strip width is calculated. The rational coefficient in this computation is selected based on guidance provided in the DDG. The results of the computation in Step 3 indicate that the minimum allowable width for the filter strip is 15.3 feet. This is much less than the available width of 200 feet. Therefore, the preliminary design proceeds to Step 4.

In Step 4, the minimum allowable filter strip length (dimension parallel to flow) is calculated. In this step, a travel time of 5.5 minutes was selected. According to the computations in Step 3, the minimum allowable filter strip length is 21.0 feet. This is approximately equal to the assumed length of 25 feet. Thus, final design efforts are warranted.

Table G.2 – Filter Strip Preliminary Design

Site Location: 1112 W 100th Street		**Evaluated by:** Don Sheldon	
Date: 8/24/2012			
Step 1: Calculate the Maximum Discharge Loading, q			**Notes**
Maximum Allowable Depth of flow, Y	0.5	(in)	Maximum is 0.5 inches
Slope of Filter Strip, S	0.03	(ft/ft)	Between 0.01 and 0.06
Manning's "n"	0.25		
$q=(1.49/n)*(Y/12)^{(5/3)}*S^{(1/2)}$	0.005	(ft^3/sec–ft)	Using Equation 5.1
Step 2: Check Velocity, V			Maximum Allowable is 0.9 ft/sec
q (from Step 1)	0.005	(ft^3/sec–ft)	
Y (from Step 1)	0.5	(in)	
$V=q/(Y/12)$	0.12	(ft/sec)	Using Equation 5.2
Step 3: Calculate the Minimum Allowable Filter Strip Width, W_{fp}			
q (from Step 1)	0.005	(ft^3/sec–ft)	
Contributing Area, A_a	0.18	(acres)	
Runoff Coefficient, C	0.85		Per DDG
$W_{fp}= (A_a*C*0.5)/q$	15.3	(ft)	Using Equation 5.3
Step 4: Calculate the Minimum Allowable Filter Strip Length, L_f			
Travel Time Through Filter Strip, T_t	5.5	(min)	Between 5 and 9 Minutes
Target Precipitation, P	1.3	(in)	1.3 inches
S (from Step 1)	0.03	(ft/ft)	
n (from Step 1)	0.25		
$L_f=(T_t^{1.25}*P^{0.625}*(S*100)^{0.5})/3.34*n$	21.0	(ft)	Using Equation 5.4

G.3 Example Final Design – Filter Strips

In order to develop the final design based on the dimensions calculated in the preliminary design, the minimum factors presented in Subsection 5.2.3 were addressed. In real world applications, the final design of a filter strip is likely to include slight adjustments in geometry and will likely include site related engineering considerations specific to the particular project. For the sake of the example, the dimensions calculated in the preliminary design have been directly applied to the final design discussed below.

Overall Site Integration – The existing site did not offer the opportunity to use areas of existing vegetation as filter strips. The existing site offers enough space to meet the desired parking area with additional room for a well designed and constructed filter strip that can sheet drain to an existing curb and gutter system. The filter strip has been placed lengthwise between the frontage road and the new parking area. The parking area has been graded to sheet drain to the filter strip. However, because the parking spaces require parking stops, which will concentrate flows upstream of the filter strip, the design has incorporated a level spreading device.

Filter Strip Cover – The selected filter strip cover in this design is Schedule C seed mix, as defined in the DDG. This grass will require little maintenance and will provide a natural appearance to the site. The application rate is 5 lbs/1,000 square feet.

Level Spreading Devices – As mentioned previously, a level spreading device is required in this design. The device selected is a gravel–filled trench. The trench is 12 inches wide by 24 inches deep. It is lined with a non–woven geotextile material and has a 1 inch drop along its leading edge.

A conceptual plan and profile drawing of the filter strip resulting from this design effort is presented in Figure G.1. This design will provide treatment for the first flush from rainfall events and will meet the water quality design requirements specified in the DDG.

Figure G.1 – Filter Strip Conceptual Plan and Profile