Volume Reduction of Highway Runoff in Urban Areas: Final Report and NCHRP Report 802 Appendices C through F

DETAILS

219 pages | 8.5 x 11 | PAPERBACK
ISBN 978-0-309-43265-8 | DOI 10.17226/22169

AUTHORS

Eric Strecker, Aaron Poresky, Robert Roseen, Ronald Johnson, Jane Soule, Venkat Gummadi, Raina Dwivedi, Adam Questad, Neil Weinstein, Emily Ayers, and Marie Venner; National Cooperative Highway Research Program; Transportation Research Board; National Academies of Sciences, Engineering, and Medicine

ACKNOWLEDGMENT

This work was sponsored by the American Association of State Highway and Transportation Officials (AASHTO), in cooperation with the Federal Highway Administration, and was conducted in the National Cooperative Highway Research Program (NCHRP), which is administered by the Transportation Research Board (TRB) of the National Academies.

TABLE OF CONTENTS

AUTHOR ACKNOWLEDGMENTS

The research reported herein was performed under NCHRP Project 25-41 by Geosyntec Consultants, with assistance from The Low Impact Development Center and Venner Consulting. Primary authors of this report were:

Eric Strecker, Principal Investigator
Aaron Poresky
Geosyntec Consultants
Portland, OR

Neil Weinstein
The Low Impact Development Center
Beltsville, Maryland

Marie Venner
Venner Consulting
Littleton, CO

The authors would like to recognize the contributions of the following additional authors:

Geosyntec Consultants
Adam Questad
Andrea Braga
Jane Soule
Lucas Nguyen
Raina Dwivedi
Renee Bordeau
Robert Roseen
Ron Johnson
Venkat Gummadi

The Low Impact Development Center
Emily Ayers
Lilantha Tennekoon

ABSTRACT

This report and the accompanying Guidance Manual summarize the research and resulting guidance developed for achieving surface runoff volume reduction of highway runoff in urban areas. Literature review, synthesis, and focused new analyses were conducted to inform and develop guidance for identifying, evaluating, selecting, and applying volume reduction approaches (VRAs). A stepwise approach was developed and included in the Guidance Manual that guides the user through the evaluation and selection of VRAs. Regulatory and physical considerations were evaluated related to volume reduction in the urban highway environment, and key factors influencing the ability to achieve volume reduction were identified. Recommendations for project-specific site assessment efforts to support volume reduction planning and design were researched and included. Nine primary VRAs were identified that are specifically applicable to the urban highway environment, as well as accompanying site planning and evaluation approaches. A framework for evaluating the applicability, feasibility, and desirability of these VRAs based on site-specific factors was developed. Processes were also developed for prioritizing VRAs and developing conceptual designs. In support of the Guidance Manual, a spreadsheet based Volume Performance Tool was developed to provide planning level estimates the performance of VRAs, and four white papers on key technical topics were developed. The findings of this research suggest that site-specific conditions are critical in determining the applicability, feasibility, desirability, and effectiveness of VRAs. Additionally, maintaining VRAs is critical for long term effectiveness.

Summary

DOTs are facing increasing requirements to reduce the volume of stormwater runoff from highways. However, implementing stormwater volume reduction approaches (VRAs) in highly constrained urban environments presents a number of challenges and constraints due to the limited space and appropriate soils typically available in the right location in the right-of-way for infiltration, evapotranspiration, on-site use, and/or flow control. Additional constraints include road safety requirements, geotechnical and structural concerns associated with saturated soils in the highway environment, costs to construct VRAs, cost and ability to maintain VRAs, water balance issues and other factors. Finally, the effectiveness of VRAs is an important factor in determining whether costs of VRAs justify the benefit of reducing runoff volume.

The purpose of this project was to develop guidance for transportation agencies on implementing surface runoff volume reduction practices that are specific to limited access urban highways and specific to the conditions encountered as part of each project. The primary result of this project was the development of the *Guidance for Achieving Volume Reduction of Highway Runoff in Urban Areas* (Guidance Manual). The intended audience of the Guidance Manual included project designers, project planners, program managers and regulators. The Guidance Manual includes text, as well as flow charts, worksheets, and other user tools to facilitate its implementation. It is accompanied by a number of technical appendices as well as a Volume Performance Tool in Microsoft Excel.

The Guidance Manual was developed based on literature review, synthesis of available information, and focused technical analysis, including consideration of both established and innovative approaches for achieving volume reduction. One of the overarching findings of this research was that achieving volume reduction of urban highway runoff is dependent on many site-specific factors and considerations. The overwhelming importance of these factors means that it was not possible to reach categorical conclusions about the feasibility, effectiveness, and/or cost of achieving volume reduction that are applicable across site and watershed conditions, projects types, design goals and various other site-specific factors. Project planning and design decisions must be supported by careful evaluation of site-specific factors.

As such, the primary outcome of this project was the development of an evaluation process with supporting references (as provided in the Guidance Manual and appendices) by which practitioners can identify, evaluate, and design feasible solutions for runoff volume reduction based on project-specific conditions. This process is presented in the Guidance Manual as a five-step process, including:

Step 1 – Establish Volume Reduction Goals.
Step 2 – Characterize Project Site and Watershed.
Step 3 – Identify Potentially Suitable VRAs – Preliminary Screening and Site Planning.
Step 4 – Prioritize VRAs.
Step 5 - Select VRAs and Develop Conceptual Designs.

For each step in this process, the Guidance Manual provides extensive supporting information as well as user tools (e.g., worksheets, flow charts, the Volume Performance Tool). Highlights of the contents of the Guidance Manual include:

- Background on regulatory requirements for volume reduction as well as the benefits and limitations to a volume reduction strategy.
- Unit process-based description and understanding of volume reduction processes, including the factors that make VRAs more or less effective.
- Guidance for conducting site assessment activities in support of volume reduction planning and design, including guidance on how to phase site assessment activities to improve the efficiency of project development.
- Detailed descriptions of nine (9) primary VRAs, including supporting fact sheets for each, as well as site planning approaches to facilitate volume reduction.
- A comprehensive framework (with accompanying worksheets and flowcharts) for evaluating the applicability, feasibility, and desirability of VRAs to help identify VRAs that are compatible with project and watershed conditions.
- A semi-quantitative approach for prioritizing VRAs based on relative life cycle costs, relative operations and maintenance (O&M) impacts to agencies, relative reliability, relative safety, and potential performance relative to volume reduction goals.
- Guidance for developing and analyzing conceptual designs, including design schematics, a whole lifecycle cost framework, the Volume Performance Tool, and guidance for adapting project plans to help improve effectiveness and/or reduce cost.
- Four (4) White Papers on key technical topics related to volume reduction including 1) infiltration testing, 2) groundwater issues, 3) geotechnical issues, and 4) permeable pavements.

The contents of the Guidance Manual are also intended to support users who require “a la carte” information on specific technical topics related to volume reduction planning or design -- the Guidance Manual represents a rigorous compilation and synthesis of technical information on the key information that is important for volume reduction. However, the full potential of the Guidance Manual can be realized by using it as part of a systematic approach for achieving volume reduction. By applying a methodical process to volume reduction planning and design (such as described in the Guidance Manual), it may be possible to identify more opportunities to achieve volume reduction while also more carefully identifying conditions in which volume reduction is not appropriate and should be limited or avoided due to potential negative impacts.

CHAPTER 1

Background

Surface runoff volume reduction can be an important element of controlling stormwater impacts in urban areas when appropriate. The 2009 National Academy of Sciences (NAS) report, *Urban Stormwater Management in the United States* presented shortcomings in the current National Pollutant Discharge Elimination Systems (NPDES) program and provided suggestions to correct the identified deficiencies. The NAS report notes, "Presently … the regulation of stormwater is hampered by its association with a statute that focuses primarily on specific pollutants and ignores the volume of discharges." The report notes that in an idealized regulatory system, "Future land-use development would be controlled to prevent increases in stormwater discharges from pre-development conditions, and impervious cover and volumetric restrictions would serve as a reliable proxy for stormwater loading from many of these developments." The report also identified roads and parking lots as the most significant land use with respect to stormwater volumes and pollution.

However due to a suite of constraints and other design restrictions associated with the urban highway environment, many volume reduction approaches (VRAs) are not applicable or must be used carefully in this application. The assessment of feasibility of VRAs in the urban highway environment must consider a broad suite of factors to ensure that the approaches do not lead to negative impacts to the highway infrastructure, human safety, surrounding infrastructure, groundwater supplies, and the environment. As a result, consideration and incorporation of VRAs can add complexity to transportation project planning, design, construction, operations, and maintenance.

Consequently, the National Cooperative Highway Research Program (NCHRP) initiated this study to develop practical, technically defensible, and comprehensive guidelines for transportation agencies on implementing volume reduction strategies that are specific to limited access urban highways. Specific objectives of the project included:

- The Guidance Manual should be applicable across a broad range of urban highway project types, site conditions, and climate zones.
- The Guidance Manual should have sound technical bases and should build from the current state of the practice relative to VRAs.
- The Guidance Manual should advance beyond an assessment of the applicability of conventional VRAs and consider innovative approaches that are specifically suited to the urban highway environment.
- The Guidance Manual should be user-friendly and readily transferable in order to facilitate streamlined adoption and use by intended users.

These objectives were addressed through a comprehensive guidance manual development process that involved literature review, technical analysis, development of numeric tools, and development of guidance manual elements (such as worksheets and flow charts). This effort culminated in the development of the Guidance Manual titled *Guidance for Achieving Volume Reduction of Highway Runoff in Urban Areas*.

References for More Information

- Section 1 and 3 of the Guidance Manual provide more information on the background and context for achieving volume reduction of urban highway runoff. This information was included in the main body of the Guidance Manual instead of this report because is useful as a direct reference by a practitioner who is seeking to gain a better understanding of the regulatory and technical background for achieving volume reduction.
- Each section of the report and technical annex provides additional topic-specific background information associated with the respective topic of that section or annex, as appropriate.

Note, to avoid redundancy, this report does not reiterate the findings that are presented in the Guidance Manual. Rather, this report provides a brief summary of these findings and provides references to the sections of the Guidance Manual where a more complete presentation of findings can be found. Additionally, this report does not present findings beyond those presented in the Guidance Manual, therefore technical citations are not included in the report. For detailed technical findings and citations on a given topic, see the respective sections Guidance Manual.

CHAPTER 2

Research Approach

To meet the objectives of the project, the research approach was divided into five tasks as introduced below.

- **Task 1** included characterizing the urban highway environment relative to runoff volume reduction. This characterization included both regulatory and technical considerations. This task also resulted in guidance for practitioners to use as part of characterizing a specific project site.
- **Task 2** included developing potential VRAs. This task involved review of existing guidance as well as recent literature.
- **Task 3** included evaluating these VRAs on a number of technical factors to identify a narrower menu of VRAs for inclusion in the Guidance Manual. This task also included development of metrics, criteria, and tools for practitioners to use in evaluating VRAs on a project-specific basis.
- **Task 4** included focused technical analyses of several key areas that the research team identified as being important as part of achieving volume reduction in the urban highway environment. The outcome of this task was four technical white papers which were included as appendices to the Guidance Manual.
- **Task 5** included compiling the results of Task 1 through 4 into a functional Guidance Manual. This included developing a stepwise structure to the Guidance Manual and integrating technical components into this structure in a usable and logical format.

The sections below summarize the efforts that were involved in completing each task.

Task 1: Characterize the Urban Highway Environment Relative to Runoff Volume Reduction

The objective of this task was to define the scope of the Guidance Manual and to develop a baseline characterization of the urban highway environment for use throughout the development of the Guidance Manual. The research team reviewed literature, regulations, and other technical documentation to develop a summary of the various motivations for reducing surface runoff volumes, underlying concepts, and constraints related to volume reduction strategies in the urban highway environment. Based on this information, the research team developed the scope of the Guidance Manual and prepared an annotated outline for Project Panel review. The information developed in this task was then summarized in the Guidance Manual with the intent of providing background and introduction for users by introducing key concepts and vocabulary. The overall goals of this task were to:

- Understand the regulatory and design considerations that exist in the urban highway environment as related to surface runoff volume reduction,

- Characterize the key types of urban highway projects and conditions that the Guidance Manual must support,
- Identify the key physical parameters that are most influential in volume reduction effectiveness and feasibility, and
- Identify site and watershed information and assessment needs for project planners and designers seeking to apply VRAs.

Subtask 1.1: Summarize the Regulatory Context

Stormwater quality regulations were identified as the main motivations for the use of stormwater VRAs. It is critical to consider that there are many other regulatory and design requirements that influence the design and construction of roadways including safety, flood management, construction site stormwater quality, and others. It was important to understand how these various considerations interact within an urban highway project to understand the context for selecting and applying VRAs. This subtask sought to characterize the general regulatory context within which the Guidance Manual is intended to be applied.

The research team reviewed and summarized stormwater management regulations that directly or indirectly mandate the consideration and use of stormwater volume reduction, including current regulations and evolving regulatory trends that could potentially affect DOTs in the future. Examples include:

- EPA goals and directions related to potential changes to NPDES discharge permits based on National Academy of Sciences (NAS) recommendations
- Section 438 of the Energy Independence and Security Act (EISA) (not applicable to urban highways, but indicative of general trends)
- Localized MS4 Stormwater Permits that require volume control to be prioritized (CA, DC, etc.)
- Total Maximum Daily Loading Limits (TMDLs) that may incentivize volume reduction or use volume as a surrogate for pollutant loads

The research team relied, in part, on work conducted as part of NCHRP 25-25 (Cost and Benefit of Transportation Specific MS4 and Construction Permitting) for this evaluation, as well as review of more recent regulatory information. The research team also reviewed and summarized the other regulatory considerations and design requirements that exist within the highway project development process, including highway design requirements for safety, flood control, construction site stormwater quality and others, as applicable.

Subtask 1.2: Characterize Urban Highway Project Attributes, Physical Setting, and Key Constraints

The types of urban highway projects that are supported by the Guidance Manual include a wide range of attributes and physical settings. For example, projects may range from new roadway construction near the urban fringe, to the addition of a new HOV lane in a depressed ultra-urban freeway section, to the construction of a freeway intersection flyover segment. The purpose of this task was to characterize the potential range of project types so that the Guidance Manual could be developed to be applicable across these ranges.

In addition, this task identified the key constraints for volume reduction that are expected to be encountered in the types of urban highway projects that the Guidance Manual supports. The research team reviewed and summarized the challenges for implementing volume reduction, including, but not limited to:

- *Physical Limitations:* Constrained urban highway settings by definition have limited surface area within the right-of-way (ROW) for siting above ground storage and infiltration or use facilities. Moreover, due to the linear nature of highways, storage and infiltration or use facilities must be distributed at collection points and located in opportunity areas.
- *Hydraulic Loadings:* Constrained urban highway settings typically have small drainage areas with a high proportion of impervious cover and little vegetated pervious area. This results in comparatively large runoff volumes compared to the opportunities that exist to mitigate runoff volume.
- *Feasibility of Infiltration:* Infiltration practices in constrained urban highway settings and dense urban areas may be restricted by limited surface area, prohibitive infiltration rates of compacted soils, geotechnical concerns for the protection of the roadway subgrade, and/or a greater likelihood of utility conflicts and contaminated soils and/or groundwater.
- *Opportunities for On-site Use:* Limited pervious/landscaped areas within urban highway ROW generally provide few opportunities for on-site usage of harvested stormwater; watershed-based use approaches beyond the ROW will typically need to be evaluated for potential usage of harvested water if harvest and use VRAs are employed for highway runoff.
- *Construction and Maintenance Costs and Practicality:* Space constraints, unsuitable soils, and utility conflicts may each limit the suite of viable VRAs and/or increase costs. Additionally, the costs and operational burdens of maintaining VRAs and the ability to safely access facilities for maintenance are important considerations.
- *Opportunities:* The unique characteristics of urban highway design and construction may provide enhanced opportunities for volume reduction if well-suited VRAs are selected. In some cases VRAs may be the most cost-effective solution. Specific opportunities for volume reduction present in the urban highway environment were introduced.

Subtask 1.3: Identify Key Factors and Opportunities in Design and Application of Stormwater Volume Reduction

As part of this task, the research team developed a brief technical explanation of volume reduction processes to establish the technical foundation for subsequent analyses and provide educational material for the end user. This explanation included descriptions of key factors in volume reduction performance, feasibility, and cost including:

- The ability to capture and store runoff.
- The ability to recover storage capacity between storms through infiltration, evapotranspiration (ET), or treatment and utilization for other uses such as irrigation and non-potable supply.
- Site-specific design factors such as local climatic characteristics, watershed characteristics, geotechnical conditions, adjacent land uses, and highway designs.
- Ratios between areas generating runoff and areas available to manage runoff.
- Cost elements that are specific to VRAs.
- Opportunities to enhance volume reduction cost effectiveness and practicability.

The purpose of this subtask was to develop information to educate users on basic principles of volume reduction and what levels of reduction are potentially reasonable and not. Additionally, the identification of key parameters was used to identify site and watershed assessment needs, as well as to formulate and communicate the key research needs to be addressed in subsequent detailed analyses (Task 4).

References to Final Work Products

The results of Task 1 formed the basis for the following sections of the Guidance Manual:

- The results of Task 1.1 are provided in Chapter 1.2 and 3.1 of the Guidance Manual.
- The results of Task 1.2 and Task 1.3 are provided Section 3.2 through 3.4 of the Guidance Manual.

Task 2: Develop Potential Volume Reduction Approaches

The objectives of this task were (1) to provide a summary of the state of the practice of volume reduction, (2) to produce a focused menu of practicable approaches specific to volume reduction in the urban highway environment, (3) to define approaches in sufficient detail to distinguish key design parameters and applicability. The results of this task formed the basis for a number of sections and an appendix of the Guidance Manual.

Subtask 2.1: Literature Review and Data Compilation

The research team reviewed and summarized recent domestic and international literature on runoff VRAs, with a focus on applicability to the urban highway environment. The literature survey included journals, conference proceedings, and research reports from academic institutions, as well as state and international DOTs. The focus of the literature evaluation included:

- *Feasibility and Design Factors:* Compilation and synthesis of information and data related to feasibility and design factors, including groundwater quality protection, geotechnical hazards, soil infiltration rates and factors of safety, identifying and quantifying use demands and ET rates, and sizing and designing storage and distribution facilities.
- *Effectiveness Data and Factors:* Compilation and synthesis of information and data related to effectiveness of volume reduction practices, including such factors as climatic patterns, runoff storage and sizing, and pathways and rates of storage recovery (i.e. infiltration, evapotranspiration, harvest and use, and release at “base flow” rates).
- *Construction and Maintenance Considerations:* Compilation and synthesis of information and data related to construction and maintenance practices, requirements, and potential constraints in urban highway environments.
- *Cost Data:* Compilation and synthesis of available cost information and data, including various cost elements, cost factors, and potential cost reduction strategies.
- *Definition of Volume Reduction Metrics:* Definition of volume reduction and the metrics used to quantify volume reduction based on review of recent literature.

The intent of this subtask was to provide a summary of the state of the practice, provide the basis for developing the menu of approaches, and identify key areas where additional research was needed.

Subtask 2.2: Develop a Preliminary Menu of Volume Reduction Approaches and Evaluation Methods

The research team compiled a preliminary inventory of conventional and innovative VRAs based on results of Subtask 2.1. From this inventory, the team worked with the Project Panel to select the most promising nine (9) approaches for further evaluation and inclusion in the Guidance Manual. For each selected VRA the research team developed a description of the system components, the key processes and design parameters, the applicability to urban highway environments, and the applicability across climate zones and watershed characteristics. The approaches and strategies that were selected include above and below ground storage facilities, above and below ground infiltration practices and facilities, associated stormwater treatment practices and facilities, and stormwater conveyance and dispersal options. A fact sheet was prepared for each of the selected VRAs. Through the consideration of urban highway characteristics and construction practices, the research team identified geometries and configurations associated with each of the VRAs that are compatible with the urban highway environment, as well as design adaptations to help improve volume reduction performance. Additionally, the research team developed recommendations for project layout (i.e., site design) to enhance opportunities for VRAs.

References to Final Work Products

The results of Task 2 formed the basis for the following sections of the Guidance Manual:

- Section 4.1 of the Guidance Manual reports the findings from the research conducted to identify the potential menu of VRAs.
- Section 4.2 of the Guidance Manual identifies the menu of VRAs that were selected for inclusion in the Guidance Manual.
- Appendix A of the Guidance Manual provides fact sheets for each of the selected VRAs.

Task 3: Evaluate Volume Reduction Approaches

The outcome of this task was the development of guidance for evaluating, comparing, and selecting applicable VRAs, where feasible. This task also resulted in the development of a spreadsheet based Volume Performance Tool for conducting quantitative comparisons between VRAs. This task included synthesis from the results of Task 1 and 2, as well as results of focused technical analysis (Task 4). The outcomes from this task formed the basis for sections of the Guidance Manual intended to provide processes and resources for: (a) selecting the most promising VRAs for the project from the menu of VRAs prepared in Task 2, and (b) comparing and prioritizing VRAs as well as the spreadsheet tool and appendix.

Subtask 3.1: Develop Evaluation and Selection Criteria; Develop Selection and Feasibility Matrices

The research team compiled and organized feasibility criteria, costs, operations and maintenance requirements, volume reduction performance and other criteria that can be used to compare the VRAs compiled in Task 2. Based on synthesis of these criteria and factors, a series of selection and evaluation matrices were developed to provide end users with tools (i.e., flow charts, tables, worksheets) to quickly determine if a given VRA is likely to be applicable and effective for their site. This guidance allows the user to take into account site-specific information about project characteristics, physical constraints, watershed characteristics, and climate, in determining the recommended VRAs for further consideration.

This guidance was also intended to provide technical basis for where VRAs may not be feasible or appropriate. The selection and evaluation matrices, flowcharts, and worksheets include considerations related to the following:

Feasibility criteria - The research team investigated volume reduction implementation feasibility and provided guidance for considering the following factors in determining the applicability, feasibility, and desirability of a VRA for a given site:

- Climate and local hydrology
- Availability of space and elevation differential
- Site soil infiltrative capacity limitations
- Potential for utility conflicts and impacts to existing infrastructure
- Potential impacts to ground water and
- Various other potential feasibility criteria.

The outcome of this investigation was the development of a process and associated criteria for evaluating the applicability, feasibility, and desirability of a given VRA. Additionally, matrices were developed to provide a relative comparison of the metrics to inform this process. Matrices included a comparison of volume reduction processes, geometric siting considerations, potential geotechnical impacts, potential water groundwater impacts, and safety considerations for the selected VRAs.

Relative costs - The research team reviewed literature to develop relative cost comparisons for the selected VRAs, including relative costs for construction, operations and maintenance, and replacement/restoration. Relative costs were summarized for the cost of VRA construction as part of a larger construction project, as well as costs of VRA construction as retrofit projects where no other construction activities are being conducted. Retrofit costs tend to be higher in most cases. Costs are highly site-specific, therefore guidance was also developed for preparing site-specific whole lifecycle cost estimates. Cost reductions and synergies to be had by using green infrastructure for stormwater conveyance were considered and the cost assessment framework provided guidance for accounting for "avoided costs" in comparing different project scenarios. In other words, guidance was provided for accounting for the avoided cost of grey infrastructure in the determining the true incremental cost of VRAs.

Operation and maintenance – The research team conducted a literature review and conducted informal interviews with DOT representatives to summarize the maintenance requirements for each of the selected VRAs. Operations and maintenance activities were identified for each VRA in a matrix format, including both routine maintenance activities and corrective maintenance activities. VRA-specific maintenance considerations and requirements were also summarized in each VRA fact sheet (Appendix A of the Guidance Manual).

Performance – The research team investigated and compiled volume reduction performance factors for each of the VRAs. These factors were used to evaluate the applicability of each type of VRA for different site and watershed conditions. The results of this compilation were expressed in terms of the space requirements and infiltration conditions necessary to support each VRA type. The Volume Performance Tool (described in Task 3.4) was used as the quantitative basis for evaluating potential levels of expected performance for numerous different geographic and site conditions and the sensitivity of key performance factors.

Subtask 3.2: Summarize Feasibility-related Research Gaps and Identify Focused Analysis Topics

As an outcome of the previous tasks, the research team identified areas that were considered to be significant research gaps and/or opportunities to consolidate available research to be applicable to the urban highway environment. Such research gaps and/or opportunities were documented and recommended for further study when encountered. Those topics that required further study and also were of specific importance in evaluating VRAs were identified for focused technical analysis as part of Task 4.

Subtask 3.3: Develop Selection and Feasibility Matrices

This subtask was originally planned as a separate step from Subtask 3.1, however this effort was combined with Subtask 3.1 to result in selection and feasibility matrices for the selected VRAs (see discussion above).

Subtask 3.4: Develop Performance Evaluation Tool

The purpose of this subtask was to develop a “Volume Performance Tool” (Tool) for planning level estimation of expected performance of VRAs. As part of developing the Tool, the research team used long term precipitation and potential evapotranspiration data in combination with continuous simulation hydrologic modeling for numerous specific climates and soil conditions to develop a repository of sizing/configuration/site conditions performance relationships and lookup tables (i.e. nomographs). The user interface for the Tool was developed in Microsoft Excel. The intended uses of this Tool are described in Chapter 5 of the Guidance Manual and a concise User’s Guide is provided as Appendix B of the Guidance Manual. Further details regarding the technical development of the Tool are provided in Annex A of this project report.

References to Final Work Products

The results of Task 3 formed the basis for several sections of the Guidance Manual:

- Section 3.4 of the Guidance Manual provides guidance for site assessment activities to support evaluation of feasibility of volume reduction processes.
- Section 4.3 of the Guidance Manual provides a summary of VRA attributes for a number of evaluation metrics. The discussions and matrices found in this section are intended to help support feasibility comparisons and prioritization of VRAs.
- Chapter 5 of the Guidance Manual presents the overall framework for selecting and applying VRAs, much of which is informed by the results of this Task. Specifically Section 5.2 of the Guidance Manual presents guidance for evaluating feasibility and desirability of VRAs, while Section 5.3 of the Guidance Manual presents guidance for prioritizing VRAs after evaluating feasibility and desirability.

Task 4: Conduct Focused Technical Analyses

This task included focused analysis and white paper development for key technical topics. The objective of this task was to conduct concise technical analysis and/or synthesize and consolidate existing literature to expand the state of the practice in areas that are important for the practical application of VRAs. The outcomes of this task included findings that are of what we believe to be highly practical relevance to users of the Guidance Manual. These findings were incorporated in the main body of the Guidance Manual, as appropriate, and the more detailed supporting documentation was incorporated into the Guidance Manual as White Papers.

An initial list of topics was developed based on areas where: (a) research gaps exist and/or consolidation of literature was needed, and (b) the topic was of specific importance in evaluating and applying VRAs. The list of topics was provided to the Project Panel and was agreed upon as part of an interim meeting with the Project Panel. The selected topics are described in the subtask headings below. The findings of these white papers significantly influenced and informed the content of the Guidance Manual related to site investigation/characterization, feasibility and desirability evaluation, VRA selection, and conceptual design approaches. These white papers are valuable for project evaluations, but are also valuable for larger regulatory development discussions as well as State or Local DOT or MS4 guidance document development.

Subtask 4.1: Infiltration Testing and Factors of Safety in Support of the Selection and Design of Volume Reduction Approaches

This white paper provides guidance for assessing the infiltration capacity of a given site, including methods and concepts that are applicable at the planning and design phases of the project, as well as guidance on selecting an appropriate factor of safety on measurements (White Paper 1; Appendix C of the Guidance Manual).

Subtask 4.2: Potential Impacts of Highway Stormwater Infiltration on Water Balance and Groundwater Quality in Roadway Environments

This white paper provides guidance for identifying potential impacts related to water balance and groundwater quality and provides recommendations for project planners and designers with respect to assessing and avoiding and/or mitigating these potential impacts (White Paper 2; Appendix D of the Guidance Manual).

Subtask 4.3: Geotechnical Considerations in the Incorporation of Stormwater Infiltration Features in Urban Highway Design

This white paper provides guidance to help identify potential geotechnical and pavement impacts of stormwater infiltration, and to help guide the development of geotechnical designs with respect to assessing and/or mitigating these potential impacts (White Paper 3; Appendix E of the Guidance Manual).

Subtask 4.4: Review of Applicability of Permeable Pavement in Urban Highway Environments

This white paper provides guidance on potential applicability of permeable pavement technologies in the urban highway environment to help owners, project managers, and designers evaluate whether permeable pavements should be considered for a specific project (White Paper 4; Appendix F of the Guidance Manual).

References to Final Work Products

The results of this task are presented in their entirety as White Papers #1 through 4 (Appendices C through F, respectively). Additionally, the findings of these white papers were incorporated into Guidance Manual recommendations related to site investigation/characterization (Section 3.4), feasibility and desirability evaluation (Section 4.3 and 5.2), VRA prioritization and selection (Section 5.3), and conceptual design approaches (Section 5.4).

Task 5: Develop Guidance for Implementing Volume Reduction Approaches

The outcome of this task was the completion of the Guidance Manual, including arrangement of the work products from previous tasks into a step-by-step approach for using the Guidance Manual.

Subtask 5.1: Compile Guidance and Develop Step-by-Step Approach and Recommended Use

Based on the intermediate outcomes of Task 1 through 4, the research team developed a stepwise approach to guide practitioners through the application of the recommendations contained in the Guidance Manual. This approach was discussed at an interim meeting with the Project Panel and general agreement with it was obtained. This approach was then used to structure the Guidance Manual. The stepwise approach itself is described in Chapter 2 of the Guidance Manual. Later chapters refer back to the steps identified in the stepwise approach.

Subtask 5.2: Introduce Watershed Scale Approaches

Recognizing the limitations of on-site VRAs (i.e., approaches within the project boundary) for many constrained urban highway environments, the research team developed a supplemental section of the Guidance Manual (Section 5.5) to introduce watershed-based alternatives for achieving volume reductions. This section introduces the topic of watershed-based alternatives, provides an inventory of the types of alternatives that may be available, and provides a list of resources related to watershed approaches. This section was informed by the draft findings of NCHRP Project 25-37, *Watershed Approaches to Mitigating Stormwater Impacts*.

Subtask 5.3: Develop Approaches for Enhancing Feasibility of Volume Reduction Approaches

Due to the highly constrained nature of the urban highway environment, many VRAs may not be feasible in many cases. The purpose of the Guidance Manual was to identify the conditions that render VRAs infeasible, but also to identify ways in which volume reduction could be feasibly achieved in a broader range of site conditions and project types. As part of this overall research effort, the research team provided various forms of guidance for enhancing the feasibility of volume reduction, including (with reference to Guidance Manual section in parentheses):

- Site assessment approaches to help better characterize site conditions and thereby improve the opportunity to identify locations applicable for VRAs (Section 3.4; Appendix C).
- Site design approaches to improve space availability and site conditions for achieving volume reduction and/or help mitigate site constraints (Section 4.2).

- VRA design adaptations to improve volume reduction performance and/or applicability to the urban highway environment (Section 4.2; Appendix A).
- VRA design measures to mitigate or eliminate feasibility constraints (Appendix A, D, E, F).
- Other options to improve volume reduction performance and/or reduce VRA cost to help reconcile performance and cost goals as part of conceptual design (Section 5.4.6).

Subtask 5.4: "Test Drive" the Guidance and Volume Performance Tool

Introduction

To obtain feedback from potential users on the Guidance Manual and to facilitate its adoption, the research team conducted "test drives" of the Guidance Manual and the Volume Performance Tool with two state level DOTs. Test drives with Washington State Department of Transportation (WSDOT) and District of Columbia Department of Transportation (DDOT) were conducted on February 19, 2014, and February 27, 2014, respectively. Both test drives included an introductory presentation about the purpose of the Guidance Manual and the Tool and an explanation of their context. This was followed by a demonstration of the Volume Performance Tool for example project scenarios. Finally, the research team facilitated discussions about potential improvements to the Guidance Manual and/or the Tool. Agency staff were given the opportunity to review both elements prior to the test drive and provide their comments during and after the test drives. The following agency staff participated in these test drives:

<u>WSDOT</u>
Mark Maurer
Alex Nguyen
Le Nguyen
Ebrahim Sahari

<u>DDOT</u>
Meredith Upchurch
Reginald Arno
Kyle Ohlson
Alit Balk
Carmen Franks

Key Input and Revisions Resulting from Test Drives

The following paragraphs summarize key points of discussion and the resulting changes that were made to the Guidance Manual and Tool.

Guidance Manual

- Both WSDOT and DDOT have already been implementing volume reduction approaches to some extent and have certain processes in place for making decisions about applying VRAs as part of project design. Therefore certain elements of the Guidance Manual were not as valuable to these agencies as they may be for a DOT that is at an earlier stage in evaluating volume reduction approaches. Specifically, several participants did not find the introduction, stepwise process, and regulatory background to be as useful for them and/or they did not understand the goals of these parts of the Guidance Manual as it applies to their situation. However, they also recognized that other DOTs would potentially benefit from these parts of the Guidance Manual. To address these comments, the research team included additional explanation in the introduction regarding how the Guidance Manual is intended to be used by different audiences, including DOTs that are earlier in the process of evaluating and implementing VRAs (or stormwater management approaches in general), and those that already have an established program.

- There was agreement by the test drive participants that the rest of the Guidance Manual (after the introduction, stepwise process, and regulatory background) was valuable regardless of DOT knowledge or presence of an existing program.
- The White Papers included in as Appendices were considered to be valuable for topic-specific details and reference.
- Overall, there were relatively few comments on the Guidance Manual from the test drive participants.

Volume Performance Tool

- The Tool was generally well received. Both WSDOT and DDOT personnel felt it was easy to use and quickly provided estimates of the volume reduction provided by a given VRA design. The sensitivity analysis included in the Tool was also considered be valuable.
- WSDOT staff were concerned that the level of precision that the Tool allows to be entered for infiltration rate was inconsistent with precision of typical infiltration measurement techniques. The research team agreed that this is the case. It was discussed that that a user should always be aware of the uncertainty in a given input when using any type of model. Discussion of appropriate user supplied inputs to the Tool has been added to the User's Manual accompanying the Tool.
- DDOT's current regulatory stormwater requirements are to retain the runoff from the 90th percentile, 24-hour storm via infiltration, ET, or harvest and use to the maximum extent practicable. In this local regulatory context, the Tool is not intended to be used as a regulatory compliance tool. In order to serve a regulatory compliance purpose, a tool would need to implement locally-acceptable sizing/design calculations and incorporate local feasibility criteria to demonstrate jurisdiction-specific regulatory compliance. The Tool was developed for a nationwide audience, therefore could not conform to each potential local regulation. Rather the Tool is intended to estimate the average long term volume reduction achieved by a given VRA conceptual design or set of conceptual design alternatives. The research team demonstrated that once design volumes are developed to meet the local regulatory requirements, the Tool can be used to rapidly evaluate the performance of different permutations of VRA conceptual designs that meet local standard and/or evaluate sensitivity of these parameters. DOTs that have sizing requirements based on a given level of long term capture (for example, 80 percent average annual capture efficiency) would potentially be able to use the Tool directly for sizing.
- It was discussed that one potential use of the Tool was to determine the benefit a VRA can provide toward achieving TDML waste load allocations (WLAs). Follow up questions asked where these calculations were found with this Tool. It has been clarified that the Tool does not perform water quality calculations, however if data are available to describe the average runoff quality from the catchment, this can be used along with the volume of runoff reduced to determine the pollutant load reduced via volume reduction. For additional information on water

quality calculations, the resulting guidance and tools from NCHRP Project 25-40 provide more information and water quality calculations.

- The Tool currently accepts inputs in the form of the design volume, flowrate, or tributary area ratio and translates this input (with the user-defined geometric assumptions specific to a VRA) into a VRA footprint area. Test drive participants suggested that it would be helpful if there was also an option for the Tool to allow inputs expressed in terms of the VRA footprint. In different applications, the user may have one input or the other. Within the Excel environment, it was not feasible to allow both forms of input within the project scope. However, to address this comment, the research team reorganized the input forms for VRAs so that it is easier to see how the calculated footprint changes as the primary input (design volume, flow rate, or tributary area ratio) changes. This will allow more rapid iteration to adjust VRAs input parameters to determine footprints that fit within the given available space.
- Test drive participants suggested that more guidance would be helpful about the range of input parameters that could be safely selected to avoid warnings within the Tool. For example, when a given VRA scenario is outside the bounds of the underlying lookup database, a warning message is returned. To address this, we included a new column with calculations showing the "recommended range" for key input parameters. Selecting a value within these parameters prevents warnings from occurring. These ranges are dynamic and update as other parameter inputs change.
- As part of the participants' beta testing period with the Tool, a few calculation errors were identified. These errors have been corrected.
- Some test driver participants wondered whether there was a way to make the Tool fit better on most monitors. In some cases, on smaller monitors, it is necessary to scroll both laterally and vertically to see the full screen while still keeping text size readable. We recognize this limitation, however this comment could not be addressed at this time because it would require changing the underlying structure of the Tool – potentially adding more tabs/steps and/or migrating out of the standard Excel interface. The Tool provides flexibility to change many inputs, provides guidance for these inputs, and provides schematics to help understand VRA conceptual design parameters – i.e., there is a lot of information to display. The user is able to hide the Guidance column of the Project Design tab (using native Excel commands), which helps the form fit into narrower or smaller monitors.
- Test drive participants appreciated the guidance that was embedded within the Tool and wanted it to be kept.
- Generally, the test drive participants thought the cells that contain calculations should be locked to prevent accidental overwriting.

Summary

In general the test drives achieved their goals of receiving feedback that we believe helped make the Guidance Manual and the Tool more effective. It was not possible to accommodate all requests; however, substantial improvements were made as a result of suggestions from test drive participants.

References to Final Work Products

The completion of Task 5 resulted in the preparation of:

- Preliminary Draft Final Guidance as a stand-alone document.

- Preliminary Draft Final Report documenting the entire research approach that includes an executive summary of the research results.
- Revised Final Guidance following one round of Panel comments; including response to comments.
- Final Revised Report following one round of Panel comments; including response to comments.

CHAPTER 3

Findings and Applications

Summary of Findings

The purpose of this project was to develop guidelines for transportation agencies on implementing volume reduction practices that are specific to limited access urban highways and specific to the conditions encountered as part of each project. As such, it was not the intent of this project to reach categorical conclusions about volume reduction or define a single preferred approach for achieving volume reduction. Rather, this project focused on developing a process, with supporting information and user tools, to assist highway professionals with identifying, evaluating, and designing feasible solutions for runoff volume management based on site-specific factors. The following paragraphs summarize key findings of this research and provide reference to how these findings were incorporated into the Guidance Manual. These summaries are not intended to be comprehensive; full detail is provided in the Guidance Manual.

Regulatory Conditions

There are various current mandates and emerging trends for volume reduction of stormwater runoff from urban highways, including:

- Total Maximum Daily Loads (TMDLs)
- Municipal separated storm sewer system (MS4) permits
- The Endangered Species Act
- Moving Ahead for Progress in the 21st Century Act
- Local flood control and/or channel protection requirements
- Water conservations and groundwater augmentation initiatives

Additionally, there are a variety of other design considerations that apply in the urban highway environment, including:

- Highway geometric design standards
- Vegetation and landscaping standards
- Highway safety standards and volume reduction
- Highway drainage and flood management

The combination of regulatory requirements that apply to a given project will vary by state, locality, and receiving water. More information and guidance on regulatory conditions is provided in Section 1.2 and 3.1 of the Guidance Manual.

Key Factors for Volume Reduction

Volume reduction of highway runoff refers to reduction of the amount of highway and right-of-way runoff that discharges directly either via overland flow or channelized flow or subsurface pipe flow to streams, rivers, lakes, the ocean or other water bodies. Therefore, to achieve volume reduction, water must be discharged to other hydrologic pathways. These pathways include:

- Evapotranspiration of water from the vegetation canopy or the ground surface (evaporation) and/or through the respiration cycle of plants (transpiration).
- Infiltration into the ground surface, followed by transpiration via plant uptake, return flow into streams, and/or percolation to deeper groundwater.
- Harvest and use of stormwater to meet demand for non-potable water on-site or in nearby locations.
- Controlled discharge of treated stormwater at rates that mimic baseflow recession characteristics of local streams (referred to as “hydrologically-referenced discharge”).

The effectiveness of a VRA for achieving volume reduction is primarily a function of (1) the capacity of the VRA to capture and store stormwater runoff and (2) the ability of the volume reduction processes associated with the VRA (i.e., infiltration, ET, and/or harvest and use) to recover the storage capacity of the VRA during and between storm events.

In addition to these fundamental performance factors, the effectiveness, feasibility, and desirability of volume reduction is influenced by both site characteristics and project characteristics. Site characteristics influencing volume reduction effectiveness, feasibility, and desirability include: local climate, soil and geologic characteristics and conditions, groundwater conditions, topography and existing site grading, watershed characteristics, project location in watershed, and adjacent land uses. Project characteristics influencing volume reduction effectiveness, feasibility, and desirability include: project type, highway type, amount of open space in medians and shoulders, shoulder width and usage, interchange spacing and type, proposed grading and drainage, highway landscaping/ vegetation, and maintenance access.

Section 3.2 of the Guidance Manual provides more information about these factors. Section 3.4 provides guidance for conducting site characterization efforts related to these factors.

Physical Characteristics of the Highway Environment

Urban highways vary significantly in their attributes and physical settings relative to achieving volume reduction. For the purpose of identifying key physical attributes, the Guidance Manual categorizes urban highways into eight (8) different representative highway types based upon geometric design variations typical of urban freeway design.

1. Ground-level highway segments
2. Ground-level highway segments with restricted cross-sections
3. Highway segments on steep transverse slopes
4. Depressed highway segments
5. Elevated highway segments constructed on embankments
6. Elevated highway segments constructed on viaducts
7. Linear interchanges
8. Looped interchanges

The definition of these highway types for the Guidance Manual was intended to help the user identify the constraints and opportunities for volume reduction that are specific to the standard highway types that are most similar to the conditions on a given project. More information about these highway types and their attributes in relation to volume reduction is provided in Section 3.3 of the Guidance Manual.

Site Assessment Activities to Support Volume Reduction Planning and Design

Site conditions have an important influence on the amount of volume reduction that may be achievable as well as the types and locations of VRAs that may be applicable. Assessing the potential of a site for the implementation of VRAs requires the review of existing information and may include the collection of site-specific measurements. A fundamental purpose of the Guidance Manual is to provide users with guidance on what information to collect and how to interpret this information. This information is critical for developing a site-specific approach for volume reduction, or determining and demonstrating when volume reduction is not appropriate.

Site assessment efforts should ideally be initiated early in the planning and design process so that VRAs can be incorporated into the project layout as it is developed. To improve the efficiency of site assessment, efforts should be phased as appropriate, with consideration of the information needed to inform decisions at each point in the project. Types of investigation methods tend to be different for early planning efforts compared to more detailed design efforts. Planning phase activities are typically intended to provide an overall characterization of volume reduction potential for the project as well as to prioritize areas where volume reduction may be feasible and effective. The role of design level investigations are to refine the results of planning level investigations, where necessary, to provide more robust information regarding the feasibility and desirability of a VRA in a given as-designed location, as well as support design-level parameters such as the long term reliable design infiltration rate.

Section 3.4 of the Guidance Manual provides guidance for conducting site assessments to support volume reduction planning and design. This section includes guidance on phasing of site assessment activities as well as guidance on the following site assessment topics:

- Topography and drainage patterns
- Off-site drainage and adjacent land uses
- Soil and geologic conditions
- Local weather patterns
- Groundwater considerations
- Geotechnical considerations
- Existing utilities
- Harvested water demand
- Responsible agencies and other stakeholders
- Local ordinances
- Watershed-based and other joint planning opportunities

Additionally, Appendix C, D, and E provide more detailed topic-specific guidance on infiltration rate evaluation and factors of safety (Appendix C), water balance and groundwater quality issues (Appendix D), and geotechnical issues (Appendix E).

Menu of Volume Reduction Approaches

Section 4.1 of the Guidance Manual provides a summary of existing stormwater control measures (SCMs) in use in the highway environment, including an evaluation of which of these SCMs provide significant volume reduction potential. Additionally, Section 4.1 reviews new and emerging concepts in volume reduction. Based on this review, a primary menu of VRAs was developed and described in Chapter 4.2 of the Guidance Manual, including:

- VRA 01 Vegetated Conveyance
- VRA 02 Dispersion
- VRA 03 Media Filter Drain
- VRA 04 Permeable Shoulders with Stone Reservoirs
- VRA 05 Bioretention without Underdrains
- VRA 06 Bioretention with Underdrains
- VRA 07 Infiltration Trench
- VRA 08 Infiltration Basin
- VRA 09 Infiltration Gallery

Other potential volume reduction concepts and approaches were also identified and described in Section 4.2 of the Guidance Manual, including:

- Harvest and use and land application
- Incidental volume reduction in other SCMs
- Real-time control of outlets for enhanced volume reduction performance and/or performance monitoring
- Hydrologically-referenced discharge to mimic natural hydrology

Finally, site planning approaches to reduce runoff volume were identified. These include approaches intended to reduce the amount of runoff that occurs, as well as approaches intended to complement the primary menu of VRAs (above) by creating opportunities to site these VRAs within projects. Site planning approaches described in Section 4.2 of the Guidance Manual include:

- Early identification of VRA opportunity locations
- Develop drainage, grading, and utility configurations to accommodate VRA opportunity locations
- Limit footprint of disturbance
- Minimize non-essential impervious surface
- Conserve and/or amend topsoil

Comparison of VRA Attributes

Each VRA has a distinct set of attributes and considerations that are inherent in its design and function. Fact sheets provided in Attachment A of the Guidance Manual provide an extended summary of each of the primary VRAs. Section 4.3 of the Guidance Manual provides summaries of key attributes of each primary VRA to facilitate comparison between VRAs as well as to serve as a concise reference. Section 4.3 provides guidance regarding:

- Volume reduction mechanisms and potential water balance issues by VRA
- Geometric siting opportunities and footprint requirements by VRA
- Relative potential geotechnical impacts by VRA

- Relative potential risk of groundwater quality impacts by VRA
- Safety considerations by VRA
- Maintenance activities by VRA

Additionally, Section 4.4 of the Guidance Manual provides additional references for VRA design and maintenance information, including selected nationwide guidance and selected state-specific DOT guidance.

Framework for Selecting and Applying VRAs

The Guidance Manual (Section 5.1) describes an overall framework for volume reduction planning and conceptual design. This framework was developed with the intent of assimilating a number of factors, including (1) project goals, (2) site and watershed information, and (3) the available menu of VRAs, to yield a preferred plan for achieving volume reduction. The outcome of this framework is intended to yield a volume reduction plan that meets the following goals:

- Is applicable to the project type
- Is feasible and desirable given site and watershed conditions
- Utilizes VRAs that are compatible with the project site as well as the future redevelopment projections for the project area
- Can be reliably and safely operated and maintained over the long term
- Is consistent with project economic constraints
- Meets project volume reduction goals

In general, the process of developing a volume reduction plan can be considered in three phases:

- Initial screening to identify potential VRAs that are potentially applicable, feasible and desirable (Section 5.2)
- Prioritization of these VRAs on a relative basis (Section 5.3)
- Conceptual design evaluation relative to project goals and constraints (Section 5.4)

VRA Screening (Applicability, Feasibility, and Desirability)

The Guidance Manual (Section 5.2) describes a tiered evaluation process for identifying potential VRAs for a project. This process involves screening to evaluate applicability, feasibility, and desirability. In general, applicability, feasibility, and desirability can be assessed by asking four fundamental questions:

- **Is a certain VRA applicable to the project?** For example, determining the applicability of a VRA can include geometric requirements for VRAs, availability of space, presence of a storm drain system, presence of demand for harvested water, and other factors.
- **Is it physically possible to implement a certain VRA based on the site conditions?** For example, do soil or geologic (i.e., bedrock, etc.) conditions render infiltration rates negligible? Does the site layout present no opportunity for a specific type of VRA?
- **Would the use of a certain VRA have the potential to result in undesirable physical consequences on the project or the site environs?** For example, would the use of a VRA pose an unacceptable elevated risk of groundwater contamination or movement of a plume? Or

would infiltration in excess of natural conditions potentially cause geotechnical issues or downgradient habitat concerns?

- **Does the cost required to construct the VRA and/or mitigate potential risks posed by the VRA outweigh the volume control benefits it would achieve?** For example, it may be physically possible to infiltrate water into clay soils at some small level and possible to mitigate soil stability issues associated with infiltration, however the resulting benefit may not warrant the added project expense of additional area consumed and/or costs.

Section 5.2 of the Guidance Manual provides guidance for evaluating and documenting the answers to each of these questions.

Assimilate Screening Results to Identify Potentially Suitable VRAs

Based on the results the initial screening of VRAs (Section 5.2), the Guidance Manual provides guidance for selecting VRAs that are compatible with the feasibility and desirability screening conditions that are identified. The feasibility of infiltration was used as the primary basis for determining which types of VRAs may be applicable, including classifying infiltration conditions into the following general management categories:

- Full Infiltration Feasible: Select and design VRAs with emphasis on providing reliable infiltration.
- Partial Infiltration Feasible, volume reduction supported by processes other than infiltration: Select and design VRAs to promote allowable level of infiltration and maximize ET.
- Limited or No Infiltration Feasible, volume reduction achieved primarily through other processes: Select VRAs to limit infiltration and provide ET, harvesting, and/or treated baseflow mimicking discharge, as applicable.

Further discussion of these categories of screening conditions is provided in Section 5.2 of the Guidance Manual with associated recommended VRA types for each category.

VRA Prioritization

After identification of potentially feasible and applicable VRAs, the Guidance Manual describes a systematic approach to prioritization of VRAs. This process generally consists of weighting and scoring VRAs based on the following factors:

- Relative whole life cycle costs
- Relative O&M impacts to agencies
- Relative reliability
- Relative safety
- Potential performance relative to volume reduction goals

Guidance for prioritizing VRAs based on each of these factors is provided in Section 5.3 of the Guidance Manual

VRA Conceptual Design

Volume reduction performance is a function of many factors, including site and watershed conditions, climate patterns, VRA sizing and design parameters, and other factors. Similarly, costs are site-specific as a function of project type (i.e., new vs. redevelopment vs. retrofit), existing infrastructure in place, material costs, construction methods and other factors. As such, a conceptual design level analysis is necessary to provide a reliable evaluation of performance and cost in comparison to other VRAs and in comparison to project goals and constraints. The conceptual design development process may be an iterative process that evaluates a range of scenarios and adapts these scenarios iteratively to identify the conceptual design that best balances volume reduction goals with site and cost constraints. Section 5.4 of the Guidance Manual provides guidance for conceptual; design development, consisting of the following general steps:

- Developing initial conceptual designs
- Using modeling tools (e.g., the Volume Performance Tool) for decision support and conceptual design adaptation
- Estimating whole lifecycle costs of conceptual designs
- Adapting conceptual designs to converge with project goals and constraints

Watershed-scale Approaches

A final section of the Guidance Manual (Section 5.5) was prepared to provide users with an introduction to watershed approaches for achieving volume reduction goals for urban highway runoff. This section was also intended to help identify when a watershed approach may be more appropriate than SCMs or VRAs located within the project site (referred to as “on-site” VRAs or SCMs). Alternatives introduced in Section 5.5 include:

- Watershed-scale management of project runoff – VRA or SCM located outside of the project boundary, but receiving runoff from the project and potentially additional areas; project runoff is managed before discharge to a receiving water.
- In-kind management of other highway runoff – VRAs or SCMs installed to manage runoff from nearby section of roadway, but outside of project site; does not manage runoff from the project.
- In-kind management of non-highway runoff – VRAs or SCMs installed to manage runoff from nearby non-roadway land uses, outside of project site; does not manage runoff from the project.
- Out of kind mitigation banking - A general category used to refer to other approaches for achieving equivalent watershed protection benefits compared to on-site VRAs or SCEM, but possibly using different approaches/mechanisms.

Intended Applications

Incorporating VRAs into the highway project development process often add complexity and introduces additional feasibility and desirability considerations when compared to a standard highway stormwater management design. Highway professionals require practical information that will help them identify, evaluate, and design feasible solutions for runoff volume management. The Guidance Manual is intended to provide a process and supporting information to identify controls that are effective, reliable, and applicable for the site, as well as provide a clear technical basis as to why some or all volume reduction controls will not meet these criteria. The Guidance Manual is intended to be immediately applicable to practice, for use by DOTs and other highway environmental professionals.

The Guidance Manual is intended to be used by a range of user types, for different purposes in the planning and design process for new projects, lane addition projects, and retrofit projects. At the early project planning level and program management level, the Guidance Manual can be used to help facilitate a mutual understanding amongst the design team regarding volume reduction goals and the way that volume reduction considerations will be incorporated into the project development process. Similarly, the Guidance Manual can be used to help scope the additional or alternative analyses that may be needed in the design process as part of achieving volume reduction or demonstrating its infeasibility. At later stages of planning, the Guidance Manual is intended to assist in identifying potentially feasible VRAs and conducting early site investigations to identify project-specific opportunities and constraints to allow for refined estimates of achievable levels of volume reduction. Finally, the Guidance Manual is intended to support designers with site-specific approaches to prioritize, select, evaluate and apply VRAs. The Guidance Manual may also serve as a resource for permit writers and compliance staff when considering the level of volume reduction that may be achievable in the urban highway environment and potential adverse impacts associated with volume reduction designs.

While the Guidance Manual provides detailed and methodical guidance for selecting and implementing VRAs, supplemented with checklists and schematics consistent with a “conceptual design” level of detail, it does not provide criteria for detailed design of specific volume reduction facilities. Additionally, the Guidance Manual considers operations and maintenance activities and costs as key considerations in the feasibility and prioritization of VRAs, but it does not provide detailed guidance for operations and maintenance. The Guidance Manual provides references to other documents that provide more information in these areas.

More information about the intended applications of the Guidance Manual can be found in Section 1.3 and Chapter 2 of the Guidance Manual.

CHAPTER 4

Conclusions and Suggested Research

Conclusions

DOTs are facing increasing requirements to reduce the volume of stormwater runoff from highways that is immediately or ultimately discharged to surface waters via overland flow or a constructed above or below ground conveyance. Implementing stormwater VRAs in highly constrained urban environment presents a number of challenges and constraints due to the limited space available in the right-of-way for infiltration, evapotranspiration, on-site use, or flow control. These constraints are confounded by road safety requirements, geotechnical and structural concerns associated with saturated soils in the near highway environment, costs to construct VRAs, cost and ability to maintain VRAs, and other factors. Finally, the effectiveness of VRAs is an important factor in determining whether costs of VRAs justify the benefits of reducing runoff volume.

This project conducted research into approaches for achieving volume reduction, including both established and innovative approaches. One of the overarching findings of this research was that achieving volume reduction of urban highway runoff is dependent on many site-specific factors and considerations. The overwhelming importance of these factors means that it was not possible to reach categorical conclusions about the feasibility, effectiveness, and/or cost of achieving volume reduction that are applicable across site and watershed conditions, projects types, design goals and various other site-specific factors. Site-specific conclusions must be supported by careful evaluation of each of these factors. As such, the primary outcome of this research was the development of a process and supporting references (as provided in the Guidance Manual) by which practitioners can identify, evaluate, and design feasible solutions for runoff volume management based on project-specific conditions. By applying a methodical process to volume reduction planning and design (such as described in the Guidance Manual), it may be possible to identify more opportunities to achieve volume reduction while also more carefully identifying conditions in which volume reduction is not appropriate.

Suggested Research

As part of this research, the research team identified several areas that are of specific importance for volume reduction planning and design which are not currently well-supported or understood by the body of science. A summary of these general areas of suggested research are identified and discussed below.

Prioritization of volume reduction and pollutant removal goals – Regulatory trends reviewed as part of this effort include an increased emphasis on volume control as the first priority for controlling stormwater runoff. However, the preference for volume control compared to equivalently protective alternatives has not been clearly and consistently demonstrated. For example, discharge of runoff volume itself to a large and geomorphically-stable waterbody does not necessarily have an impact on that water body, except potentially as a function of the pollutants that may be transported with this volume. In this case, it is not clear to the research team that a regulatory preference for volume control is necessary in comparison to an alternative approach that provides the necessary removal and control of pollutants. In addition, regulations that have included a hierarchy of considering volume reduction as the top priority also typically specify a design storm (e.g., the 85th percentile, 24-hour storm event) that may or may not have a correlation to pre-development hydrology. Further research on costs and benefits of control approaches (including both volume reduction and non-volume reduction approaches) could be conducted to help determine conditions in which a strict hierarchy of control approaches is appropriate (i.e., evaluating volume reduction first) and where this strict hierarchy may unnecessarily complicate the project development process and/or impose cost burdens that are not justified by stormwater control benefits. Additionally, research could be conducted on whether retention of a design storm provides an adequate correlation to pre-development hydrology. In some situations, increased infiltration of runoff over natural levels may cause other impacts. Additional research of the potential impacts of a watershed wide preference to reduce surface runoff volumes should also be evaluated. Guidance on including infiltration and groundwater evaluations in watershed planning efforts would be helpful.

Maintenance needs and lifespan of VRAs – The research team found that there is relatively little empirical data available to the general scientific community about the maintenance requirements of VRAs (and SCMs in general), particularly in a uniform scientific format. Similarly, there is little empirical information about the lifespan of VRAs and how VRA design decisions (e.g., pretreatment, footprint size) influences lifespan. Research could be conducted to more fully evaluate the available data that may be available directly from DOTs and potentially identify a standard reporting protocol(s) for VRA/SCM maintenance activities. For the lifespan of VRAs and in particular those that rely on infiltration, long-term performance data on infiltration rates and how they change with overall time, seasonally, and with changing tributary watershed conditions (for example, reconstruction, lane widening, change in deicing practices, etc.) would be helpful. The long-term deployment of soil moisture probes within infiltration media for example to look at changes in the wetting fronts over time would be very valuable. NCHRP Project 25-40 evaluated long term performance and cost of a number of stormwater control measures, including some VRAs, and helps begin to address this research need.

Whole lifecycle costs of VRAs – There remains relatively little empirical information about the whole lifecycle cost of VRAs/SCMs, particularly in a consistently-reported format that allows direct comparison between different studies and VRA/SCM types. Studies that reported capital cost tended to differ in terms of what components of the design are considered to be attributable to the VRA/SCM cost as well as how avoided costs are considered (for example, the avoided cost of traditional pavement through the use of permeable pavement). Research could be conducted to expand the inventory of costs studies available, refine past studies to result in more comparable reporting, and/or develop a standard reporting protocol for VRA/SCM costs. NCHRP Project 25-40 evaluated whole lifecycle costs of a number of stormwater control measures, including some VRAs, and helps being to address this research need.

Reliability of infiltration measurement techniques – The reliable long term infiltration rate of a VRA is a key factor in volume reduction planning and design. However, many infiltration measurement methods that are currently in use for stormwater design were originally intended for different purposes (e.g., water well development, septic system leach field design). While some research has been completed in this area (see references in Appendix C of the Guidance Manual), there remains an opportunity to conduct research to better understand the transferability and reliability of these measurements to the long term reliable rates expected in full scale VRAs.

Full scale experience with permeable shoulders and travel lanes – To date, there is relatively limited full scale experience with permeable pavement technologies in the highway environment. As such, significant uncertainty remains about the longevity, durability, and maintenance needs of permeable pavements in this environment. Permeable pavements may present a significant opportunity to achieve volume reduction in constrained urban environments, and their broader application could be facilitated with more full scale, long term research. Appendix F of the Guidance Manual also provides a more extensive list of suggested research related to permeable pavement.

Annex A

Volume Performance Tool

Technical Documentation

1 Introduction and Purpose

The performance of volume reduction approaches (VRAs) is a function of many factors, including local climate and hydrology, storage volume, VRA design (i.e., footprint, depth, and discharge rates), underlying soil properties, and other factors. Because volume reduction occurs to different degrees in storms with different sizes, shapes, and antecedent conditions, it is necessary to utilize long term hydrologic and hydraulic modeling methods (i.e., continuous simulation, rather than design event simulation) to provide a reliable estimate of long term volume reduction.

The Volume Performance Tool (tool) was developed as part of NCHRP Project 25-41 *Guidance for Achieving Volume Reduction of Highway Runoff in Urban Areas* to allow a user to estimate the approximate volume reduction performance of a VRA or series of VRAs (i.e., treatment train), given the site location and planning level information about the site conditions and the VRA design. The purpose of this annex is to provide technical documentation for the Volume Performance Tool.

2 Model Overview

2.1 Functionality and Applicability

The tool is intended to be used to evaluate a wide variety of single and multi-component VRA design scenarios for a user-defined location. To provide "real-time" estimates of volume reduction performance (i.e., immediately available, without creating and executing a continuous simulation model run), the tool queries the results of thousands of pre-executed long term hydrologic and hydraulic model simulations executed as part of the development of the tool. Using scaling and interpolation methods developed for this purpose (as discussed further in Sections 3.5 and 3.6), site-specific estimates can be derived for a specific design scenario based on a pre-determined array of hypothetical simulations conducted at the location of gages within each climate division. The tool itself is an Excel spreadsheet application in which the user selects a location, provides planning level project information and the tool provides an estimate of long term volume reduction. The tool is intended to allow DOTs to quickly evaluate the relative benefits of various conceptual design scenarios as well as to assist in develop sizing criteria.

The tool is not intended solely for highway projects; however, assumptions have been made as part of the development of the tool such that the tool is expected to be most reliable for the conditions typically encountered in highway environment (specifically smaller catchments with higher percentage impervious area). Volume reduction performance of VRAs does not generally exhibit significant sensitivity to catchment size, as volume reduction processes tend to occur at somewhat longer timescales than peak flow phenomena; therefore the tool is considered to be reliable for planning level analysis of other types of catchments as well (e.g., larger, less impervious catchments).

The tool is intended to provide a planning level estimate of volume reduction performance to assist in prioritizing VRAs at a planning level. Where more rigorous site-specific estimates of volume reduction performance are desired, the results of the tool should be refined through the use of a more detailed model that is developed specifically to represent local conditions.

2.2 User Experience

The tool is intended to be accessible to a wide variety of practitioners around the country without significant prior effort required in reviewing user guidance or developing inputs. The tool consists of a macro-enabled Excel spreadsheet that contains a “lookup database” – a directory of data files that contain summary statistics from the long term hydrologic and hydraulic model runs completed as part of developing the tool. The user navigates through the tool using navigation buttons or tabs, populating information about the specific project and VRA scenario and viewing estimated volume performance results. The User’s Guide for the tool (Appendix B of the Guidance Manual) provides step by step guidance for using the tool and provides more explanation of the user interface and experience.

3 Technical Framework

3.1 Volume Reduction Mechanisms

The total amount of volume reduction achieved in a VRA is a function of the amount of water that enters the VRA and does not immediately overflow (i.e., the amount of water that is captured), and the portion of the captured water that is “lost” via infiltration, ET, and/or consumptive use (i.e., the total of all three is the volume reduction), such that it does not discharge directly to surface water.

When evaluating capture efficiency and volume reduction, each VRA can be considered to consist of a set of storage compartments, each with a distinct storage volume, discharge rate, and pathway by which water discharges (i.e., surface discharge, infiltration, ET). Figure 1 illustrates this concept. When storage capacity is available in a given compartment, then that compartment of the VRA can capture additional inflow. When storage capacity is not available in a given compartment to accept additional inflow, then inflowing water either fills the next storage compartment of the VRA, or bypasses the system (if no additional storage is available). The capture and volume reduction performance of a VRA is primarily a function of the amount of storage volume provided and the rate at which the storage drains to volume reduction pathways (i.e., infiltration, ET, consumptive use) versus surface discharge pathways.

The menu of VRAs that is supported as part of the *Guidance Manual* includes VRAs with retention storage only, as well as systems with retention and detention storage. In the case of systems with retention storage only (e.g., infiltration trenches), the volume reduction performance is a function of the capture efficiency only – all water that is captured is lost, as there is no surface discharge from these VRAs besides the overflow that occurs when retention storage is filled. In the case of VRAs with a combination of detention and retention storage (e.g., bioretention with underdrains), the total volume reduction performance is the product of the capture efficiency (the portion of overall runoff volume that is captured and managed) and the volume reduction efficiency (the portion of the captured water that is lost).

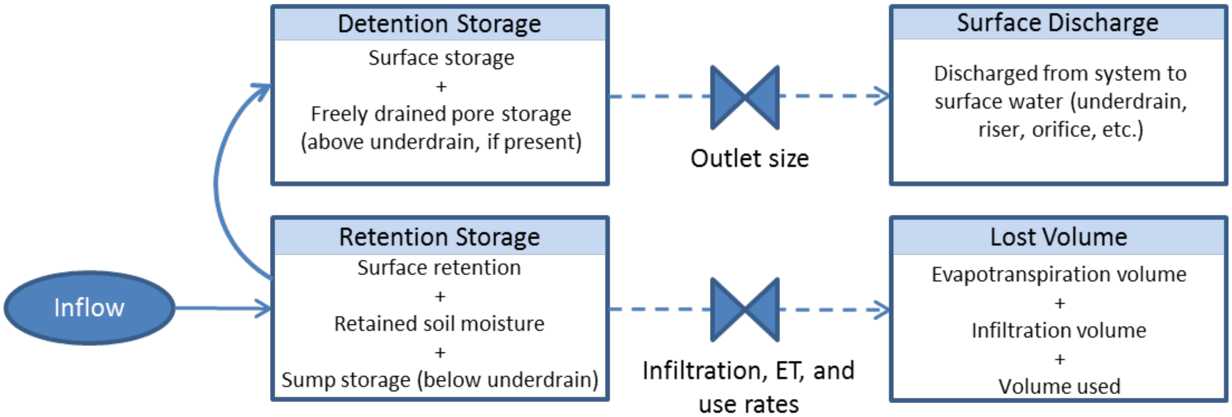

Figure 1. Schematic Representation of VRAs for Purpose of Capture Efficiency and Volume Reduction Analysis

3.2 Capture Efficiency

Capture efficiency (or “percent capture”) is a metric that measures the percent of rainfall that is captured and managed by a VRA (i.e., does not bypass or immediately overflow). Captured stormwater may be infiltrated, evapotranspired, or retained for harvest and use, and/or treated and release. Capture efficiency is typically expressed as an average capture rate over a long period of time, for example annual average percent capture. Runoff volume that is not captured by a VRA is referred to as bypass or overflow. Volume reduction processes can only occur in a VRA when water is captured.

Long term capture efficiency is primarily a function of the VRA storage volume (relative to the size and runoff potential of the watershed), the drawdown rate and pattern of the storage compartment, and local precipitation patterns. Practically, this means that the following parameters can be isolated as primary predictors of capture efficiency for the purpose of developing an approximate predictive tool:

- **Normalized storage volume**, expressed as an equivalent precipitation depth over the watershed that would produce a runoff volume equivalent to the VRA storage volume. For example, a 3,000 cu-ft storage volume for a watershed that is 1 acre with a runoff coefficient of 0.9 would translate to an equivalent precipitation depth of 0.92 inches [3,000 cu-ft × 12 in/ft / (1 ac × 43,560 sq-ft/ac × 0.9)].
- **Drawdown time** of the storage volume. For VRA storage elements with nominally consistent drawdown rates regardless of season (i.e., infiltration, filtration, orifice-controlled surface discharge), the representative drawdown time can be expressed in hours. For example, a bioretention area with a storage depth of 18 inches and an underlying design infiltration rate of 0.5 inches per hour would have a nominal drawdown time of 36 hours (18 inches / 0.5 in/hr). For VRA storage elements with seasonally varying drawdown rates (i.e., storage drained by ET or irrigation-based consumptive use), the concept of a representative drawdown time is not applicable. In this case, the ET storage depth (i.e., the amount of potential ET that must occur for the stored water to empty) is a more appropriate indicator of how quickly storage is recovered and can be used (along with local climate data input to the model) as a predictor of long term capture efficiency.

By isolating these two most important predictive variables, a limited number of continuous simulation model runs and associated results can be used to describe the expected long term performance of a wide range of VRA types and configurations. For example, the results of a long term model simulation for a 0.75-inch normalized storage depth with 24-hour drawdown would be representative of a wide range of different VRA configurations. The two examples would both be reliably represented by this single model run.

Example 1: 20,000 cu-ft infiltration basin draining 8.2 acres of pavement (equates to 0.75-inch equivalent storm), with 3-foot ponding depth and a design infiltration rate of 1.5 inches per hour (equates to 24 hour drawdown time).

Example 2: 300 cu-ft bioretention area with underdrains with a tributary area of 0.122 acres of pavement (equates to 0.75-inch equivalent storm), with 12 inches of ponding storage depth and a design media filtration rate of 0.5 inches per hour (equates to 24-hour drawdown time).

It be seen that an infinite number of potential design combinations could be reflected by this single model run.

An array of continuous simulation runs were executed in the EPA Storm Water Management Model (SWMM, version 5.0.022), as described further in Section 4, to encompass the range of normalized storage volumes and drawdown times (or ET depths) that the tool supports. For each of the combinations of storage volume and drawdown time (or ET depth), the capture efficiency was calculated using results from the long term SWMM model. For each combination of design variables, the percent capture was calculated as:

Percent Capture = $100[1 - (V_{by}/V_c)]$

Where: V_{by} = the total volume bypassed or overflowed over the simulation period

V_c = the total runoff volume flowing into the VRA over the simulation period

3.3 Volume Reduction Efficiency

Volume reduction efficiency refers to the portion of the “captured” volume that is lost to infiltration, ET, or consumptive use and does not discharge directly to surface water. Within the tool, the following assumptions were made:

- For storage compartments without a surface discharge pathway (i.e., retention storage), the volume reduction efficiency was set to 100 percent of the capture efficiency (i.e., complete retention of all water that is captured).
- For storage compartments with surface discharge as well as significant volume loss pathways, the volume reduction efficiency was estimated by computing the average loss rate as a fraction of the average total discharge rate. For example, if the average surface discharge rate during the drawdown period is 2 inches per hour and the average infiltration plus ET loss rate during that period is 0.5 inches per hour, then the volume reduction efficiency would be estimated as 20 percent (0.5 / (2 + 0.5)).
- For storage elements with only surface discharge pathways (i.e., lined systems with limited ET), the volume reduction efficiency was assumed to be zero.

3.4 Total Volume Reduction Performance

The total volume reduction performance was expressed in terms of:

- **Watershed relative volume reduction** – the relative reduction in surface discharge volume compared to the same tributary area without controls, calculated as:

  Watershed relative volume reduction = Σ (% Cap) × (%VolRed)

  Where:

  % Cap = long term average annual capture efficiency

  % VolRed = long term volume reduction as a percentage of captured volume

- **Watershed absolute volume reduction** – the difference in average annual runoff volume between the project condition without controls and the project condition with controls, calculated as:

  Watershed absolute volume reduction = Watershed relative volume reduction (%, computed per above) × Baseline Avg Annual Runoff Volume (without controls)

  Where:

  Baseline Avg Annual Runoff Volume (cu-ft) = Average Annual Precipitation Depth (inches) × Area (ac) × Runoff Coefficient / 12 inches/ft

3.5 Accounting for Multi-Compartment VRAs and Treatment Trains

Many VRAs include a combination of retention and detention storage compartments that fill and drain in different orders and at different rates, and therefore are best represented as two or more discrete storage compartments. For each storage compartment, the methods described above can be used to estimate capture efficiency and volume reduction for that individual compartment. However, because the response between storage volume and capture efficiency is non-linear (i.e., above some storage volume there is a trend of increasingly diminishing incremental returns with incremental addition of storage), it is not reliable to simply add the capture efficiency achieved by each compartment as if each was an independent VRA. Similarly, where two VRAs are to be placed in series, it is not reliable to analyze each independently and simply sum the independent results. Holding all else equal, the first VRA compartment will tend to provide greater incremental capture efficiency than the second, and so on.

To account for this non-linearity, a graphical/tabular method was implemented by a macro within the tool to estimate the total performance of multi-compartment VRAs and treatment trains, including compartments with different discharge pathways and different drain times for each discharge pathway. The method consists of the following steps and is illustrated graphically in Figure 2. These steps are conducted by the tool. The user is not responsible for implementing the steps listed below.

1. Order VRA compartments in terms of which compartment would fill first, before others would fill. For example, in the case of a bioretention area with a gravel sump below the underdrains, assume that the ET storage in soil pores would fill first and the sump would fill second before water would pool up into the detention storage compartments. Generally, it can be assumed that ET storage will fill first (as soil wets), infiltration storage will fill next (as the facility fills from the bottom up), and detention storage will be the last to fill.

2. For the first compartment, the tool computes the equivalent storm depth (i.e., the storm depth that would be required to produce runoff from the watershed equal to the storage volume of the compartment) and drawdown time based on user inputs and finds the corresponding percent capture by querying the applicable lookup database. For the purpose of this example, assume the first compartment is the stone sump below a bioretention area with elevated underdrains and has storage equal to the runoff from a storm depth of 0.5 inches and a drawdown time of 72 hours. This combination yields a capture efficiency of 27 percent from the example nomograph (Figure 2).
3. Calculate the storage volume (as an effective storm depth) and drawdown time for the second compartment. In this example, this is the detention storage above the underdrains of the facility. This compartment has storage equal to the runoff from a storm depth of 0.75 inches and drains in 12 hours.
4. Using the percent capture of 27 percent obtained from Step 2, find the equivalent storm depth from the lookup database corresponding to the drawdown time of the next storage compartment (12 hours). In this example, the 27 percent capture achieved by the first compartment corresponds to an equivalent precipitation depth of 0.2 inches for the 12-hour drawdown scenario.
5. Next, "traverse" the nomograph curve (or table) up the 12-hour drawdown line a distance corresponding to the storage volume provided in the next compartment. In the example below, the second compartment has a storage volume equivalent to storm depth of 0.75 inches and a drawdown time of 12 hours. Therefore, the nomograph is traversed from 0.20 to 0.95 inches in the x-axis, corresponding to an increase from 27 percent capture to 75 percent capture in the y-axis.
6. Find the corresponding percent capture from the current position on the nomograph/table. In this case, the two compartments collectively achieve approximately 75 percent capture (27 percent by the first compartment plus 48 percent by the second compartment).
7. Repeat steps 2-5 for each remaining BMP compartment or treatment train element.

The validity of this approach has been demonstrated as part of previous work (Orange County Public Works, 2011, Ventura County Watershed Protection District, 2011) and was tested and demonstrated for this specific application in comparison to more explicit model representations as part of development of the tool (See Section 7).

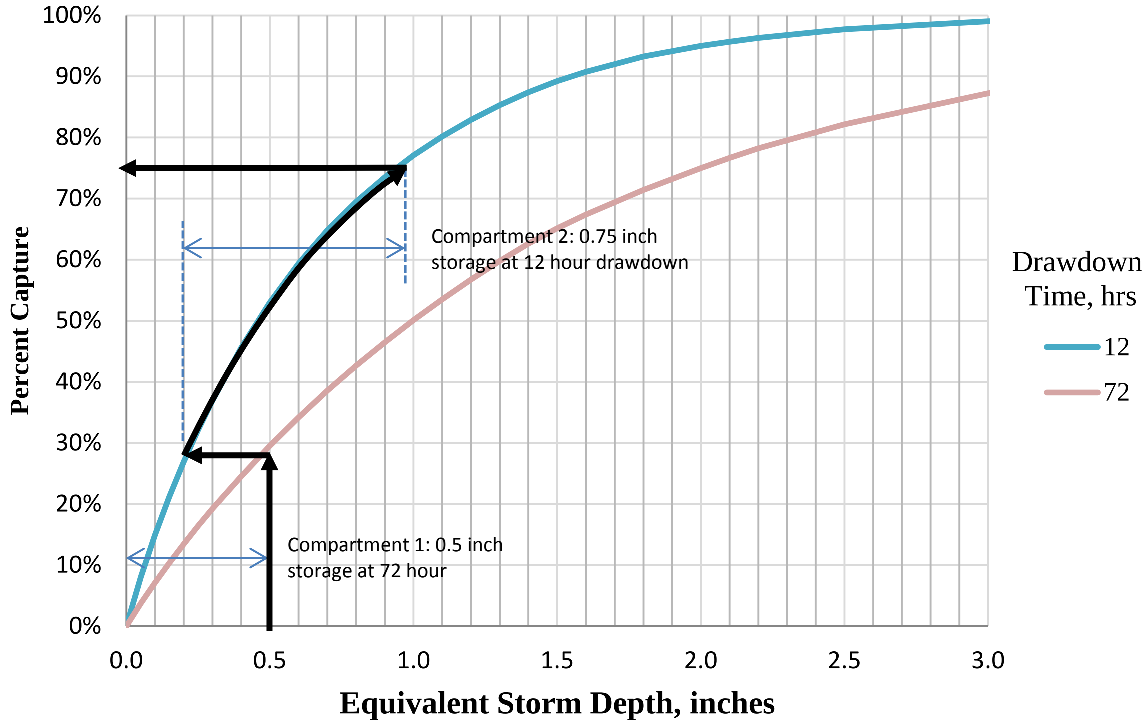

Figure 2: Example application of method for incorporating multi-compartment VRAs (hypothetical location)

3.6 Approach for Localizing Model Estimates

The tool was pre-packaged with lookup databases containing model results for 344 climate divisions (represented by a single point location precipitation/weather gage for each division) representing a wide range of conditions across the contiguous US. However, within each climate division there can be significant variability in precipitation patterns that may influence capture efficiency and volume reduction performance of VRAs. For example, within Los Angeles County alone, the average annual precipitation depth ranges from less than 10 inches up to more than 30 inches per year, and the 85th percentile, 24-hour storm depth (the local regulatory storm for sizing of water quality facilities) ranges from less than 0.75 inches to more than 1.5 inches (Los Angeles County Department of Public Works, 2012). As a result of intra-region variability, which likely occurs to different degrees in different climate divisions, there is potential for error to be introduced into the estimates provided by the tool, which are based on only one point location within each climate division. In areas of the climate division where storms are larger and/or more intense than the location of the gage that was analyzed, the tool would tend to over-estimate site-specific performance (actual performance would be less than what the tool predicts), and vice versa. However, it was not practicable to conduct continuous simulation at every possible project location because of the lack of reliable hourly data in some areas and the computational burden and associated quantity of data that would need to be pre-packaged with the tool.

To address this limitation, the research team developed and tested an approach for “localizing” estimates within each climate division to improve site-specific reliability. This approach was based on the finding that the 85th percentile, 24-hour storm depth provides a reliable scaling factor for translating model results within a climate division -- when the sizes of VRAs are scaled from point to point based on the relative magnitudes of the 85th percentile, 24-hour storm event at each point, then similarity in long term capture efficiency and volume reduction appear to be maintained. For example the capture efficiency would be expected to be similar for the following two points because their design volume as a fraction of the 85th percentile, 24-hour runoff volumes are the same:

Point A:

Runoff volume from 85th percentile, 24-hour storm = 3,200 cu-ft

Actual VRA sized for 2,000 cu-ft

Design volume as fraction of 85th percentile, 24-hour runoff volume = 2,000/3,200 = 0.625

Point B:

Runoff volume from 85th percentile, 24-hour storm = 5,000 cu-ft

Actual VRA sized for 3,125 cu-ft

Design volume as fraction of 85th percentile, 24-hour runoff volume = 3,125/5,000 = 0.625

In other words, in order to achieve the same capture efficiency at Point B as Point A, the actual VRA size would need to be scaled up by the ratio of the 85th percentile, 24-hour storm depths. Alternately, if a VRA of the same size was provided at both locations, higher capture efficiency would be expected at Point A than Point B because Point A has a smaller 85th percentile, 24-hour storm event. The reliability of this method is discussed in Section 7.

This relationship provides a method for estimating the performance at a given point in a division based on continuous simulation model conducted elsewhere within the division (i.e., at the main gage selected for each division). The scaling and localization method was implemented as follows; calculations are conducted within the tool (i.e., not by the user).

1. From the lookup database supporting the tool, the 85th percentile, 24-hour depth for each of the 344 gages are provided to the user. The 85th percentile, 24-hour depth was calculated by tabulating all daily precipitation totals (hourly or daily precipitation records can be used), filtering the list to exclude daily totals less than 0.11 inches, and ranking the list to identify the 85th percentile value (USEPA 2009, EISA Technical Guidance).
2. The user has the option to either use the default storm depth for that climate division (implicitly this means that the project is represented by the rainfall of the primary gage for that division) or enter the 85th percentile, 24-hour storm depth for their specific project location. This can be relatively easily calculated by the user from local gage data (hourly or daily) or looked up from a local reference.
3. The lookup database of pre-packaged simulation results was indexed both by the absolute storage volumes associated with each simulation (precipitation depths, inches) as well as the storage volume as a fraction of the 85th percentile, 24-hour storm depth at the gage location that was modeled (unitless fraction).
4. The storage volume provided in the VRAs at the project is divided by the user-entered 85th percentile, 24-hour storm depth for their project location (if provided) to compute the normalized storage volume (fraction of the 85th percentile storm depth, unitless) for the model scenario. The normalized storage volume is then used to query the lookup database to estimate capture efficiency.

For example:

1. The 85^{th} percentile, 24-hour depth at the climate division gage that is used to represent the overall division in the modeling is 0.95 inches.
2. The 85^{th} percentile, 24-hour depth at the actual project location is estimated to be 0.80 inches based on the user's analysis of precipitation records in the vicinity of the site.
3. The user enters the 0.80 inches into the field in the tool that allows for location-specific override.
4. A project proposes to construct a VRA that has a storage volume equivalent to the runoff from a 0.6 inch storm event.
5. Therefore, the proposed VRA has a normalized unitless storage volume of 0.75 (fraction of the 85^{th} percentile, 24-hour storm depth = 0.6 inches / 0.8 inches).
6. The results of the pre-packaged model simulations corresponding to a normalized unitless storage volume of 0.75 are used to estimate the capture efficiency of the project-specific VRA (interpolation is required if an exact match is not returned). In comparison, if "localization" was not implemented, the tool would look up the model result for a normalized unitless storage volume of 0.63 (0.6/0.95), which would tend to underestimate the actual performance at the project location.

4 Modeling Methodology and Parameters for Tool Development

4.1 Types of Modeling Conducted

Three sets of modeling runs were completed in the EPA Storm Water Management Model (SWMM) Version 5.0.022 to develop the underlying "lookup databases" to support the tool. This section describes the inputs to the SWMM model and the model increments that were used in these supporting model runs.

- **Consistent drawdown runs:** Consistent drawdown runs were used to represent VRA compartments that can be approximated as draining at a relatively consistent rate throughout a long term continuous simulation (e.g., infiltration, media filtration, orifice discharge). The template model setup developed for these runs included a tributary subcatchment draining to a storage unit of a given size (varied between runs) modeled with a drawdown rate (varied between runs) that was held constant throughout each simulation. Continuous rainfall-runoff processes were simulated to estimate the continuous runoff hydrograph. Routing through the storage unit was simulated to estimate the long term capture efficiency associated with the given configuration.
- **ET drawdown runs:** ET runs were used to represent VRA compartments that drain via ET processes, at rates that inherently vary with climatic factors throughout the year. The template model setup developed for these runs included a tributary subcatchment draining to a storage unit of a given size (varied between runs) modeled with a given stored water depth (varied between runs) that was drawn down at the applied ET rate (varies on a monthly basis and between locations). Continuous rainfall-runoff processes were simulated to estimate the continuous runoff hydrograph. Routing through the storage unit was simulated to estimate the long term ET loss associated with the given configuration.

- **Dispersion runs:** Dispersion runs were used to represent VRA types that cannot be simply divided into compartments because water is dispersed in a thin layer and is acted upon by both infiltration and ET processes. The template model setup developed for these runs included a tributary subcatchment draining to two broad, shallow storage units in series (area varied between runs to represent different proportions of pervious area receiving dispersion). The first storage unit was used to represent water stored in the “suction storage” of soil pores that did not freely drain via gravity. This was filled first and was drawn down at the rate established by ET inputs. This storage unit also received flow from a “dummy catchment” with 100 percent imperviousness and zero depression storage; effectively representing precipitation directly on the dispersion area. The second storage unit had the same footprint as the first storage unit (i.e., equal to the size of the dispersion area) and received flow when the first storage unit overflowed. These storage units were effectively “stacked” in the model. This storage unit represented the freely drained pore storage (i.e., drained by gravity) in the amended media and any surface ponding in closed depressions. This storage unit was drained via Green-Ampt infiltration processes based on the assigned infiltration parameters (varied between runs). The depth of stored water in the first and second storage compartments was calculated based on the assumed depth of soil amendments (varied between runs) and typical amended soil properties. Continuous rainfall-runoff processes were simulated to estimate the runoff hydrograph. Routing through the storage units was simulated to estimate the long term capture efficiency associated with the given configuration.

4.2 SWMM Model Inputs

A consistent set of SWMM inputs were used for each of the model runs described in Section 4.1.

4.2.1 Precipitation Data

Hourly precipitation datasets from the National Climatic Data Center (NCDC) were obtained for all available gages in the conterminous United States (approximately 6,700 gages). These gages were analyzed to estimate the period of record of observations, the fraction of the record that was missing or qualified, and the resolution of precipitation depth measurements. For each climate division (defined by NOAA, see Figure 4), a precipitation gage was selected that had at least 30 years of precipitation data, less than 5 percent missing or qualified data, and precipitation depth resolution of 0.01 inches, where possible. Where no gage within the climate division met these criteria, the gage that most closely met these criteria was selected. Section 9 lists the precipitation gages that were selected. Precipitation datasets were used directly as inputs to the EPA SWMM model.

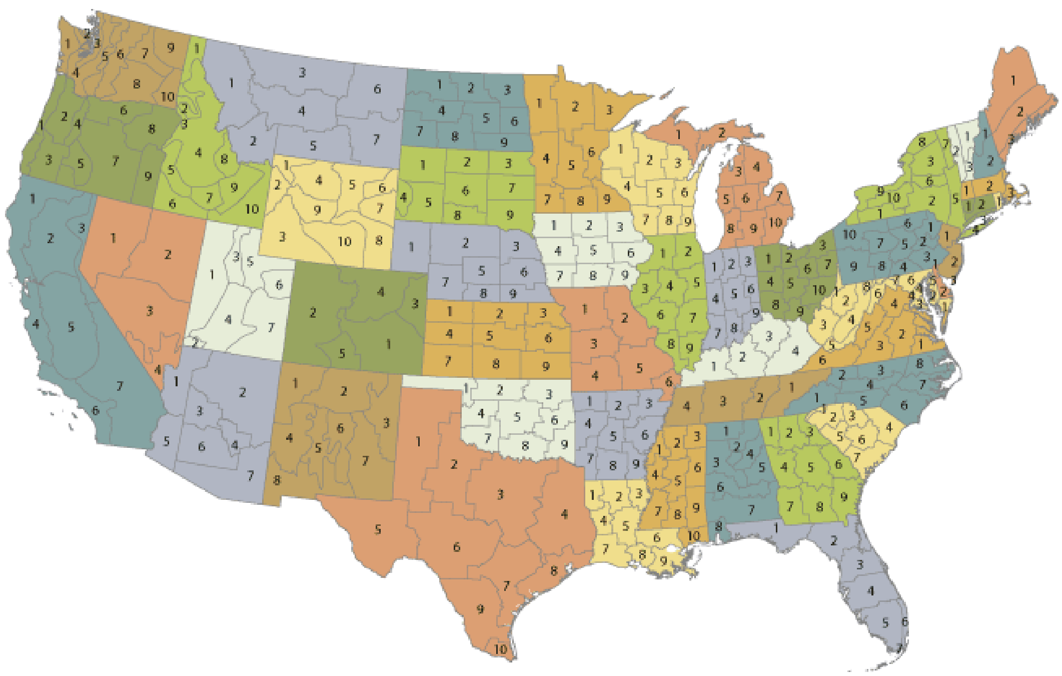

Figure 3. NCDC Climate Divisions

4.2.2 Evapotranspiration Data

Reference potential evapotranspiration data were obtained from the United States Geologic Service Oak Ridge National Laboratory (Vogel and Sankarasubramania 2005). The dataset has 40 years (1950 to 1990) of derived monthly ET estimates for 1,469 stations in the conterminous US. The closest available gage to each of the selected precipitation gages was selected from this dataset. Reference potential ET data were averaged for each month to estimate monthly normal potential ET valves. These monthly normal values were used in SWMM simulations, with an adjustment factor of 0.7 to account for actual potential ET being typically less than the reference potential ET for most cover types. For VRAs that are sensitive to ET rates, the user has the option to set the ET adjustment factor (also referred to as “crop coefficient”) to reflect the plant palette that is proposed.

4.2.3 SWMM Hydrologic Parameters

Table 1 contains the parameters that were used in the SWMM simulation of rainfall-runoff.

Table 1. SWMM Runoff Generation Parameters

SWMM Runoff Parameters	Units	Values	Source/Rationale
Wet time step	minutes	15	Standard assumption when using hourly precipitation inputs
Dry time step	hours	4	Standard assumption when using hourly precipitation inputs
Routing time step	seconds	120	Balance between stability and runtimes.
Precipitation Time Resolution	Hours	1.0	Hourly precipitation data available at each gage, mostly at 0.01 inch depth resolution.
Period of Record	years	1/1/1980 to 12/31/2009	NOAA National Climatic Data Center.
Evapotranspiration	in/ month	Varies by location	USGS Oak Ridge National Laboratory (Vogel and Sankarasubramanian, 2005)
Area	acres	1.0	Typical of smaller catchments in urban highway environment; not a sensitive parameter for volume reduction or capture performance of VRAs from 0.1 to more than 50 acres. This lack of sensitivity was demonstrated in model results from NCHRP 25-20(01).
Imperviousness	%	100	Simulations were conducted with a range of imperviousness to bracket the range that may be present in tributary areas. This was used to develop an adjustment method such that the results of 100% impervious catchments could be used to represent the performance of VRAs treating catchments less than 100% impervious (See Section 7).
Characteristic Flow Path Length	ft	86	86 ft path length; 500 ft width, per assumptions used in NCHRP 25-31. Not significantly sensitive for analysis of volume reduction processes.
Overland Slope	ft/ft	0.02	Represents typical cross slope on roadways; varies by site. Consistent with assumptions used in NCHRP 25-31. Not significantly sensitive for analysis of volume reduction processes with hourly precipitation inputs.
Depression storage, impervious	inches	0.05	ASCE 1992 (low range of estimate for impervious surface)
Depression storage, pervious	inches	0.1	ASCE 1992 (low range of estimates for lawns; characteristic of highway embankments, usually greater slope than lawns)
Impervious Manning's n		0.012	McCuen et al. 1996 – Based on concrete (typical of concrete and asphalt wearing coarse)

SWMM Runoff Parameters	Units	Values	Source/Rationale
Pervious Manning's n		0.15	McCuen et al. 1996 – Based on short grass
Infiltration Model	NA	Green-Ampt	Based on availability of parameters and supporting documentation.
Saturated Hydraulic Conductivity	in/hr	Varies by soil texture class, see Table 2 below.	
Suction Head	inches	Varies by soil texture class, see Table 2below.	
Initial Moisture Deficit	fraction	Varies by soil texture class, see Table 2 below.	

Table 2. SWMM Green-Ampt Infiltration Parameters

SWMM Runoff Parameters	Units	Soil Texture Class Groupings				Source/Rationale
		Sand – Sandy Loam	Sandy Loam – Sandy Clay Loam	Sandy Clay Loam – Silty Clay	Clay	
Assumed Texture Class-> (Rawls et al. 1983)		**Loamy Sand**	**Silt Loam**	**Sandy Clay Loam**	**Clay**	
Hydraulic Conductivity (mid-point and range)	in/hr	1.2 (0.43 – 4.7)	0.26 (0.06 to 0.43)	0.04 (0.02 to 0.06)	0.01 (0 to 0.02)	Rawls et al. 1983, based on assumed texture classes
Suction Head	inches	2.4	6.7	8.7	11.4	Rawls et al. 1983, based on assumed texture classes
Initial Moisture Deficit	fraction	0.36	0.29	0.21	0.16	Rawls et al. 1983, based on assumed texture classes (Porosity – Avg (Field Capacity, Wilting Point))

4.2.4 Storage Volume Increments

For each precipitation gage, storage volume increments were defined by 10 standard multiples of the 85th percentile, 24-hour storm depth for that gage (unitless) that bracket the range of typical VRA sizing:

- ⇒ 0.1
- ⇒ 0.2
- ⇒ 0.4
- ⇒ 0.6
- ⇒ 0.8
- ⇒ 1.0
- ⇒ 1.25
- ⇒ 1.5
- ⇒ 2.0
- ⇒ 3.0

For each model run, the 85th percentile, 24-hour storm depth was known at the modeled gage. For each configuration, the storage volume was calculated using the equation:

V (cu-ft) = 1.0 acre × 43,560 sq-ft/ac × Volumetric Runoff Coefficient × 85th pctl Storm Depth (inches) × Fraction of 85th pctl Storm Depth /12 inches/ft

The volumetric runoff coefficient used for the purpose of computing storage volume was assumed to be 0.9 for full impervious catchments.

4.2.5 Drawdown Increments

For compartments of VRAs that can be approximated by a consistent drawdown time (i.e., drained by infiltration or by a surface discharge outlet), model increments were defined by 10 standard drawdown times that are relevant for most VRAs (hours):

- ⇒ 2
- ⇒ 3
- ⇒ 6
- ⇒ 12
- ⇒ 24
- ⇒ 48
- ⇒ 72
- ⇒ 120
- ⇒ 180
- ⇒ 360

The tool is not intended for VRAs with very rapid drain times or very long drain times. Increments were developed based on our experience with the spacing of increments needed to obtain reliable results. Drawdown times were implemented in the model by setting the discharge rate from the SWMM storage unit (Q) to:

$$Q\ (cfs) = V\ (cu\text{-}ft) / (Drawdown\ Time\ (hours) \times 3600\ sec/hr)$$

4.2.6 ET Depth Increments

For compartments of VRAs that hold water and are drained primarily by ET, the "ET depth", defined as the amount of ET that must occur for the ET storage to be substantially recovered, were used as model increments. The following ET depth increments were modeled (inches):

- ⇒ 0.5
- ⇒ 0.75
- ⇒ 1
- ⇒ 2
- ⇒ 3
- ⇒ 5
- ⇒ 10

The low range was based on 3 inches of amended soil media with typical ET storage capacity (approximately the difference between the field capacity and the wilting point). The upper range was based on cistern that captures enough water to meet irrigation demand for 10 inches worth of ET over the area to be irrigated.

4.2.7 Dispersion Model Runs

Where VRAs operate primarily via dispersion of water over pervious areas (i.e., filter strips, vegetated swales) the concept of a "storage volume" and "drawdown time" can be difficult for the user to estimate from planning level design parameters. A separate lookup database was developed based on (1) the ratio of pervious area receiving flow to impervious area contributing flow, (2) the underlying soil infiltration rate of the receiving pervious area, and (3) the depth of amended soils. Increments of these parameters were simulated to develop the lookup database as shown in Table 3.

Table 3. Increments of Input Parameters for Dispersion Runs

Ratio of Impervious Area Draining to VRA to Pervious Area Receiving Dispersion	Soil Properties of Underlying Soils Below Amended Soil in Dispersion Area			Depth of Soil Amendment or Decompaction, inches
	Saturated Hydraulic Conductivity, in/hr	Initial Moisture Deficit, inches/ inch	Suction Head, inches	
50	0.00	0.16	11.4	0
20	0.01	0.16	11.4	3
10	0.05	0.21	8.7	6
5	0.10	0.21	8.7	12
2.5	0.20	0.29	6.7	24
1.0	0.50	0.29	6.7	
0.5	1.00	0.36	2.4	
0.25	2.00	0.36	2.4	

4.2.8 Summary of Supporting Model Runs

Table 4 provides a summary of the supporting model runs that were executed to provide the back-end database to support the tool. Sensitivity testing indicated that these increments provide reliable basis for interpolation.

Table 4. Summary of Supporting Model Runs

Parameter	Number of Increments
Consistent Drawdown Model Runs (Infiltration, Surface Discharge)	
Climate Divisions	344
Modeled Imperviousness of Tributary Area	1
Supported Imperviousness	0 to 100% (continuous scale; more reliable above 50%)
Modeled Soil Type	Not Applicable (100% impervious)
Supported Soil Type	User can select between 4 soil texture classes or enter a user-defined soil infiltration rate within the range supported.
Storage Volume	10
Drawdown Time	10
Total – Consistent Drawdown Runs	*34,400*
ET Drawdown Model Runs	
Climate Divisions	344
Modeled Imperviousness of Tributary Area	1
Supported Imperviousness	0 to 100% (continuous scale; more reliable above 50%)
Modeled Soil Type	Not Applicable (100% impervious)
Supported Soil Type	User can select between 4 soil texture classes or enter a user-defined soil infiltration rate within the range supported.
Storage Volume	10
ET Depth Increments	7
Total – ET Runs	*24,080*

Parameter	**Number of Increments**
Dispersion Model Runs	
Climate Divisions	344
Ratio of Impervious to Pervious	8
Soil Properties below Amended Soil	8
Amended Soil Depth	5
Total – Dispersion Model Runs	*110,080*
Total Number of Model Runs	***168,560***

Model runs were executed using macros that automate the development of SWMM files and execute these files in batch simulation model. Computationally it would have been possible to conduct more runs, however the size of the lookup database would increase and potentially create excessive file sizes for the Tool. Additionally, the opinion of the research team was that additional model runs would not substantially improve the reliability of estimates. However, if an individual DOT wanted to develop a customized version of the spreadsheet for their state, additional densities of meteorological stations could be run and/or additional more refined increments, etc.

Each SWMM run produced a summary report file (.rpt) that contained summary results of the simulation. Among many reported parameters, the rpt file included a summation of the volume that enters the storage element and overflows the storage element, which can be used to determine the baseline runoff (without controls) and the average annual percent capture for that scenario. A simple Excel Macro was customized to “mine” the relevant data from each of the rpt files and populate a database of summary results where each line of the database describes the model run and includes the summary output statistics.

5 Summary of Tool Calculations

The tool conducts a number of key calculations to translate the results of the SWMM model runs to estimates of long term volume reduction. This section provides a summary of calculations. Full details of calculations can be inspected via review of Excel formulas and macros.

- Based on user inputs about project location, the precipitation summary statistics are accessed from a precipitation data lookup table in the tool. The 85th percentile, 24-hour storm depth and the average annual storm depth are later used in tool calculations.
- User inputs about the tributary area to a VRA are used to estimate baseline average annual runoff volume. They are also later used to normalize the VRA design parameters relative to the catchment and precipitation parameters.
- VRAs are selected by the user from a drop down menu. This loads an input form that is unique to the selected VRA type. Each input form contains the design parameters, default parameters, and the associated calculations. Guidance is provided within the tool for specifying input parameters. Intermediate calculations are hidden from the user. Each VRA template has a unique set of rules about the compartments it contains, the order in which these compartments are analyzed, and any checks or limits that are enforced between compartments.
- Each VRA template has a unique set of calculations embedded within it to translate VRA design parameters into underlying lookup indices that can be used to obtain values from the lookup database. For example, these calculations perform functions such as:

  - Estimate the volume of each compartment based on the design parameters specified by the user and the default parameter assumptions.
  - Normalize the volume of each compartment in terms of the runoff volume produced from an equivalent storm depth over the tributary area. This calculation involves both the volumes of each compartment as well as the tributary area parameters. This is the step where the effects of VRA design, catchment size, imperviousness, and soil type are each considered: when a catchment is smaller, less impervious, and/or has soils with less runoff potential, a VRA compartment of a given volume is equivalent to the runoff from a larger storm event, thereby achieves higher capture efficiency, and vice versa.
  - Normalize the equivalent storm depth as a fraction of the local 85th percentile, 24-hour precipitation depth. This is the step where the effect of the local precipitation patterns are considered. When the local 85th percentile, 24-hour storm is larger, a VRA compartment of a given volume is smaller as a fraction of the runoff from this event, thereby achieves lower capture efficiency, and vice versa.
  - Calculate the drawdown parameters for each compartment (drawdown time, ET depth) based on specified parameters such as infiltration rate, depth of compartments, etc.
  - Calculate intermediate information about the specified VRA design, such as calculated VRA footprint, to provide feedback to the user and support iterative use of the Tool.

- After the lookup indices have been calculated (e.g., compartment volume as a the runoff volume from a certain fraction of the 85th percentile, 24-hour storm volume), two-dimensional interpolation algorithms are applied to obtain estimates from the lookup database for the exact combination of lookup indices (i.e., interpolation on two key indices within the lookup table). For compartments where some amount of volume reduction has already been provided in a previous compartment or VRA in a treatment train, these lookup/interpolation algorithms also account for “upstream” capture by implementing the algorithm described in Section 3.5.

- In some cases, two different design elements have the potential to control the performance. For example, in a bioretention area with underdrains, capture efficiency can be controlled by either the surface ponding volume and the rate at which water enters the amended media at the surface or by the total volume VRA volume and the rate at which water leaves the system as a whole. In cases such as this, both calculations are conducted to determine which computation controls.

- Validation checks are conducted to provide warnings when the specified set of design parameters and defaults results in lookup indices that are out of the bounds of the lookup database. Reference calculations are also conducted for key parameters to provide recommended ranges for design parameters so that the lookup indices remain within the supported range.

- Composite capture efficiency and volume reduction performance of the VRA or treatment train is tabulated, and these percentages are applied to the baseline runoff volume to calculate long term average quantities of volume reduced, volume treated, and volume bypassed (cu-ft per year).

- A sensitivity analysis interface allows the user to specify a high and low bound for up to three key parameters per VRA. An underlying sensitivity analysis algorithm is then executed to cycle through the tool calculations for the high and low bounds of each specified sensitivity parameter, holding the other specified parameters fixed at their original assigned value.

6 Simplifying Assumptions and Reliability

A number of simplifying assumptions were made as part of developing the tool. These are summarized below:

1. General rainfall patterns are assumed to be approximately homogenous within a climate division. While scaling to localize model estimates is recommended within climate divisions, as discussed above, and should significantly improve site-specific estimates, this approach assumes that the overall patterns of rainfall are relatively similar within a climate division such that the overall trends developed from modeling at a single gage location is representative throughout the climate division that the gage represents. The use of the 85th percentile, 24-hour storm depth as a basis for scaling has been demonstrated to be reliable in several areas (OCPW 2011; VCWPD 2011). Additional analysis was conducted as part of developing this tool (described in Section 7) to demonstrate that this approach is generally reliable across the contiguous US.
2. An approximate method was used to isolate and account for multiple VRA compartments instead of explicit modeling of all potential VRA configurations. The graphical method of summing multiple compartments of a VRA has been demonstrated to provide reliable results versus more explicit representation of VRAs (OCPW 2011; University of Missouri 2012). The validity of this approach has been evaluated against more explicit representations as part of this project (described in Section 7) and was found to provide reasonably reliable results.
3. Each VRA type was assumed to follow a prescribed drawdown pattern that does not change over the duration of the model or as a function of water depth/hydraulic head in the VRA. Time-variable factors such as temperature effects on infiltration rate and decline in infiltration rate as a result of clogging are not represented by the model. However, guidance is provided for selecting design infiltration rates that reflect the average long term conditions, accounting for seasonal variability in infiltration rates and inevitable declines in infiltration rates with time.
4. Snowfall and snowmelt were not simulated as part of developing lookup tables. A water equivalent approach was used where all precipitation was assumed to fall as liquid rain. While this assumption may introduce error in some climates, there are limited portions of the contiguous US that receive more than 10 to 20 percent of annual precipitation by snowfall. Failure to account for snowfall/snowmelt would potentially introduce considerable errors in peak flow estimation in some areas (i.e., rain on snow events); however, cumulative long term errors in runoff volume introduced for this portion of precipitation would generally have a relatively minor influence on long term volume reduction estimates. In some cases, rain on snow events could overwhelm a VRA in a way that would not be predicted by a simple water equivalent model. In other cases, snow pack could effectively serve as “detention” storage and would allow a greater amount of runoff to be captured (and then volume lost) than would be predicted by using the water equivalent assumptions. Therefore errors may approximately balance for purposes of estimating volume reduction.

5. Model scenarios were based on discrete increments of input parameters, and interpolation is required between entries in the lookup database. This approach assumes that response is approximately linear between discrete model scenarios. This assumption is generally reliable with appropriate selection of simulation increments to provide a greater number of increments where the response is most non-linear.
6. VRAs are assumed to follow an ordered filling and draining regime, where ET storage is assumed to fill first, followed by the lowest freely drained storage elements, followed by storage compartments located at a higher elevation. In essence, a VRA fills from the bottom to the top after satisfying ET storage. For the majority of storm events, this assumption is representative. However, under peak runoff conditions, it may be possible for upper storage elements to fill prior to lower storage elements because of limitations on the rate at which water is conveyed between elements. For example, in the case of a bioretention area with elevated underdrains, it may be possible for the surface ponding area to fill to a point of overflow before water can flow through the amended media to entirely fill the underdrain sump. Where this condition is possible, a secondary check is provided in the tool to determine if it controls the capture efficiency calculations.
7. Hourly precipitation records were used. For small catchments, these records are known to mask peak short term runoff rates, which occur as a response to sub-hourly, high intensity precipitation. This may be especially important for peak event overflows of VRAs. However, long term volume reduction performance is a function of processes that occur at a longer time interval than short-interval peak intensities and the cumulative total of many smaller events as well as less frequent larger events. Additionally, most VRAs include volumetric storage capacity that tends to equalize short-duration peak runoff rates. Therefore the use of hourly precipitation inputs is considered reliable for assessment of volume reduction performance of volume-based VRAs.
8. Estimates are derived from model runs of 100 percent impervious catchments. By developing a distinct runoff coefficient equation (runoff coefficient as a function of impervious cover) for each soil type, the normalized results of performance simulations conducted for 100 percent impervious catchments are translated to represent actual catchments that are less than 100 percent. For example, with correction for runoff coefficient in the application of the lookup database, the following example design scenarios (and many others) are represented by the same continuous simulation results:

    - A VRA sized at 3,000 cu-ft receiving runoff from 2 acre catchment at a runoff coefficient of 0.45 is equivalent to the runoff from a 0.91 inch storm event [(3,000 cu-ft × 12 in/ft) / (2 ac × 43,560 sq-ft/ac × 0.45) = 0.91 inches].
    - A VRA sized at 6,000 cu-ft receiving runoff from a 2 acre catchment with a runoff coefficient of 0.9 is equivalent to the runoff from a 0.91 inch storm event [(6,000 cu-ft × 12 in/ft) / (2 ac × 43,560 sq-ft/ac × 0.90) = 0.91 inches].

    This simplification is necessary to reduce the number of scenarios that must be simulated to develop the lookup database and maintain a reasonable file size for the tool. While minor non-linearity may be expected between capture efficiency and runoff coefficient, the selection of an appropriate runoff coefficient equation allows this simplification to be acceptable for planning level purposes. This simplification was evaluated (see Section 7) and found to be generally reliable.

Overall, these simplifying assumptions are expected to have a relatively minor impact on the model results and are considered to be reliable for planning level estimates of volume reduction performance. As with any modeling tool, a major source of error can arise from the user entering erroneous input parameters or entering input parameters that do not reflect realistic design scenarios.

7 Supplemental Case Study Analyses to Evaluate Reliability of Tool Methodology

Section 6 identified several simplifying assumptions for which further evaluation of reliability was deemed necessary. As part of early tool development and validation efforts, the research team performed focused analyses to test the reliability of these simplifying assumptions. For each simplifying assumption, example analyses were conducted explicitly (i.e., without simplification) and the results of these analyses were compared to what would be obtained using the simplification that was proposed. The simplifying assumptions that were evaluated include:

Simplifying Assumption 1: Normalized results from performance simulations conducted for 100 percent impervious catchments can be used to represent catchments with different imperviousness and soil types via post-processing methods. This allows for fewer model permutations to be run and reduces the file size of the tool.

Simplifying Assumption 2: The 85th percentile, 24-hour storm depth can be used as a normalizing and scaling factor to “localize” model results to account for intra-climate division variability as described in Section 3.6. This provides improved spatial resolution by allowing reliable scaling within the influence area of each precipitation gage that was modeled.

Simplifying Assumption 3: The graphical method presented in Section 3.5 can account for the non-linearity of capture response observed in multi-compartment VRAs and treatment trains and reliably approximate the performance of these types of systems. This allows the effects of standard single-compartment runs to be summed after model runs are complete, rather than running models for all potential configurations of multi-compartment or treatment train VRAs. This significantly reduces the number of modeling runs required and reduces the file size of the tool.

Simplifying Assumption 4: Results of evapotranspiration (ET) runs using monthly normal ET data are comparable to model estimates developed using a daily time series of ET data and therefore monthly data can be utilized.

To test the validity of these assumptions, three clusters of precipitation gages were selected to represent distinctly different climate zones across the country. Within each cluster, gages were selected to represent distinctly different local geographic settings that could lead to different VRA performance (i.e., elevation, topography, distance from major water bodies). Precipitation gages used in the analysis are shown in Figure 4.

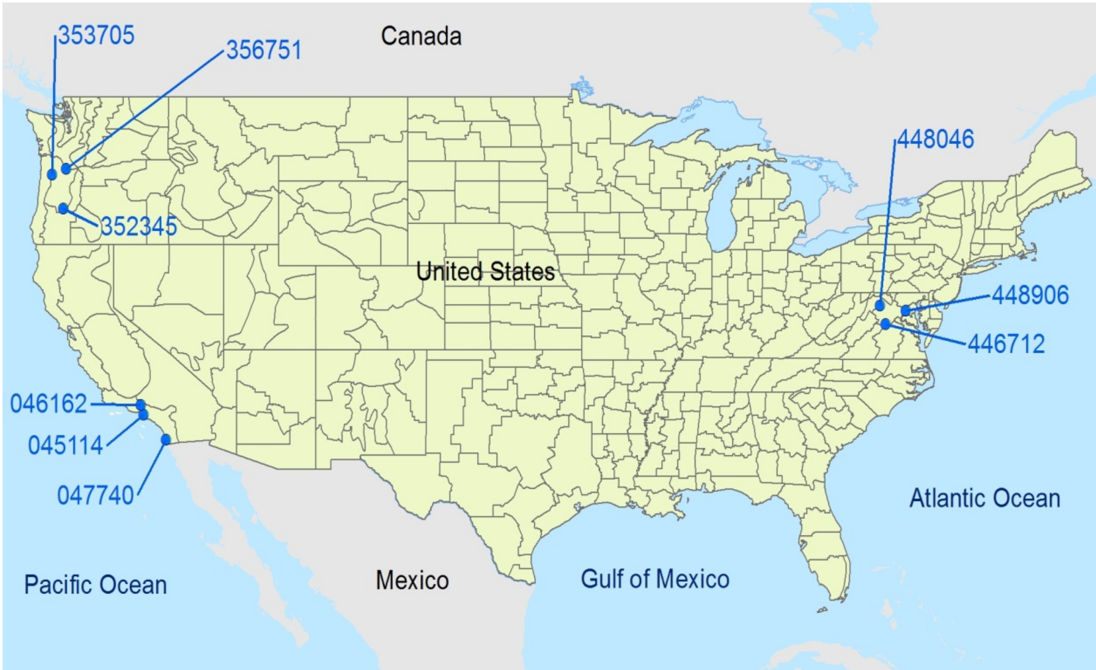

Figure 4. Precipitation gages used for testing the reliability of simplifying assumptions (with NCDC climate division boundaries shown)

7.1 Evaluation of Simplifying Assumption 1

7.1.1 Hypothesis

Our hypothesis was that the results of performance simulations conducted for 100 percent impervious catchments can be translated to represent actual catchments that are less than 100 percent by developing a distinct event-based runoff coefficient equation for each soil type. The runoff coefficient equation would be used in the “backend” of the tool to estimate the “equivalent precipitation depth” over the watershed that would produce a runoff volume equivalent to the VRA storage volume. In other words, we hypothesized that two catchments with varying characteristics (imperviousness, soil type, area) that have the same “equivalent precipitation depth” will achieve approximately the same capture efficiency. For example, with correction for runoff coefficient in the application of the lookup database, the following example design scenarios (and many others) could be represented by the same continuous simulation results:

- A VRA sized at 3,000 cu-ft receiving runoff from 2 acre catchment at a runoff coefficient of 0.45 is equivalent to the runoff from a 0.91 inch storm event [(3,000 cu-ft × 12 in/ft) / (2 ac × 43,560 sq-ft/ac × 0.45) = 0.91 inches].
- A VRA sized at 6,000 cu-ft receiving runoff from a 2 acre catchment with a runoff coefficient of 0.9 is equivalent to the runoff from a 0.91 inch storm event [(6,000 cu-ft × 12 in/ft) / (2 ac × 43,560 sq-ft/ac × 0.90) = 0.91 inches].

The form of the runoff coefficient equation proposed for the purpose of computing the “equivalent precipitation depth” is:

$$Runoff\ Coefficient = m \times imp + b$$

Where: imp = impervious cover as a fraction from 0 to 1; m and b are coefficients ranging from 0 to 1.

Different coefficients were developed for different soil types.

7.1.2 Analyses

In order to test this hypothesis, consistent drawdown runs were performed for combinations of (i) 4 soil types; (ii) 5 impervious fractions; (iii) 10 storage volume increments and (iv) 10 drawdown increments (2,000 runs for each gage) using the SWMM parameters listed Section 4 for the following three precipitation gages: (i) Washington D.C. Reagan Airport (COOP ID: 448906); (ii) Los Angeles International Airport (COOP ID: 45114) and (iii) Portland International Airport (COOP ID: 356751).

Model results allowed us to compare between (1) the percent capture achieved based on modeling of explicit combinations of imperviousness and soil type, and (2) the percent capture achieved for each distinct combination using the hypothesized approach based on 100 percent imperviousness runs normalized the using the runoff coefficient equation. Coefficients m and b were developed for each soil type by minimizing the sum of range of percent capture estimates for five "equivalent precipitation depths" (0.2", 0.5", 1", 1.5" and 2") using m and b as variables (solver function in Excel). Each combination of "equivalent precipitation depth" and VRA drawdown time had a range of percent capture calculated as the difference between maximum and minimum percent captures from 25%, 50%, 75% and 100% impervious catchment runs. In other words, where the range was zero, the approach worked perfectly to normalize for different soil types and imperviousness. Where the range was greater, the normalization routine did not account for some aspects of the response.

Coefficients estimated in this analysis for Washington D.C. gage are shown in Table 5. The reliability of the developed coefficients for a different precipitation gage was tested by using the coefficients developed using Washington D.C. gage in the runs conducted for Los Angeles and Portland gages. The average range of capture efficiency observed at each gage for each soil type (across combinations of 5 equivalent precipitation depths and 10 drawdown increments = 50 total data points) is presented in Table 5.

Table 5. Analysis Performed to Test Simplifying Assumption 1

Based on Washington D.C. Runs			Average (Max) Range of Differences in Capture Efficiency[1]		
Soil Type	**m**	**b**	**Washington D.C.**	**Los Angeles**	**Portland**
Loamy Sand	0.90	0.00	0.1% (0.7%)	0.1% (0.3%)	0.1% (0.2%)
Silt loam	0.86	0.04	1.9% (3.5%)	2.0% (4.5%)	2.3% (5.1%)
Sandy Clay Loam	0.37	0.53	3.6% (10%)	3.4% (6.3%)	5.5% (20%)
Clay	0.15	0.75	1.5% (6%)	1.2% (4.2%)	3.3% (11%)

1 – Range of differences in capture efficiency is the absolute difference between min and max percent capture (after normalization) for a given precipitation depth, drawdown time, and given soil type, based on a range of model runs from 25 to 100 percent imperviousness. For example, if the 25 percent impervious run yielded 45 percent capture after normalization and the 100 percent impervious run yielded 48 percent capture after normalization, then the reported range for that combination of inputs would be 3 percent.

Note: Runoff coefficient equation m and b are not intended to be used to yield estimates of long term runoff coefficient; they have been developed specifically to provide a "best fit" for this application.

7.1.3 Summary of Findings

Based the summary presented in Table 5 and evaluation of individual runs (not reported), average ranges of capture efficiency differences observed in the analysis for Washington D.C. are relatively small and are generally within the anticipated uncertainty associated with selection of model input parameters. When runoff coefficient formulae are transferred to other regions, maximum differences tended to increase slightly. In all three regions, results of model runs with lower imperviousness (25 percent) tended to control the ranges (i.e., tended to be most different from the mean), while results from 50, 75, and 100 percent imperviousness runs tended to be more closely spaced. For low imperviousness, where errors appear to be greatest, the simplification resulted in under-prediction of capture which is considered to be conservative.

These errors are generally within the range of certainty in model inputs and highway catchments tend to have imperviousness well above 50 percent. Overall the post-processing approach for addressing differences in imperviousness performs relatively well and tends to result in a minor underestimate of capture for lower imperviousness watersheds. This validates the usage of this simplifying assumption.

7.2 Evaluation of Assumption 2

7.2.1 Hypothesis

Our hypothesis was that the 85th percentile, 24-hour precipitation depth can be used to localize model results as described in Section 3.6.

7.2.2 Analysis

In order to test the “localization” approach presented in Section 3.6, a consistent drawdown run was modeled with 10 storage increments and 10 drawdown increments (100 runs) for 9 precipitation gages from three climate divisions (3 from each climate division; shown in Figure 4 and Table 6). A one acre 100 percent impervious catchment was used as the contributing watershed for these runs.

The results of model runs from each of the 3 rain gages within each of the three clusters were first extracted, then were compared to the estimate that would have been obtained had only the first gage in each cluster been run and results had normalized to the other gages based on the relative ratios of the 85th percentile, 24-hour precipitation depths (as proposed in Section 3.6). The range of capture efficiencies indicates how much error is introduced via the localization approach versus explicit runs of each gage. The average range of capture efficiencies estimated from the analysis for the three gages in each climate division from the 100 runs is presented in Table 6.

Table 6. Analysis Performed to Test Simplifying Assumption 2

Climate Division	Coop Id	Station Name	85th Percentile (in)	Elevation (ft)	Average (Max) Range of Differences in Capture Efficiency[1]
4404	448906	WASHINGTON REAGAN AP	0.94	33	1.6% (4.2%)
	448046	STAR TANNERY	0.90	3117	
	446712	PIEDMONT RSCH STN	1.10	1706	
3502	356751	PORTLAND INTL AP	0.63	62	2.2% (10%)
	353705	HASKINS DAM	1.25	2480	
	352345	DISSTON 1 NE LAYING CK	0.82	3996	
0406	45114	LOS ANGELES INTL AP	1.02	318	2.6% (14%)
	47740	SAN DIEGO WSO AP	0.78	49	
	46162	NEWHALL S FC32CE	1.70	4045	

1 – Range of differences in capture efficiency is the difference between the percent capture from each gage-specific analysis and the percent capture estimated by "localizing" results from the first gage in each cluster for each combination of equivalent precipitation depth, drawdown time.

7.2.3 Summary of Findings

Based on the summary in Table 6 and evaluation of individual runs (not reported), the localization approach appears to provide a reliable basis on average for scaling within each of the climate divisions. We do not see reason to believe that a significantly better localization, on average, would be achieved using a different percentile rainfall depth. Maximum differences occur where drawdown times are very short (2 to 3 hours) and storage volumes are very small (0.1 to 0.3 inch equivalent storm depths).

7.3 Evaluation of Simplifying Assumption 3

7.3.1 Hypothesis

Our hypothesis was that the graphical approach for summing multiple compartments and treatment trains presented in Section 3.5 is reliable for applications of the tool to quantify percent capture and percent volume reduction provided by typical VRA configurations that have multiple compartments.

7.3.2 Analysis

In order to test the approach presented in Section 3.5, the following two analyses were performed and then the results from each analysis were compared. Both "Analysis 1" and "Analysis 2" (described below) were performed for the following three precipitation gages: (i) Washington D.C. Reagan Airport (COOP ID: 448906); (ii) Los Angeles International Airport (COOP ID: 45114) and (iii) Portland International Airport (COOP ID: 356751). A one acre 100 percent impervious catchment was selected in the analyses as the contributing watershed.

Analysis 1 – Explicit Representation of Multi-Compartment VRAs: SWMM was used to estimate composite capture efficiency for a two compartment VRA. The model in this analysis was set up such that the first compartment receives runoff from the catchment and the second compartment receives overflow from the first compartment (or fills when the capacity of the first is exceeded). Capture efficiency in this analysis was estimated using the equation:

Capture efficiency = 1 - (Combined overflow leaving system)/(Runoff)

Two multi-compartment scenarios (A and B) were modeled as explicit configurations in SWMM. Increments used in the analysis are presented in Table 7.

Scenario A has a constant drawdown compartment followed by another consistent drawdown compartment. This could be representative of a permeable shoulder reservoir that overflows to a bioretention area, or could be representative of a bioretention area with elevated underdrains that has two distinct storage compartments that fill in a pre-determined order. Scenario B has an ET compartment followed by a consistent drawdown compartment. This could represent a filter strip that flows to an infiltration trench, or a bioretention area where water is first intercepted in soil pores before filling the gravity storage elements of the system. Each scenario has a total of 256 runs for each precipitation gage.

Table 7. Increments used in Explicit SWMM Runs

	Compartment 1			**Compartment 2**	
	Storage Increment	**Drawdown (hrs.)**		**Storage Increment**	**Drawdown (hrs.)**
Scenario A	0.2	12		0.2	12
	0.6	24		0.6	24
	1	48		1	48
	2	72		2	72
	Storage Increment	**ET Depth (in)**		**Storage Increment**	**Drawdown (hrs.)**
Scenario B	0.2	0.5		0.2	12
	0.6	1		0.6	24
	1	2		1	48
	2	5		2	72

Analysis 2 – Graphical Method: The same two scenarios were evaluated using the lookup database generated from runs with single storage compartments by applying the graphical method described in Section 3.5.

The results of the graphical method (Analysis 2) were compared to the results of the explicit representation (Analysis 1). The average absolute difference in capture efficiency between the two analyses for each precipitation gage is presented in Table 8.

Table 8. Analysis Performed to Test Simplifying Assumption 3

	Absolute Difference in Capture Efficiency – Average (Max) of 256 Parallel Model Runs	
	Scenario A	**Scenario B**
Washington D.C.	0.5% (2%)	0.7% (4.7%)
Los Angeles	0.5% (2.4%)	0.7% (3.4%)
Portland	0.8% (3.3%)	0.7% (4.1%)

7.3.3 Summary of Findings

The analysis performed to test the graphical method for two typical VRA scenarios supports this simplifying assumption and validates the usage of the graphical approach presented Section 3.5. The relative error produced using this approach appears to be much smaller than the uncertainty in model inputs (based on the summary in Table 8 and evaluation of individual runs, not reported).

Greater error may be introduced in some configurations; additionally this approach does not account for the details of all routing configurations that could be used. However, conducting explicit simulations of all potential combinations of storage compartments and routing configurations could yield model runs in the range of millions or billions and would be computationally prohibitive. Alternatively, simply adding the capture efficiency of multiple compartments (rather than using the graphical method) is theoretically flawed. Therefore, this represents a reliable approach for balancing rigor with computational constraints.

7.4 Evaluation of Simplifying Assumption 4

7.4.1 Hypothesis

Our hypothesis was that monthly normal ET data are adequate for model runs and provide a reasonable approximation of model results that would be obtained using a daily time series of data.

7.4.2 Analysis

In order to understand the sensitivity of temporal resolution of ET data used in the simulations on model results, parallel scenarios were evaluated. Each scenario consisted of a compartment that holds water and drains primarily by ET (storage equivalent to the runoff volume from the 24-hour, 85th percentile storm event with a 2-inch ponding depth that drains to ET only) and receives runoff from a 100 percent impervious 1 acre catchment. Scenarios were modeled using both monthly normal ET data and daily ET data for a 10 year record (10/1/1999 to 9/30/2009) for the following three precipitation gages: (i) Washington D.C. Reagan Airport (COOP ID: 448906); (ii) Los Angeles International Airport (COOP ID: 045114) and (iii) Portland International Airport (COOP ID: 356751). Results from this analysis are summarized in Table 9 below.

Table 9: Estimates of Capture Efficiency with Usage of Different Temporal Resolution ET Data

	Total Runoff (10^6 gallon)			Capture Efficiency		
	Daily ET data	Monthly Normal ET data	% Difference	Daily ET data	Monthly Normal ET data	Difference in Capture
Washington D.C.	9.73	9.38	3.6%	39%	40%	-1.0%
Los Angeles	2.47	2.41	2.3%	49%	51%	-2.2%
Portland	7.35	7.06	3.8%	20%	21%	-0.8%

7.4.3 Summary of Findings

Simulations with monthly normal ET data generated slightly smaller runoff quantities and slightly greater capture percentages than simulations with daily ET data. However, considering the small differences in predicted estimates and the simplicity offered by the use of monthly normal ET data, monthly normal data was considered most appropriate for this analysis.

7.5 Summary

Based on the analyses performed, the simplifying assumptions that were used are expected to have a relatively minor impact on the model results and are considered to be reliable for planning level estimates of volume reduction performance.

The greatest errors are typically encountered in watersheds with lower imperviousness and for design configurations where drawdown times and equivalent storm event storage volumes are small (i.e., systems are more sensitive to short-duration intensities/volumes). In general, these conditions are likely to be relatively rare in the highway environment for typical VRAs designs.

More rigorous analytical tools are recommended in cases where a greater degree of control is needed over analyzing specific watershed or VRA details and/or where the simplifying assumptions of this Tool are not acceptable.

8 References

American Society of Civil Engineers (ASCE) (1992). Design and Construction of Urban Stormwater Management Systems, New York, NY

Los Angeles County Department of Public Works (LACDPW) (2012). Los Angeles County Water Resources, Precipitation Website. http://www.ladpw.org/wrd/precip/. Last accessed: 12/31/2012.

McCuen, R., P. Johnson, and R. Ragan (1996). Highway Hydrology, FHWA-SA-96-067, Federal Highway Administration, Washington, DC

National Climatic Data Center (NCDC) (2012). http://www.ncdc.noaa.gov/

Orange County Public Works (OCPW) (2011). Technical Guidance Document for the Preparation of Conceptual/Preliminary and/or Project Water Quality Management Plans (WQMPs). http://www.ocwatersheds.com/DocmgmtInternet/Download.aspx?id=638

Rawls, W., D. Brakensiek, and N. Miller (1983). ”Green-ampt Infiltration Parameters from Soils Data.” J. Hydraul. Eng., 109(1), 62–70.

University of Missouri (2012). Stormwater Quality Master Plan for the University of Missouri at Columbia. Prepared by Geosyntec Consultants.

US Environmental Protection Agency (USEPA) (2009). Technical Guidance on Implementing Section 438 of the Energy Independence and Security Act of 2008. http://www.epa.gov/owow/NPS/lid/section438/pdf/final_sec438_eisa.pdf

Ventura County Watershed Protection District (VCWPD) (2011). Ventura County Technical Guidance Manual for Stormwater Control Measures, July 13, 2011. www.vcstormwater.org/technicalguidancemanual.html

9 Precipitation Gages Supported by Tool

State	Climate Division Number	Climate Division Name	Selected COOP Station	COOP Station Name	County	Elevation, MSL	Calculated 85th Percentile, 24-hour Storm Depth	Calculated 95th Percentile, 24-hour Storm Depth	Calculated Average Annual Precipitation Depth, inches
AL	1	NORTHERN VALLEY	014064	HUNTSVILLE INTL AP	MADISON	624	1.24	2.00	54.9
AL	2	APPALACHIAN MOUNTAIN	010831	BIRMINGHAM AP ASOS	JEFFERSON	615	1.20	1.93	45.1
AL	3	UPPER PLAINS	010063	ADDISON	WINSTON	818	1.30	2.10	51.2
AL	4	EASTERN VALLEY	014209	JACKSONVILLE	CALHOUN	688	1.20	1.92	45.6
AL	5	PIEDMONT PLATEAU	012124	DADEVILLE 2	TALLAPOOSA	733	1.30	2.00	51.7
AL	6	PRAIRIE	015550	MONTGOMERY AP ASOS	MONTGOMERY	202	1.25	2.01	51.0
AL	7	COASTAL PLAIN	010140	ALBERTA	WILCOX	175	1.40	2.20	55.2
AL	8	GULF	015478	MOBILE REGIONAL AP	MOBILE	215	1.45	2.42	58.6
AR	1	NORTHWEST	032356	EUREKA SPRINGS 3 WNW	CARROLL	1,420	1.20	1.91	43.2
AR	2	NORTH CENTRAL	032794	GILBERT	SEARCY	620	1.18	1.95	42.9
AR	3	NORTHEAST	030458	BATESVILLE LVSTK	INDEPENDENCE	571	1.30	2.10	45.9
AR	4	WEST CENTRAL	032574	FT SMITH RGNL AP	SEBASTIAN	449	1.20	1.92	42.3
AR	5	CENTRAL	034248	LITTLE ROCK ADAMS FLD	PULASKI	258	1.28	2.10	29.3
AR	6	EAST CENTRAL	036920	STUTTGART 9 ESE	ARKANSAS	198	1.30	2.20	47.8
AR	7	SOUTHWEST	032810	GILLHAM DAM	POLK	520	1.50	2.40	54.3
AR	8	SOUTH CENTRAL	030220	ARKADELPHIA 2 N	CLARK	196	1.44	2.30	52.5
AR	9	SOUTHEAST	035754	PINE BLUFF	JEFFERSON	215	1.40	2.20	49.3
AZ	1	NORTHWEST	024645	KINGMAN #2	MOHAVE	3,539	0.80	1.20	9.0
AZ	2	NORTHEAST	029439	WINSLOW AP	NAVAJO	4,886	0.48	0.75	7.3
AZ	3	NORTH CENTRAL	020487	ASH FORK 3	YAVAPAI	5,075	0.69	1.10	12.3
AZ	4	EAST CENTRAL	026323	PAYSON	GILA	4,850	0.90	1.50	19.3

State	Climate Division Number	Climate Division Name	Selected COOP Station	COOP Station Name	County	Elevation, MSL	Calculated 85th Percentile, 24-hour Storm Depth	Calculated 95th Percentile, 24-hour Storm Depth	Calculated Average Annual Precipitation Depth, inches
AZ	5	SOUTHWEST	029660	YUMA WSO AP	YUMA	206	0.58	1.11	2.9
AZ	6	SOUTH CENTRAL	026481	PHOENIX AP	MARICOPA	1,107	0.67	1.02	7.2
AZ	7	SOUTHEAST	028820	TUCSON INTL AP	PIMA	2,549	0.66	1.10	11.3
CA	1	NORTH COAST DRAINAGE	042910	EUREKA WFO WOODLEY IS	HUMBOLDT	20	0.88	1.37	39.2
CA	2	SACRAMENTO DRNG.	040161	ALTURAS	MODOC	4,400	0.50	0.72	12.3
CA	3	NORTHEAST INTER. BASINS	048873	TERMO 1 E	LASSEN	5,300	0.49	0.80	10.0
CA	4	CENTRAL COAST DRNG.	047769	SAN FRANCISCO WSO AP	SAN MATEO	8	0.89	1.39	20.2
CA	5	SAN JOAQUIN DRNG.	043257	FRESNO YOSEMITE INT'L	FRESNO	333	0.68	0.97	10.9
CA	6	SOUTH COAST DRNG.	045114	LOS ANGELES INTL AP	LOS ANGELES	97	1.02	1.60	12.3
CA	6	SOUTH COAST DRNG.	047740	SAN DIEGO WSO AP	SAN DIEGO	15	0.78	1.25	9.8
CA	7	SOUTHEAST DESERT BASIN	044232	INDEPENDENCE	INYO	3,950	0.70	1.30	4.8
CO	1	ARKANSAS DRAINAGE BASIN	053477	GRANADA	PROWERS	3,484	0.80	1.40	14.6
CO	2	COLORADO DRAINAGE BASIN	053488	GRAND JUNCTION WALKER	MESA	4,858	0.42	0.61	8.5
CO	3	KANSAS DRAINAGE BASIN	050304	ARAPAHOE	CHEYENNE	4,020	0.80	1.30	15.5
CO	4	PLATTE DRAINAGE BASIN	051179	BYERS 5 ENE	ADAMS	5,100	0.70	1.20	14.5
CO	5	RIO GRANDE DRAINAGE BASIN	057337	SAGUACHE	SAGUACHE	7,701	0.44	0.64	8.1
CT	1	NORTHWEST	065445	NORFOLK 2 SW	LITCHFIELD	1,340	1.04	1.69	50.8
CT	2	CENTRAL	063456	HARTFORD	HARTFORD	190	1.01	1.60	44.9
CT	3	COASTAL	060806	BRIDGEPORT SIKORSKY AP	FAIRFIELD	5	0.96	1.60	41.7
DE	1	NORTHERN	079595	WILMINGTON NEW CASTLE	NEW CASTLE	79	0.99	1.62	37.7
DE	2	SOUTHERN	073570	GEORGETOWN 5 SW	SUSSEX	45	1.10	1.70	40.0
FL	1	NORTHWEST	080211	APALACHICOLA AP	FRANKLIN	20	1.40	2.39	54.9
FL	2	NORTH	080975	BRANFORD	SUWANNEE	30	1.29	2.20	25.8
FL	3	NORTH CENTRAL	082158	DAYTONA BEACH INTL AP	VOLUSIA	31	1.20	2.03	49.1
FL	4	SOUTH CENTRAL	085612	MELBOURNE WFO	BREVARD	35	1.30	2.10	48.1
FL	5	EVERGLADES	083186	FT MYERS PAGE FLD AP	LEE	15	1.40	2.30	54.9

State	Climate Division Number	Climate Division Name	Selected COOP Station	COOP Station Name	County	Elevation, MSL	Calculated 85th Percentile, 24-hour Storm Depth	Calculated 95th Percentile, 24-hour Storm Depth	Calculated Average Annual Precipitation Depth, inches
FL	6	LOWER EAST COAST	085663	MIAMI INTL AP	MIAMI-DADE	29	1.28	2.20	58.9
GA	1	NORTHWEST	092485	DALLAS 7 NE	PAULDING	1,100	1.20	1.98	49.3
GA	2	NORTH CENTRAL	090451	ATLANTA/MUN., GA.	FULTON	1,010	1.14	1.80	49.0
GA	3	NORTHEAST	091619	CARNESVILLE 4 N	FRANKLIN	866	1.28	1.95	46.6
GA	4	WEST CENTRAL	092166	COLUMBUS METO AP	MUSCOGEE	392	1.18	1.90	48.8
GA	5	CENTRAL	095443	MACON MIDDLE GA AP	BIBB	343	1.13	1.86	44.8
GA	6	EAST CENTRAL	090495	AUGUSTA BUSH FLD AP	RICHMOND	132	1.10	1.77	43.4
GA	7	SOUTHWEST	093028	EDISON	CALHOUN	294	1.30	2.15	48.3
GA	8	SOUTH CENTRAL	096879	PEARSON	ATKINSON	205	1.29	2.00	46.5
GA	9	SOUTHEAST	097847	SAVANNAH INTL AP	CHATHAM	46	1.18	1.95	48.8
IA	1	NORTHWEST	136975	REMSEN	PLYMOUTH	1,330	0.97	1.50	28.1
IA	2	NORTH CENTRAL	137602	SHELL ROCK	BUTLER	912	0.92	1.50	30.5
IA	3	NORTHEAST	138009	STRAWBERRY POINT	CLAYTON	1,200	0.93	1.52	34.4
IA	4	WEST CENTRAL	137708	SIOUX CITY AP	WOODBURY	1,095	0.87	1.35	25.9
IA	5	CENTRAL	132203	DES MOINES AP	POLK	957	0.91	1.50	32.3
IA	6	EAST CENTRAL	130608	BELLEVUE L&D 12	JACKSON	603	0.98	1.54	33.1
IA	7	SOUTHWEST	131245	CARSON 3NNE	POTTAWATTAMIE	1,090	1.06	1.86	32.7
IA	8	SOUTH CENTRAL	132195	DERBY	LUCAS	1,190	1.00	1.63	34.2
IA	9	SOUTHEAST	138688	WASHINGTON	WASHINGTON	690	1.00	1.60	33.9
ID	1	PANHANDLE	101079	BONNERS FERRY	BOUNDARY	2,075	0.59	0.90	23.3
ID	3	NORTH CENTRAL PRAIRIES	105241	LEWISTON AP	NEZ PERCE	1,436	0.41	0.62	12.1
ID	3	NORTH CENTRAL CANYONS	103143	FENN RS	IDAHO	1,560	0.68	1.00	36.4
ID	4	CENTRAL MOUNTAINS	107327	PRAIRIE	ELMORE	4,780	0.68	1.05	22.2
ID	5	SOUTHWESTERN VALLEYS	101022	BOISE AIR TERMINAL	ADA	2,814	0.42	0.61	11.5
ID	6	SOUTHWESTERN HIGHLANDS	103811	GRASMERE 3 S	OWYHEE	5,140	0.50	0.80	10.9
ID	7	CENTRAL PLAINS	103677	GOODING 1 S	GOODING	3,643	0.50	0.80	10.3

State	Climate Division Number	Climate Division Name	Selected COOP Station	COOP Station Name	County	Elevation, MSL	Calculated 85th Percentile, 24-hour Storm Depth	Calculated 95th Percentile, 24-hour Storm Depth	Calculated Average Annual Precipitation Depth, inches
ID	8	NORTHEASTERN VALLEYS	105169	LEADORE	LEMHI	6,000	0.40	0.60	8.5
ID	9	UPPER SNAKE RIVER PLAINS	107211	POCATELLO RGNL AP	POWER	4,478	0.42	0.63	11.2
ID	10	EASTERN HIGHLANDS	104456	IDAHO FALLS 16 SE	BONNEVILLE	5,828	0.50	0.70	16.4
IL	1	NORTHWEST	115751	MOLINE WSO AP	ROCK ISLAND	592	0.96	1.56	36.9
IL	2	NORTHEAST	111577	CHICAGO MIDWAY AP 3SW	COOK	620	0.90	1.50	36.4
IL	3	WEST	110082	ALEXIS 1 SW	WARREN	680	0.97	1.60	33.2
IL	4	CENTRAL	116711	PEORIA GTR PEORIA RGNL	PEORIA	650	0.93	1.47	35.3
IL	5	EAST	114198	HOOPESTON 1NE	VERMILION	710	0.90	1.50	36.1
IL	6	WEST SOUTHWEST	118179	SPRINGFIELD LINCOLN AP	SANGAMON	594	0.90	1.47	34.9
IL	7	EAST SOUTHEAST	116159	NEWTON 6 SSE	JASPER	510	1.00	1.60	39.7
IL	8	SOUTHWEST	115983	MURPHYSBORO 2 SW	JACKSON	550	1.10	1.74	43.9
IL	9	SOUTHEAST	112353	DIXON SPRINGS AG CTR	POPE	527	1.15	1.82	45.8
IN	1	NORTHWEST	125535	MEDARYVILLE 5 N	PULASKI	695	0.90	1.50	36.2
IN	2	NORTH CENTRAL	128187	SOUTH BEND AP	ST. JOSEPH	773	0.80	1.33	37.7
IN	3	NORTHEAST	123037	FORT WAYNE AP	ALLEN	791	0.82	1.30	36.5
IN	4	WEST CENTRAL	120922	BRAZIL	CLAY	680	1.00	1.61	41.7
IN	5	CENTRAL	124259	INDIANAPOLIS INTL AP	MARION	790	0.91	1.47	40.4
IN	6	EAST CENTRAL	120132	ALPINE 2 NE	FAYETTE	850	0.92	1.46	41.1
IN	7	SOUTHWEST	122738	EVANSVILLE REGIONAL AP	VANDERBURGH	400	1.03	1.70	44.0
IN	8	SOUTH CENTRAL	126580	OOLITIC PURDUE EX FRM	LAWRENCE	650	1.05	1.60	43.6
IN	9	SOUTHEAST	120482	BATESVILLE WTR WKS	RIPLEY	970	1.00	1.50	43.4
KS	1	NORTHWEST	143153	GOODLAND RENNER FLD	SHERMAN	3,656	0.74	1.27	17.5
KS	2	NORTH CENTRAL	141767	CONCORDIA BLOSSER MUNI	CLOUD	1,469	0.97	1.60	27.5
KS	3	NORTHEAST	143810	HORTON	BROWN	1,030	1.10	1.90	35.4
KS	4	WEST CENTRAL	141730	COLLYER 10 S	TREGO	2,407	0.90	1.40	19.8
KS	5	CENTRAL	144178	KANOPOLIS LAKE	ELLSWORTH	1,492	1.07	1.70	25.5

State	Climate Division Number	Climate Division Name	Selected COOP Station	COOP Station Name	County	Elevation, MSL	Calculated 85th Percentile, 24-hour Storm Depth	Calculated 95th Percentile, 24-hour Storm Depth	Calculated Average Annual Precipitation Depth, inches
KS	6	EAST CENTRAL	148167	TOPEKA BILLARD MUNI AP	SHAWNEE	881	1.07	1.78	34.7
KS	7	SOUTHWEST	142164	DODGE CITY RGNL AP	FORD	2,582	0.86	1.44	20.5
KS	8	SOUTH CENTRAL	148830	WICHITA MID-CONTINENT	SEDGWICK	1,321	1.10	1.76	30.2
KS	9	SOUTHEAST	141351	CASSODAY 2SW	BUTLER	1,440	1.15	1.80	32.2
KY	1	WESTERN	151631	CLINTON 4 S	HICKMAN	350	1.20	1.90	46.5
KY	2	CENTRAL	154954	LOUISVILLE INTL AP	JEFFERSON	488	0.95	1.51	43.9
KY	3	BLUE GRASS	154746	LEXINGTON BLUEGRASS AP	FAYETTE	980	0.94	1.50	45.5
KY	4	EASTERN	151080	BUCKHORN LAKE	PERRY	780	0.90	1.44	45.8
LA	1	NORTHWEST	168440	SHREVEPORT AP	CADDO	254	1.32	2.16	42.5
LA	2	NORTH CENTRAL	169803	WINNFIELD 3 N	WINN	160	1.40	2.30	56.0
LA	3	NORTHEAST	169806	WINNSBORO 5 SSE	FRANKLIN	80	1.46	2.30	54.0
LA	4	WEST CENTRAL	165266	LEESVILLE	VERNON	28	1.40	2.40	52.8
LA	5	CENTRAL	169357	VIDALIA #2	CONCORDIA	60	1.50	2.55	54.5
LA	6	EAST CENTRAL	160549	BATON ROUGE METRO AP	EAST BATON ROUGE	64	1.36	2.30	57.7
LA	7	SOUTHWEST	165078	LAKE CHARLES AP	CALCASIEU	9	1.47	2.55	55.6
LA	8	SOUTH CENTRAL	165021	LAFAYETTE	LAFAYETTE	25	1.50	2.70	60.3
LA	9	SOUTHEAST	166660	NEW ORLEANS AP	JEFFERSON	4	1.44	2.40	61.3
MA	1	WESTERN	193985	KNIGHTVILLE DAM	HAMPSHIRE	630	1.00	1.68	44.5
MA	2	CENTRAL	190736	BLUE HILL	NORFOLK	625	1.08	1.75	50.2
MA	3	COASTAL	190770	BOSTON	SUFFOLK	12	0.96	1.57	43.1
MD	1	SOUTHEASTERN SHORE	188005	SALISBURY FAA AP	WICOMICO	48	0.92	1.86	44.1
MD	2	CENTRAL EASTERN SHORE	183090	FEDERALSBURG	CAROLINE	20	1.10	1.90	43.8
MD	3	LOWER SOUTHERN	186915	PATUXENT RIVER	ST. MARY'S	38	1.10	2.00	14.3
MD	4	UPPER SOUTHERN	180465	BALTIMORE WASH INTL AP	ANNE ARUNDEL	156	0.99	1.62	41.7
MD	6	NORTHERN CENTRAL	180470	BALTIMORE CITY	BALTIMORE (CITY)	14	1.05	1.72	41.2
MD	7	APPALACHIAN MOUNTAIN	184030	HANCOCK	WASHINGTON	384	0.90	1.30	34.4

State	Climate Division Number	Climate Division Name	Selected COOP Station	COOP Station Name	County	Elevation, MSL	Calculated 85th Percentile, 24-hour Storm Depth	Calculated 95th Percentile, 24-hour Storm Depth	Calculated Average Annual Precipitation Depth, inches
MD	8	ALLEGHENY PLATEAU	188315	SINES DEEP CREEK	GARRETT	2,040	0.74	1.16	46.9
ME	1	NORTHERN	171175	CARIBOU WFO	AROOSTOOK	624	0.70	1.06	37.0
ME	2	SOUTHERN INTERIOR	178641	SWANS FALLS	OXFORD	400	1.00	1.60	43.1
ME	3	COASTAL	176905	PORTLAND JETPORT	CUMBERLAND	45	0.97	1.62	43.7
MI	1	WEST UPPER	204090	IRON MT KINGSFORD WWTP	DICKINSON	1,071	0.79	1.20	28.3
MI	2	EAST UPPER	207366	SAULT STE MARIE SNDRSN	CHIPPEWA	722	0.62	1.02	32.6
MI	3	NORTHWEST	200662	BELLAIRE	ANTRIM	625	0.70	1.10	31.2
MI	4	NORTHEAST LOWER	200164	ALPENA CO RGNL AP	ALPENA	684	0.66	1.04	27.7
MI	5	WEST CENTRAL LOWER	205712	MUSKEGON CO AP	MUSKEGON	625	0.71	1.15	32.1
MI	6	CENTRAL LOWER	203170	GLADWIN	GLADWIN	775	0.80	1.40	18.3
MI	7	EAST CENTRAL LOWER	203580	HARBOR BEACH	HURON	600	0.70	1.10	29.5
MI	8	SOUTHWEST LOWER	203333	GERALD R FORD INTL AP	KENT	803	0.82	1.33	35.8
MI	9	SOUTH CENTRAL LOWER	204155	JACKSON 3N	JACKSON	950	0.80	1.26	30.6
MI	10	SOUTHEAST LOWER	202846	FLINT BISHOP INTL AP	GENESEE	770	0.71	1.16	26.2
MN	1	NORTHWEST	218235	THIEF LAKE REFUGE	MARSHALL	1,142	0.80	1.30	23.2
MN	2	NORTH CENTRAL	214026	INTERNATIONAL FALLS AP	KOOCHICHING	1,183	0.68	1.14	24.3
MN	3	NORTHEAST	212248	DULUTH INTL AP	ST. LOUIS	1,433	0.79	1.31	29.4
MN	4	WEST CENTRAL	210112	ALEXANDRIA CHANDLER FLD	DOUGLAS	1,416	0.83	1.31	11.1
MN	5	CENTRAL	217294	ST CLOUD MUNI AP	SHERBURNE	1,018	0.83	1.39	26.7
MN	6	EAST CENTRAL	215435	MINNEAPOLIS/ST PAUL AP	HENNEPIN	872	0.80	1.33	27.7
MN	7	SOUTHWEST	218323	TRACY	LYON	1,403	0.90	1.40	25.2
MN	8	SOUTH CENTRAL	215987	NORTHFIELD 2 NNE	DAKOTA	890	0.90	1.42	29.1
MN	9	SOUTHEAST	217004	ROCHESTER INTL AP	OLMSTED	1,304	0.85	1.44	29.5
MO	1	NORTHWEST PRAIRIE	234358	KANSAS CITY INTL AP	PLATTE	1,005	1.01	1.76	37.2
MO	2	NORTHEAST PRAIRIE	237455	ST LOUIS LAMBERT AP	ST. LOUIS	531	0.96	1.57	37.2
MO	3	WEST CENTRAL PLAINS	235987	NEVADA WTP	VERNON	820	1.20	1.90	39.3

State	Climate Division Number	Climate Division Name	Selected COOP Station	COOP Station Name	County	Elevation, MSL	Calculated 85th Percentile, 24-hour Storm Depth	Calculated 95th Percentile, 24-hour Storm Depth	Calculated Average Annual Precipitation Depth, inches
MO	4	WEST OZARKS	237976	SPRINGFIELD WSO AP	GREENE	1,259	1.10	1.78	42.0
MO	5	EAST OZARKS	238620	VIENNA 2 WNW	MARIES	770	1.11	1.74	41.3
MO	6	BOOTHEEL	233999	HORNERSVILLE	DUNKLIN	250	1.25	2.07	46.4
MS	1	UPPER DELTA	221743	CLEVELAND 3 N	BOLIVAR	140	1.40	2.30	50.4
MS	2	NORTH CENTRAL	227815	SARDIS DAM	PANOLA	303	1.32	2.16	53.9
MS	3	NORTHEAST	229003	TUPELO RGNL AP	LEE	361	1.30	2.13	54.4
MS	4	LOWER DELTA	228445	STONEVILLE EXP STN	WASHINGTON	127	1.40	2.10	50.0
MS	5	CENTRAL	225062	LEXINGTON	HOLMES	285	1.40	2.23	54.1
MS	6	EAST CENTRAL	228374	STATE UNIV	OKTIBBEHA	185	1.30	2.11	51.8
MS	7	SOUTHWEST	227714	RUTH 1 SE	LINCOLN	443	1.43	2.35	56.4
MS	8	SOUTH CENTRAL	227220	PURVIS 2N	LAMAR	378	1.40	2.30	59.8
MS	9	SOUTHEAST	225776	MERIDIAN AP	LAUDERDALE	294	1.30	2.12	55.2
MS	10	COASTAL	227840	SAUCIER EXP FOREST	HARRISON	229	1.50	2.50	66.9
MT	1	WESTERN	245745	MISSOULA INTL AP	MISSOULA	3,192	0.41	0.64	13.2
MT	2	SOUTHWESTERN	241309	BUTTE 8 S	SILVER BOW	5,700	0.50	0.80	14.8
MT	3	NORTH CENTRAL	241737	CHOTEAU	TETON	3,845	0.63	1.00	10.6
MT	4	CENTRAL	244055	HELENA AP ASOS	LEWIS AND CLARK	3,828	0.45	0.73	11.0
MT	5	SOUTH CENTRAL	240807	BILLINGS INTL AP	YELLOWSTONE	3,581	0.51	0.85	13.7
MT	6	NORTHEASTERN	241088	BREDETTE	ROOSEVELT	2,638	0.65	1.02	12.3
MT	7	SOUTHEASTERN	248169	TERRY 21 NNW	PRAIRIE	3,142	0.63	1.06	13.9
NC	1	SOUTHERN MOUNTAINS	310301	ASHEVILLE	BUNCOMBE	2,238	0.82	1.37	37.8
NC	2	NORTHERN MOUNTAINS	319675	YADKINVILLE 6 E	YADKIN	875	1.00	1.63	43.1
NC	3	NORTHERN PIEDMONT	313630	GREENSBORO AP	GUILFORD	890	0.98	1.56	42.1
NC	4	CENTRAL PIEDMONT	317069	RALEIGH AP	WAKE	416	0.99	1.57	42.2
NC	5	SOUTHERN PIEDMONT	311690	CHARLOTTE DOUGLAS AP	MECKLENBURG	728	1.01	1.67	42.4
NC	6	SOUTHERN COASTAL PLAIN	319457	WILMINGTON INTL AP	NEW HANOVER	33	1.24	2.19	54.7

State	Climate Division Number	Climate Division Name	Selected COOP Station	COOP Station Name	County	Elevation, MSL	Calculated 85th Percentile, 24-hour Storm Depth	Calculated 95th Percentile, 24-hour Storm Depth	Calculated Average Annual Precipitation Depth, inches
NC	7	CENTRAL COASTAL PLAIN	319476	WILSON 3 SW	WILSON	110	1.10	1.74	38.5
NC	8	NORTHERN COASTAL PLAIN	311458	CAPE HATTERAS AP	DARE	11	1.26	2.16	55.8
ND	1	NORTHWEST	329425	WILLISTON SLOULIN FLD	WILLIAMS	1,902	0.57	1.02	13.4
ND	2	NORTH CENTRAL	320492	BALFOUR 3 SW	MCHENRY	1,615	0.76	1.30	15.9
ND	3	NORTHEAST	321435	CAVALIER 7NW	PEMBINA	890	0.90	1.50	19.3
ND	4	WEST CENTRAL	327585	RIVERDALE	MCLEAN	1,977	0.80	1.35	16.6
ND	5	CENTRAL	322018	DAWSON	KIDDER	1,730	0.79	1.38	17.5
ND	6	EAST CENTRAL	322859	FARGO HECTOR INTL AP	CASS	900	0.74	1.30	20.0
ND	7	SOUTHWEST	327530	RICHARDTON ABBEY	STARK	2,470	0.70	1.21	17.0
ND	8	SOUTH CENTRAL	320819	BISMARCK MUNI AP	BURLEIGH	1,651	0.66	1.12	15.9
ND	9	SOUTHEAST	320382	ASHLEY	MCINTOSH	2,014	0.80	1.40	17.6
NE	1	PANHANDLE	257665	SCOTTSBLUFF AP	SCOTTS BLUFF	3,945	0.61	0.99	14.6
NE	2	NORTH CENTRAL	258760	VALENTINE MILLER AP	CHERRY	2,590	0.73	1.22	18.3
NE	3	NORTHEAST	255995	NORFOLK AP	MADISON	1,551	0.88	1.48	24.8
NE	5	CENTRAL	253395	GRAND ISLAND CTR NE AP	HALL	1,840	0.88	1.45	23.9
NE	6	EAST CENTRAL	254795	LINCOLN AP	LANCASTER	1,190	0.94	1.55	17.8
NE	7	SOUTHWEST	256065	NORTH PLATTE RGNL AP	LINCOLN	2,778	0.81	1.25	19.4
NE	8	SOUTH CENTRAL	252560	EDISON	FURNAS	2,120	0.90	1.41	21.2
NE	9	SOUTHEAST	258395	SYRACUSE	OTOE	1,100	1.00	1.70	30.0
NH	1	NORTHERN	275639	MT WASHINGTON	COOS	6,267	1.02	1.77	92.2
NH	2	SOUTHERN	271683	CONCORD ASOS	MERRIMACK	346	0.85	1.33	38.1
NJ	1	NORTHERN	286026	NEWARK INTL AP	ESSEX	7	0.99	1.61	44.0
NJ	2	SOUTHERN	280311	ATLANTIC CITY INTL AP	ATLANTIC	60	1.01	1.63	41.2
NJ	3	COASTAL	281351	CAPE MAY 2 NW	CAPE MAY	20	1.00	1.66	37.3
NM	1	NORTHWESTERN PLATEAU	293142	FARMINGTON AG SCI CTR	SAN JUAN	5,625	0.50	0.70	7.7
NM	2	NORTHERN MOUNTAINS	292837	EL VADO DAM	RIO ARRIBA	6,740	0.50	0.80	14.3

State	Climate Division Number	Climate Division Name	Selected COOP Station	COOP Station Name	County	Elevation, MSL	Calculated 85th Percentile, 24-hour Storm Depth	Calculated 95th Percentile, 24-hour Storm Depth	Calculated Average Annual Precipitation Depth, inches
NM	3	NORTHEASTERN PLAINS	292030	CONCHAS DAM	SAN MIGUEL	4,244	0.80	1.30	13.7
NM	4	SOUTHWESTERN MOUNTAINS	292250	CUBERO	CIBOLA	6,195	0.60	0.90	10.9
NM	5	CENTRAL VALLEY	290234	ALBUQUERQUE INTL AP	BERNALILLO	5,310	0.51	0.77	8.6
NM	6	CENTRAL HIGHLANDS	297094	PROGRESSO	TORRANCE	6,297	0.62	1.02	12.8
NM	7	SOUTHEASTERN PLAINS	290600	ARTESIA 6S	EDDY	3,366	0.78	1.30	10.7
NM	8	SOUTHERN DESERT	294426	JORNADA EXP RANGE	DONA ANA	4,266	0.65	1.07	9.2
NV	1	NORTHWESTERN	269171	WINNEMUCCA AP	HUMBOLDT	4,296	0.39	0.58	7.9
NV	2	NORTHEASTERN	262631	ELY AIRPORT	WHITE PINE	6,262	0.44	0.63	9.1
NV	3	SOUTH CENTRAL	268170	TONOPAH AIRPORT	NYE	5,395	0.42	0.63	2.3
NV	4	EXTREME SOUTHERN	264436	LAS VEGAS AP	CLARK	2,131	0.58	0.88	4.2
NY	1	WESTERN PLATEAU	303983	HORNELL ALMOND DAM	STEUBEN	1,325	0.70	1.20	32.2
NY	2	EASTERN PLATEAU	300687	BINGHAMTON GREATER AP	BROOME	1,595	0.74	1.16	37.1
NY	3	NORTHERN PLATEAU	303851	HIGHMARKET	LEWIS	1,763	0.90	1.50	52.5
NY	4	COASTAL	305811	NEW YORK LA GUARDIA AP	QUEENS	11	1.00	1.68	43.1
NY	5	HUDSON VALLEY	300042	ALBANY INTL AP	ALBANY	275	0.80	1.27	36.7
NY	6	MOHAWK VALLEY	308586	TRIBES HILL	MONTGOMERY	300	0.80	1.30	36.4
NY	7	CHAMPLAIN VALLEY	309389	WHITEHALL	WASHINGTON	119	0.87	1.35	36.1
NY	8	ST. LAWRENCE VALLEY	301185	CANTON 4 SE	ST. LAWRENCE	448	0.70	1.14	33.9
NY	9	GREAT LAKES	307167	ROCHESTER INTL AP	MONROE	533	0.65	1.03	32.0
NY	10	CENTRAL LAKES	308383	SYRACUSE HANCOCK AP	ONONDAGA	413	0.70	1.13	36.8
OH	1	NORTHWEST	338357	TOLEDO EXPRESS WSO AP	LUCAS	669	0.76	1.22	32.6
OH	2	NORTH CENTRAL	336196	OBERLIN	LORAIN	816	0.78	1.24	34.7
OH	3	NORTHEAST	330058	AKRON CANTON WSO AP	SUMMIT	1,208	0.72	1.21	36.3
OH	4	WEST CENTRAL	337935	SPRINGFIELD NEW WWKS	CLARK	930	0.90	1.40	38.6
OH	5	CENTRAL	331786	COLUMBUS WSO AP	FRANKLIN	810	0.79	1.32	38.0
OH	6	EAST CENTRAL	334865	MANSFIELD WSO AP	RICHLAND	1,295	0.80	1.30	34.2

State	Climate Division Number	Climate Division Name	Selected COOP Station	COOP Station Name	County	Elevation, MSL	Calculated 85th Percentile, 24-hour Storm Depth	Calculated 95th Percentile, 24-hour Storm Depth	Calculated Average Annual Precipitation Depth, inches
OH	7	NORTHEAST HILLS	334992	MASSILLON	STARK	930	0.90	1.40	11.2
OH	8	SOUTHWEST	332075	DAYTON WSO AP	MONTGOMERY	1,000	0.80	1.34	37.5
OH	9	SOUTH CENTRAL	334004	JACKSON 3 NW	JACKSON	800	0.90	1.45	36.7
OH	10	SOUTHEAST	338378	JENKINS DAM BURR OAK	ATHENS	760	0.80	1.30	35.1
OK	1	PANHANDLE	343002	EVA	TEXAS	3,574	0.80	1.30	15.1
OK	2	NORTH CENTRAL	343304	FORT SUPPLY 3SE	WOODWARD	2,030	1.00	1.71	21.4
OK	3	NORTHEAST	348992	TULSA INTL AP	TULSA	650	1.23	1.95	39.1
OK	4	WEST CENTRAL	345648	MAYFIELD	BECKHAM	2,005	1.10	1.90	23.9
OK	5	CENTRAL	346661	OKLAHOMA CITY WILL ROGERS AP	OKLAHOMA	1,285	1.17	1.87	33.2
OK	6	EAST CENTRAL	348497	STIGLER 1 SE	HASKELL	570	1.35	2.30	43.1
OK	7	SOUTHWEST	343281	FT COBB	CADDO	1,285	1.20	2.01	28.8
OK	8	SOUTH CENTRAL	344865	KINGSTON 5 SSE	MARSHALL	684	1.34	2.18	38.0
OK	9	SOUTHEAST	340670	BENGAL 4 NNW	LATIMER	667	1.40	2.40	48.3
OR	1	COASTAL AREA	350328	ASTORIA AP PORT OF	CLATSOP	9	0.88	1.42	67.8
OR	2	WILLAMETTE VALLEY	356751	PORTLAND INTL AP	MULTNOMAH	19	0.63	0.98	36.7
OR	3	SOUTHWESTERN VALLEYS	355429	MEDFORD INTL AP	JACKSON	1,297	0.60	0.97	19.1
OR	4	NORTHERN CASCADES	352697	ESTACADA 24 SE	CLACKAMAS	2,200	0.88	1.40	54.0
OR	5	HIGH PLATEAU	353232	GERBER DAM	KLAMATH	4,850	0.60	1.20	17.7
OR	6	NORTH CENTRAL	356546	PENDLETON E OR RGNL AP	UMATILLA	1,486	0.41	0.62	12.1
OR	7	SOUTH CENTRAL	354670	LAKEVIEW 2 NNW	LAKE	4,890	0.50	0.79	15.4
OR	8	NORTHEAST	356845	PRAIRIE CITY RS	GRANT	3,540	0.47	0.70	14.6
OR	9	SOUTHEAST	354321	JORDAN VALLEY	MALHEUR	4,390	0.45	0.71	12.5
PA	1	POCONO MOUNTAINS	369705	WILKES-BARRE INTL AP	LUZERNE	930	0.78	1.29	37.0
PA	2	EAST CENTRAL MOUNTAINS	360106	ALLENTOWN AP	LEHIGH	390	0.99	1.57	44.1
PA	3	SOUTHEASTERN PIEDMONT	366889	PHILADELPHIA INTL AP	PHILADELPHIA	10	0.99	1.60	41.5
PA	4	LOWER SUSQUEHANNA	363699	HARRISBURG CAPITAL CY	YORK	340	0.84	1.47	32.5

State	Climate Division Number	Climate Division Name	Selected COOP Station	COOP Station Name	County	Elevation, MSL	Calculated 85th Percentile, 24-hour Storm Depth	Calculated 95th Percentile, 24-hour Storm Depth	Calculated Average Annual Precipitation Depth, inches
PA	5	MIDDLE SUSQUEHANNA	367931	SELINSGROVE 2 S	SNYDER	420	1.00	1.70	24.6
PA	6	UPPER SUSQUEHANNA	368905	TOWANDA 1 S	BRADFORD	760	0.80	1.21	32.7
PA	7	CENTRAL MOUNTAINS	362265	DU BOIS 7 E	CLEARFIELD	1,670	0.75	1.15	40.6
PA	8	SOUTH CENTRAL MOUNTAINS	364001	HOLLIDAYSBURG 2 NW	BLAIR	990	0.81	1.30	36.4
PA	9	SOUTHWEST PLATEAU	366993	PITTSBURGH ASOS	ALLEGHENY	1,203	0.71	1.13	34.5
PA	10	NORTHWEST PLATEAU	362682	ERIE WSO AP	ERIE	730	0.73	1.21	36.6
RI	1	ALL	376698	PROVIDENCE	KENT	60	1.05	1.75	45.6
SC	1	MOUNTAIN	384581	JOCASSEE 8 WNW	OCONEE	2,500	1.60	2.80	82.4
SC	2	NORTHWEST	383747	GRNVL SPART INTL AP	SPARTANBURG	943	1.12	1.86	48.8
SC	3	NORTH CENTRAL	389327	WINNSBORO	FAIRFIELD	560	1.20	1.90	35.6
SC	4	NORTHEAST	385306	LORIS 2 S	HORRY	90	1.20	1.90	46.8
SC	5	WEST CENTRAL	386209	NEWBERRY	NEWBERRY	476	1.20	1.90	44.9
SC	6	CENTRAL	381939	COLUMBIA METRO AP	LEXINGTON	225	1.19	1.92	45.1
SC	7	SOUTHERN	381544	CHARLESTON INTL AP	CHARLESTON	40	1.20	2.00	49.4
SD	1	NORTHWEST	394864	LEMMON	PERKINS	2,567	0.71	1.20	16.7
SD	2	NORTH CENTRAL	396282	ONAKA 2N	FAULK	1,610	0.80	1.40	18.1
SD	3	NORTHEAST	390020	ABERDEEN RGNL AP	BROWN	1,297	0.79	1.32	18.6
SD	4	BLACK HILLS	396427	PACTOLA DAM	PENNINGTON	4,720	0.70	1.20	18.5
SD	5	SOUTHWEST	392557	EDGEMONT	FALL RIVER	3,610	0.62	1.02	15.0
SD	6	CENTRAL	396170	OAHE DAM	STANLEY	1,660	0.80	1.30	15.0
SD	7	EAST CENTRAL	394127	HURON AP	BEADLE	1,280	0.78	1.28	20.0
SD	8	SOUTH CENTRAL	395620	MISSION	TODD	2,587	0.90	1.41	20.8
SD	9	SOUTHEAST	397667	SIOUX FALLS AP	MINNEHAHA	1,428	0.85	1.41	23.9
TN	1	EASTERN	401656	CHATTANOOGA AP	HAMILTON	671	1.12	1.79	53.1
TN	2	CUMBERLAND PLATEAU	406170	MONTEREY	PUTNAM	1,860	1.19	1.82	55.7
TN	3	MIDDLE	406402	NASHVILLE ASOS	DAVIDSON	600	1.05	1.75	47.8

State	Climate Division Number	Climate Division Name	Selected COOP Station	COOP Station Name	County	Elevation, MSL	Calculated 85th Percentile, 24-hour Storm Depth	Calculated 95th Percentile, 24-hour Storm Depth	Calculated Average Annual Precipitation Depth, inches
TN	4	WESTERN	405954	MEMPHIS INTL AP	SHELBY	254	1.30	2.02	51.6
TX	1	HIGH PLAINS	415411	LUBBOCK INTL AP	LUBBOCK	3,254	0.91	1.55	15.8
TX	2	LOW ROLLING PLAINS	419729	WICHITA FALLS MUNI AP	WICHITA	1,017	1.13	1.86	27.5
TX	3	NORTH CENTRAL	413284	FT WORTH MEACHAM FLD	TARRANT	687	1.20	2.03	28.6
TX	4	EAST TEXAS	419665	WHEELOCK	ROBERTSON	420	1.36	2.20	36.6
TX	5	TRANS PECOS	412797	EL PASO AP	EL PASO	3,918	0.60	1.04	8.5
TX	6	EDWARDS PLATEAU	418845	TARPLEY	BANDERA	1,390	1.30	2.22	31.3
TX	7	SOUTH CENTRAL	410428	AUSTIN CAMP MABRY	TRAVIS	670	1.21	2.01	32.3
TX	8	UPPER COAST	419364	VICTORIA ASOS	VICTORIA	115	1.32	2.30	30.5
TX	9	SOUTHERN	414191	HINDES	ATASCOSA	360	1.30	2.38	23.5
TX	10	LOWER VALLEY	411136	BROWNSVILLE INTL AP	CAMERON	24	1.17	2.28	25.7
UT	1	WESTERN	422090	DELTA	MILLARD	4,620	0.45	0.69	7.7
UT	2	DIXIE	427516	ST GEORGE	WASHINGTON	2,770	0.55	0.80	7.1
UT	3	NORTH CENTRAL	427598	SALT LAKE CITY INTL AP	SALT LAKE	4,225	0.51	0.80	15.3
UT	4	SOUTH CENTRAL	426135	NEPHI	JUAB	5,128	0.50	0.75	14.3
UT	5	NORTHERN MOUNTAINS	422385	ECHO DAM	SUMMIT	5,470	0.50	0.70	14.0
UT	6	UINTA BASIN	427395	ROOSEVELT RADIO	UINTAH	5,014	0.46	0.70	6.8
UT	7	SOUTHEAST	420738	BLANDING	SAN JUAN	6,032	0.58	0.90	11.6
VA	1	TIDEWATER	446139	NORFOLK INTL AP	NORFOLK (CITY)	30	1.01	1.76	43.9
VA	2	EASTERN PIEDMONT	447201	RICHMOND INTL AP	HENRICO	164	1.01	1.69	43.4
VA	3	WESTERN PIEDMONT	445120	LYNCHBURG INTL AP	CAMPBELL	940	0.93	1.55	40.6
VA	4	NORTHERN	448906	WASHINGTON/NAT., VA.	ARLINGTON	10	0.94	1.50	39.6
VA	5	CENTRAL MOUNTAIN	447285	ROANOKE INTL AP	ROANOKE	1,175	0.93	1.50	40.2
VA	6	SOUTHWESTERN MOUNTAIN	448547	TROUT DALE 3 SSE	GRAYSON	2,839	0.90	1.44	45.4
VT	1	NORTHEASTERN	431565	CORINTH	ORANGE	1,180	0.80	1.30	37.8
VT	2	WESTERN	431081	BURLINGTON WSO AP	CHITTENDEN	330	0.69	1.10	34.5

State	Climate Division Number	Climate Division Name	Selected COOP Station	COOP Station Name	County	Elevation, MSL	Calculated 85th Percentile, 24-hour Storm Depth	Calculated 95th Percentile, 24-hour Storm Depth	Calculated Average Annual Precipitation Depth, inches
VT	3	SOUTHEASTERN	438556	UNION VILLAGE DAM	ORANGE	460	0.77	1.26	34.3
WA	1	WEST OLYMPIC COAST	456858	QUILLAYUTE AP	CLALLAM	185	1.20	1.97	101.9
WA	2	NE OLYMPIC SAN JUAN	456678	PORT TOWNSEND	JEFFERSON	100	0.46	0.72	22.3
WA	3	PUGET SOUND LOWLANDS	456114	OLYMPIA AP	THURSTON	188	0.79	1.27	47.9
WA	4	E OLYMPIC CASCADE FOOTHILLS	453357	GREENWATER	PIERCE	1,730	0.79	1.31	52.6
WA	5	CASCADE MOUNTAINS WEST	458009	STAMPEDE PASS	KITTITAS	3,959	1.07	1.80	80.3
WA	6	EAST SLOPE CASCADES	454849	LUCERNE 1 N	CHELAN	1,200	0.68	1.16	26.7
WA	7	OKANOGAN BIG BEND	453515	HARRINGTON 1 NW	LINCOLN	2,160	0.50	0.66	12.7
WA	8	CENTRAL BASIN	458207	SUNNYSIDE	YAKIMA	747	0.40	0.60	7.6
WA	9	NORTHEASTERN	457938	SPOKANE INTL AP	SPOKANE	2,353	0.45	0.66	16.5
WA	10	PALOUSE BLUE MOUNTAINS	456789	PULLMAN 2 NW	WHITMAN	2,545	0.51	0.80	21.4
WI	1	NORTHWEST	470349	ASHLAND EXP FARM	BAYFIELD	650	0.80	1.34	28.9
WI	2	NORTH CENTRAL	476939	RAINBOW RSVR TOMAHAWK	ONEIDA	1,600	0.74	1.20	27.9
WI	3	NORTHEAST	476510	PESHTIGO	MARINETTE	600	0.80	1.30	28.7
WI	4	WEST CENTRAL	475948	NEW RICHMOND	ST. CROIX	1,000	0.90	1.50	29.7
WI	5	CENTRAL	471676	CLINTONVILLE	WAUPACA	802	0.82	1.39	30.1
WI	6	EAST CENTRAL	473269	GREEN BAY A S INTL AP	BROWN	687	0.74	1.18	27.8
WI	7	SOUTHWEST	474546	LANCASTER 4 WSW	GRANT	1,040	0.90	1.50	32.7
WI	8	SOUTH CENTRAL	474961	MADISON DANE CO AP	DANE	866	0.83	1.42	32.1
WI	9	SOUTHEAST	475479	MILWAUKEE MITCHELL AP	MILWAUKEE	670	0.79	1.33	31.9
WV	1	NORTHWESTERN	466859	PARKERSBURG	WOOD	620	0.78	1.18	37.4
WV	2	NORTH CENTRAL	465002	LAKE LYNN	MONONGALIA	900	0.75	1.20	38.4
WV	3	SOUTHWESTERN	461570	CHARLESTON YEAGER AP	KANAWHA	910	0.79	1.24	43.0
WV	4	CENTRAL	462718	ELKINS RANDOLPH CY AP	RANDOLPH	1,979	0.72	1.12	44.3
WV	5	SOUTHERN	469011	UNION 3 SSE	MONROE	2,110	0.80	1.20	35.7
WV	6	NORTHEASTERN	465739	MATHIAS	HARDY	1,540	0.80	1.30	33.6

State	Climate Division Number	Climate Division Name	Selected COOP Station	COOP Station Name	County	Elevation, MSL	Calculated 85th Percentile, 24-hour Storm Depth	Calculated 95th Percentile, 24-hour Storm Depth	Calculated Average Annual Precipitation Depth, inches
WY	1	YELLOWSTONE DRAINAGE	485345	LAKE YELLOWSTONE	TETON	7,870	0.43	0.70	20.2
WY	2	SNAKE DRAINAGE	486440	MORAN 5 WNW	TETON	6,798	0.50	0.77	23.5
WY	3	GREEN AND BEAR DRAINAGE	487845	ROCK SPRINGS AP	SWEETWATER	6,741	0.40	0.60	4.2
WY	4	BIG HORN	488852	TENSLEEP 4NE	WASHAKIE	4,815	0.60	0.90	12.9
WY	5	POWDER, LITTLE MISSOURI, TONGU	488155	SHERIDAN AP	SHERIDAN	3,945	0.52	0.83	14.3
WY	6	BELLE FOURCHE DRAINAGE	487270	PINE TREE 9 NE	CAMPBELL	5,111	0.60	1.11	11.5
WY	7	CHEYENNE & NIOBRARA DRAINAGE	486660	NEWCASTLE	WESTON	4,315	0.60	1.00	15.0
WY	8	LOWER PLATTE	481570	CASPER WSCMO	NATRONA	5,338	0.45	0.80	11.9
WY	9	WIND RIVER	485390	LANDER AP	FREMONT	5,557	0.60	0.97	12.9
WY	10	UPPER PLATTE	487105	PATHFINDER DAM	NATRONA	5,918	0.50	0.75	9.2

Appendices C through F

NCHRP Report 802: Volume Reduction of Highway Runoff in Urban Areas: Guidance Manual

Appendix C

Infiltration Testing and Factors of Safety in Support of the Selection and Design of Volume Reduction Approaches (White Paper #1)

1 Introduction

Characterization of potential infiltration rates is a critical step in evaluating the degree to which infiltration can be used to reduce stormwater runoff volume. There are numerous methods for measuring or estimating infiltration rates of soils; however not all methods are equally applicable to stormwater facility siting and design. Likewise, the concept of a “factor of safety” has a range of general meanings in engineering design, and has a more specific meaning in its use for stormwater infiltration design and resulting volume reduction.

The purpose of this white paper is to synthesize guidance on infiltration rate characterization that is specific to stormwater volume reduction. This white paper is intended to provide guidance to help answer the following questions:

1. How and where does infiltration testing fit into the project development process?
2. What methods are commonly used to assess and measure infiltrative capacity for stormwater applications?
3. What factors should I consider in selecting the most appropriate testing method for my project?
4. Do I need to apply a factor of safety to infiltration rates? If so, how should I select and apply this factor?

This white paper is intended to provide an overview of infiltration testing and how it fits into the development process. This paper is based on a review of stormwater guidance documents, focused literature review of key topics, Geosyntec’s design and construction experience, and professional judgment. The paper is not intended to be an exhaustive reference on infiltration testing. It does not attempt to discuss every method for testing, nor is it intended to provide step-by-step procedures for each method. The user is directed to supplemental resources (referenced in this white paper) or other appropriate references for more specific information.

Note, that this white paper does not consider other feasibility criteria that may make infiltration infeasible, such as groundwater contamination and geotechnical considerations (these will be covered under separate white papers). In general, infiltration testing should only be conducted if other feasibility criteria have been evaluated and cleared.

2 Role of Infiltration Testing in Different Stages of Project Development

How and where does infiltration testing fit into the project development process?

In the process of planning and designing infiltration facilities, there are a number of ways that infiltration testing or estimation factors into project development. At the project planning phases, a designer faces the questions: *Where within my project area is infiltration potentially feasible? What volume reduction approaches are potentially suitable for my project?* If this initial screening returns positive results and volume reduction approaches that include infiltration are selected, then the designer faces further questions at later phases of design: *What infiltration rates should I use to design volume reduction facilities? What factor of safety should I apply*?

2.1 Site Assessment / Project Planning Phase

Site assessment efforts for infiltration potential should ideally be initiated early in the design process so that volume reduction approaches can be incorporated into the project layout as it is developed. At this phase, it may still be possible to adjust the project layout to preserve areas that have good opportunity for infiltration and configure project grading and drainage so that water can be routed to these areas. There are many factors that influence highway design, such that these projects may have less opportunity for adjustment to layout (i.e., alignment) than other types of projects; however adjustments to grading and drainage routing to improve infiltration opportunities may still be possible if this is initiated early in the design process. At this phase of project development, the project team is faced with two key questions:

- *Where within my project area is infiltration potentially feasible?*
- *What volume reduction approaches are potentially suitable for my project?*

The amount of information available to answer these questions at the planning phase of project development may vary. For example, at this phase, project planners may have access to extensive geotechnical investigation reports from previous projects, the results of early investigations for the project of interest, and other information, or may be faced with much more limited data such as county soils maps. A key tradeoff exists between the costs of acquiring additional data and adequacy of existing data for planning-level decisions. If too little information is available, then key opportunities for stormwater infiltration may be missed. For example, in conditions with high variability in soils, testing at 1,000-foot spacing may not identify significant areas of permeable soils between borings that have high opportunity for infiltration. Similarly, if too little information is available, the potential for infiltration may be over-stated. For example, in Seattle, the failure of a right-of-way infiltration project was partially attributed to inadequate spatial resolution of infiltration testing data collected at the project planning phase and associated over-estimation of infiltration capacity (Colwell and Tackett, undated). However, conducting infiltration tests can be costly, and there is a practical limit to how much effort can be allocated to this line item in the project budget.

Existing guidance has addressed this tradeoff in in a variety of ways (see further detail in Section 3). Some jurisdictions allow the use of simpler testing methods at the project planning phase that are less precise but also less costly. This can allow the planning-level investigation to cover a relatively broad scope with the intent of identifying areas where more intensive investigation will be focused. Other jurisdictions allow projects to rely only on mapped data, such as Natural Resources Conservation Service (NRCS) county soil surveys for planning-level assessment, but it is suggested that basic field screening of can be useful for early identification of fatal flaws that may not be caught using desktop methods. At the planning phase, it is generally recommended that screening for other feasibility factors, such as groundwater contamination, depth to groundwater, setbacks from wells and structures, and other criteria be applied first, before identifying potential areas for infiltration testing. This approach can improve efficiency by conducting testing in more focused locations after other feasibility factors have already been assessed and cleared.

In the absence of specific local guidance, the decision of whether to collect additional data at the project planning phase, and at what resolution, should be based on project-specific factors, such as:

- How variable are the soil conditions at the site? Can I reliably interpolate between a fewer number of tests?
- What are the project goals relative to volume reduction? How important is it to provide an exhaustive investigation and quantification of infiltration opportunities? Do applicable regulations require a rigorous demonstration that infiltration is infeasible?
- How much would infiltration testing cost as a portion of the project budget? Could other design costs potentially be reduced (i.e., conveyance, flood control) if increased budget were allocated to thoroughly investigating infiltration opportunities?

Section 3 provides a summary of the planning-level screening methods currently in use by selected DOTs and other agencies, and Section 4 provides additional information related to selecting and applying these methods.

2.2 BMP Design Phase

At the BMP design phase, a more detailed and accurate assessment is needed to quantify infiltration rates for each BMP location. When designing a project to meet specific surface runoff volume reduction goals, the rate at which water percolates into underlying soils is a critical design parameter that affects the time it takes for the BMP to drain as well as the amount of storage capacity available to accept runoff from subsequent storm events. Therefore an accurate estimate of design infiltration rates is clearly a critical need. Overestimating infiltrative capacity can result in failed facilities (i.e., facilities that drain more slowly than intended), with subsequent cost implications due to remediation efforts for these facilities. Underestimating infiltrative capacity can result in over-designing infiltration facilities and associated costs. At this phase of project development, the project team is faced with two key questions:

- *What infiltration rates should I use to design volume reduction facilities?*
- *What factor of safety should I apply?*

When developing an infiltration testing strategy to establish design infiltration rates for infiltration facilities, a tradeoff exists between the costs of testing and the quality and resolution of the data available to support design decisions. Similar to planning-level screening, there is always the option to collect additional data; however there is a conceptual “point of diminishing returns” beyond which the cost of acquiring additional data may not necessarily lead to better decision making.

Uncertainty that remains in the measured infiltration rate after field testing can, in part, be mitigated through the use of a higher factor of safety for design, as discussed in greater detail in Section 5. As such, there is a tangible economic tradeoff between the cost of acquiring more data and the additional facility construction cost associated with using a higher factor of safety in design (i.e., larger volumes, larger footprint). Guidance manuals generally grant discretion to the project engineer and plan review agency to use professional judgment to balance this tradeoff.

As a general rule, the value obtained from each design-level infiltration test can be improved by:

1. Using a tiered approach for investigation (i.e., planning-level screening in advance of design-level testing) such that more rigorous design-level tests are conducted only in areas where BMPs are likely to be placed.
2. Using proven testing methods that are acceptable to local jurisdictions and provide reliable estimates of infiltration rate.
3. Selecting methods that are applicable for the project conditions.

Section 4 provides a summary of commonly used testing methods for establishing design infiltration rates and provides discussion to assist in selecting testing methods.

3 Review of Infiltration Testing and Estimation Approaches Currently in Use by DOTs and Other Agencies

What methods are commonly used to assess and measure infiltrative capacity?

Guidance developed by selected state and local agencies was reviewed to provide a summary of the approaches that are currently in use by DOTs and other agencies for measuring and estimating infiltration capacity. Approaches to infiltration testing and estimation vary as summarized in Table 1. While this summary is not exhaustive, it is intended to provide the user with an introduction to the types of requirements and practices currently in place. A discussion of findings from this review is provided below. Greater detail on several of these infiltration testing methods is provided in Section 4.

Discussion of Findings Related to Review of Existing Guidance

With respect to the use of different methods for project planning versus design, recommendations vary by jurisdiction. Orange County, CA, Portland, OR, Maryland, and Caltrans include specific approaches for planning-level screening versus design, that include the use of simpler tests for planning-level screening versus more comprehensive approaches for design-level investigation. Other jurisdictions allow the use of desktop resources such as NRCS county soil surveys and/or other maps for initial screening, however these methods should always be supplemented with field observations. Still other jurisdictions do not specifically recognize a two-stage approach for planning-level and design-level exploration.

The more specific two-stage approach adopted by Orange County and Portland appears to be a function of the regulatory context under which the guidance was developed. In these areas, the project proponent must first evaluate infiltration, and must demonstrate that infiltration is infeasible before considering other options. For example, in Orange County, the municipal stormwater permit specifically requires each project to conduct a “rigorous” feasibility assessment for infiltration. As such, the respective guidance manuals are more specific about what must be done at the planning phase to determine if infiltration is feasible or not. The Caltrans *Infiltration Basin Siting Study* (Caltrans, 2003) was undertaken with the specific intent of identifying infiltration opportunities over a broad geographic area, therefore the study deliberately adopted a phased approach for narrowing down the number of potential sites for testing. In contrast, guidance developed by other jurisdictions was developed under a regulatory context in which infiltration is an option, but is not necessarily required. In this context, more freedom is granted to project proponents regarding site assessment to support BMP selection. For example, a designer may still be interested in determining where infiltration is feasible, but the proponent would not be required to demonstrate that they have adequately assessed infiltration opportunities before moving on to other non-infiltration BMP options. Current trends in stormwater regulations are moving toward requiring evaluation of infiltration feasibility.

In general, each jurisdiction accepts a number of different testing methods. More than a dozen infiltration testing methods are specified in the seven guidance manuals reviewed. In addition, some jurisdictions allow other tests to be used at the discretion of a project professional. As might be expected, some of these methods are simply variations on similar tests, although there remain a broad range of distinctly different approaches.

Six of the seven references reviewed specify that safety factors must be applied to field test results for use in assessing feasibility and/or for design. Approaches for selecting safety factors vary between jurisdictions. Some require a mandatory use of a specific factor of safety and others allow project proponents to select factors of safety based on a number of design considerations.

Table 1. Selected State and Local Government Approaches to Infiltration Testing

Jurisdiction	Planning Phase Requirements	Design Phase Requirements	Safety Factors
Orange County, CA	Small projects may rely solely on regional NRCS soils maps and data that are already available for the project, such as geotechnical investigations, groundwater maps, etc. If Hydrologic Soil Group (HSG) D soils or other severely limiting feasibility constraints are identified (i.e., very shallow groundwater, mapped contaminant plume), then no further investigation is needed to demonstrate infeasibility of infiltration. Larger projects must conduct infiltration measurements at the planning phase unless other factors make infiltration feasible. A "Simple Open Pit Infiltration Test" is recommended, however any method approved for design-level testing may also be used. At the planning phase, a licensed geotechnical engineer does not necessarily need to conduct the simple open pit test.	If initial feasibility screening finds that infiltration is potentially feasible, then the project must conduct detailed testing. Any of the following tests may be used to establish design infiltration rate, under the supervision of the project geotechnical professional: • Open Pit Falling Head • Well Permeameter Method (USBR 7300-89 test) • Percolation Test procedure from Riverside County (with conversion factor) • Double Ring Infiltrometer • Single Ring Infiltrometer • Other methods as approved by project engineer and reviewing agency	For planning-level feasibility screening, the infiltration rate from field measurements must be adjusted by a factor of safety of 2.0 as part of screening whether infiltration is feasible. If the adjusted infiltration rate is less than 0.3 in/hr, then infiltration is considered infeasible. For design purposes, a matrix must be used to select the design factor of safety considering site suitability (methods used, soil texture, site variability, depth to groundwater) and design factors (tributary area, level of pretreatment, redundancy of treatment, and compaction) to compute the design factor of safety. This method yields a total factor of safety from 2 to 9, Discretion is granted to the designer and reviewer.
Portland, OR	Allows "Simplified Approach Open Pit Infiltration Test" for initial infiltration rate screening. This test can be conducted by a "nonprofessional".	Any of the following tests may be used to establish design infiltration rate: • Open pit falling head • Encased falling head (6" single-ring) • Double ring infiltrometer (ASTM D3385)	Minimum safety factors depend on testing methods and the type of project: • Open pit falling head: 2 • Encased falling head test: 2 • Double ring infiltrometer: Public facilities: 1 Private facilities: 2 Higher safety factors may be used at the discretion of the engineer and reviewer.

Jurisdiction	Planning Phase Requirements	Design Phase Requirements	Safety Factors
Caltrans[1]	In this study, screening for potential locations with suitable infiltration rates consisted of the following steps: • Conduct desktop analysis, including NRCS Soil Survey hydrologic soil groups and clay content. • Characterize subsurface lithography and depth to groundwater using boreholes; collect continuous cores. • Measure hydraulic conductivity of core samples obtained from borings (laboratory). • Conduct in-hole tests using USBR 7300-89 well permeameter method. • Evaluate other feasibility criteria such as setbacks from roadway and depth to groundwater.	For sites identified as higher potential, the study conducted detailed assessment, including: • Conduct in-hole hydraulic conductivity tests using USBR 7300-89 well permeameter method at higher spatial resolution. At least 4 tests performed at each site. • For some sites, the study recommended additional tests be conducted at the design level, including extended tests conducted over 48 hours with duplicate runs, to confirm design rates.	Applied factor of safety of 2.0 to borehole testing results after converting borehole measurements to estimates of vertical hydraulic conductivity.
Maryland (including MDOT)	Initial infeasibility screening involves one field test per facility, regardless of type or size, or use of previous testing data, such as the following: • On-site septic percolation testing, which can establish initial rate, water table and/or depth to bedrock, • Geotechnical report on the site prepared by a qualified geotechnical consultant, or • NRCS Soil Mapping showing an unsuitable soil group such as a hydrologic group “D” soil in a low lying area or a Marlboro Clay (expansive).	If initial testing yields the finding that probable infiltration rate is greater than 0.52 in/hr, then the project must conduct both of the following tests to establish design infiltration rate: • Dig test pit to evaluate depth to groundwater, depth to bedrock, soil texture, and other factors. • Conduct encased falling head infiltration test (5 inch diameter). The use of lab testing to establish infiltration rates is explicitly prohibited.	Factors of safety are not explicitly considered in the manual.

Jurisdiction	Planning Phase Requirements	Design Phase Requirements	Safety Factors
Wisconsin (including WisDOT)	Allows use of desktop resources based on soil texture to evaluate infiltration potential. Requires field verification of some characteristics.	No specific infiltration test required. If a double ring infiltrometer is used, the test must be done per ASTM D3385.	Safety factor based on ratio of permeability of various soil horizons in 5 feet below proposed facility bottom elevation. Analysis of groundwater mounding potential must be conducted.
New Jersey (including NJDOT)	Does not provide specific guidance for planning-phase testing.	The design manual specifies the following testing methods for determining design infiltration rate: • Tube permeameter test (laboratory) • Percolation Test • Pit Bailing Test –Procedure given • Basin flooding test for bedrock • Double ring infiltrometer (ASTM 3385) • USBR 7300-89 (Well permeameter test) • Other recognized constant head permeability test	Post-construction testing is required to demonstrate that the facility drains within 72 hours with a safety factor of 2.

Jurisdiction	Planning Phase Requirements	Design Phase Requirements	Safety Factors
Western Washington (including WSDOT)	Specific criteria for planning phase assessment are not included in the Manual.	The manual's simplified approach for sites less than 1 acre allows the use of any of the following: • Large-scale Pilot Infiltration Test (PIT) (100 sq-ft surface area) • Smaller-scale PIT (20 to 32 sq-ft surface area) • Smaller-scale tests such as double-ring or falling head area allowed with appropriate correction factor • For unconsolidated soils, a grain size analysis method may be used. The manual requires a detailed approach for larger tributary areas, including : • Subsurface explorations (test holes or test pits) to a minimum depth below the system. • Continuous sampling to a minimum depth below the system. • One of the following tests: o Large-scale Pilot Infiltration Test (PIT) preferred o Smaller-scale tests such as double-ring or falling head allowed with appropriate correction factor o Grain size analysis allowed to estimate infiltration rate for unconsolidated soils; must collect one test per stratum encountered in borings. • Assessment of "infiltration receptor" to evaluate capacity.	Three safety factors are multiplied to yield the total safety factor. These factors account for (1) testing methods, (2) system geometry, and (3) potential for clogging. The combined range of safety factors ranges from approximately 2 to 18. For larger projects, a groundwater mounding model must be run to evaluate potential groundwater mounding issues.

1 – As conducted as part of Infiltration Basin Site Selection Study, Volume I, Report No. CTSW-RT-03-025 (Caltrans, 2003).

4 Guidance for Selecting Infiltration Testing Methods

How do I determine which methods are most appropriate for my project?

In order to select an infiltration testing method, it is important to understand how each test is applied and what specific physical properties the test is designed to measure. Infiltration testing methods vary considerably in these regards. For example, a borehole percolation test is conducted by drilling a borehole, filling a portion of the hole with water, and monitoring the rate of fall of the water. This test directly measures the 3 dimensional flux of water into the walls and bottom of the borehole. An approximate correction is applied to indirectly estimate the vertical hydraulic conductivity from the results of the borehole test. In contrast, a double-ring infiltrometer test is conducted from the ground surface and is intended to provide a direct estimate of vertical (one-dimensional) infiltration rate at this point. Both of these methods are applicable under different conditions.

Tests can be differentiated based on a three key factors:

Scale of test: The testing methods described below range from small-scale point measurements to larger scale methods that inundate up to more than 100 square feet. While the cost of testing at larger scales can be prohibitive due to amount of excavation and water needed, the advantage of larger tests is that these tests tend to be more resistant to error introduced by spatial variability in soil properties. Particularly in soils with high variability (i.e., complex layering, non-uniform consistency), the results of small tests may be biased by localized properties that do not necessarily represent the bulk infiltration rate of the larger area of soils that the infiltration system would overlay. For example, a point measurement in an area overlying a small sand lens could significantly over-predict infiltration rates in comparison to what would be expected in full-scale implementation. Larger scale tests also tend to better approximate the "dimensionality" of BMPs, as discussed below.

Dimensionality of test: While some testing procedures attempt to constrain the direction of infiltration to one dimension (e.g., double-ring infiltrometers), each testing method tends to include some degree of lateral infiltration and vertical infiltration as a function of its dimensionality. This also tends to be true of infiltration BMPs, which infiltrate water into the surrounding soils through both their bottoms and their side walls, in various proportions. Ideally, a test would be used that approximates the dimensionality of the proposed infiltration system to be constructed. However, for purposes of normalizing testing methods, standardized tests are typically used. In selecting a test, and determining what potential for error it may include, the dimensionality of the test in comparison to the dimensionality of the proposed BMP is important to consider.

Elevation of infiltrating surface: Testing should be conducted at and below the elevation of the proposed infiltrating surface. In some cases this may be well below the existing ground surface, which may influence the testing that can be conducted. From a practical perspective, it is not possible to conduct a double ring infiltrometer test at an elevation far below the ground surface without extensive excavation. However a well permeameter or borehole method would be well suited in this case. The presence of lower permeable materials at or below the bottom of the BMP, such as fine-grained soil (clays and silts) at depth can significantly reduce the infiltration capacity. Surface testing may miss this.

This section provides a summary and comparison of the common testing methods currently in use. Table 2 provides a matrix comparison of these methods.

4.1 Desktop Approaches and Data Correlation Methods

This section reviews common methods used to evaluate infiltration characteristics based on desktop-available information available, such as GIS data. This section also introduces methods for estimating infiltration properties via correlations with other measurements.

4.1.1 NRCS Soil Survey Maps and Similar

NRCS Soil Surveys are generally available nationwide, and provide a wealth of information about the general geographic distribution of soil units, as well as properties of these soil units near the ground surface (http://websoilsurvey.nrcs.usda.gov/app/HomePage.htm) Soil survey information, specifically characterization of hydrologic soil groups (HSGs), soil texture classes, and presence of hydric soils, can provide useful information for quickly evaluating infiltration potential on a broad geographic context. Several of the jurisdictions reviewed in Section 3 allow the use of soil surveys and/or similar datasets for planning-level screening. Geologic maps may also be available for certain areas at similar spatial resolution. These tend to be more appropriate than the NRCS soils surveys for evaluating the properties of geologic formations below the surficial soil layer, and they can also provide useful information for infiltration feasibility screening.

However, guidance manuals and studies generally recommend that these types of datasets should be used with care. The FHWA Urban Drainage Design Manual (Federal Highway Administration, 2001) states that “Although infiltration rates are published in many county soils reports, it is advised that good field measurements be made to provide better estimates for these parameters.” Similarly, the Orange County (CA) Technical Guidance Document allows the use of soils maps for feasibility screening as an option for small projects, but only on the condition that the soil type is HSG D and the mapped soil type is confirmed through information available at the specific site. Caltrans (2003) found that use of HSG D classification to exclude study locations resulted in excluding locations that may actually have been found to have good properties for infiltration had site testing been conducted. Generally, confirming mapped conditions with available data from the site (e.g., soil borings, observed soil textures, biological indicators) can provide an inexpensive means of improving the reliability of using regional maps.

4.1.2 Grain Size Analysis

Hydraulic conductivity can be estimated indirectly from correlations with soil grain size distributions. While this method is approximate, correlations have been relatively well established for some soil conditions. One of the most commonly used correlations between grain size parameters and hydraulic conductivity is the Hazen (1892, 1911) empirical formula (Philips and Kitch, 2011), but a variety of others have been developed.

WADOE and WSDOT accept estimates of infiltration rate developed based on soil grain size distribution. Their method, developed from local experience, uses the ASTM soil size distribution test procedure (ASTM D422), which considers the full range of soil particle sizes, to develop soil size distribution curves. An empirical formula was derived (Massmann 2003, and Massmann et al., 2003) to relate the D10, D60 and D90 to the saturated hydraulic conductivity of an unconsolidated soil sample. The D10, D60, and D90 are the grain sizes for which 10 percent, 60 percent and 90 percent of the sample (by weight) is finer. This analysis must be done for each soil layer encountered below the system to a minimum depth. WADOE and WSDOT accept this method only for soils that have not been consolidated by glacial advance.

Philips and Kitch (2011) found that this method did not consistently align with direct measurements. For their test sites, it was found to result in considerably high estimates compared to direct tests at some sites in which the in-situ material was consolidated to some degree. This is expected, given that compaction of soil has been observed to have significant influence on infiltration rates (Pitt et al. 2008; Cedergren, 1997). Several researchers have also noted high sensitivity of soil infiltration rate to the percent of fines (Cedergren 1997; Hinman 2009, and others) which may not be adequately accounted for using regression methods. For these reasons, grain size methods are considered to have limited reliability for estimating infiltration rates.

4.1.3 Cone Penetrometer Testing

Hydraulic conductivity can also be estimated indirectly from cone penetrometer testing (CPT). A cone penetrometer test involves advancing a small probe into the soil and measuring the relative resistance encountered by the probe as it is advanced. The signal returned from this test can be interpreted to yield estimated soil types and the location of key transitions between soil layers. Correlations have also been developed between CPT data and hydraulic conductivity (Lunne et al. 1997). Philips and Kitch (2011) found this method to be highly variable compared to direct measurement. Additional field experience with these methods has not been identified. In general, this method may be useful as an initial planning tool, but does not appear to be reliable for decision making in most cases.

4.2 Surface and Shallow Excavation Methods

This section describes tests that are conducted at the ground surface or within shallow excavations close to the ground surface. These tests are generally applicable for cases where the bottom of the infiltration system will be near the existing ground surface. They can also be conducted to confirm the results of borehole methods after excavation/site grading has been completed.

4.2.1 Simple Open Pit Test

The Simple Open Pit Test is most appropriate for planning-level screening of infiltration feasibility. Although it is similar to Open Pit Falling Head tests used for establishing a design infiltration rate (see below), the Simple Open Pit Test is less rigorous and is generally conducted to a lower standard of care. Portland (OR) and Orange County (CA) allow this test to be conducted by a nonprofessional as part of planning-level screening phase.

The Simple Open Pit Test is a falling head test in which a hole at least 2 feet in diameter is filled to a level of 6” above the bottom. Water level is checked and recorded regularly until either an hour has passed or the entire volume has infiltrated. The test is repeated two more times in succession and the rate at which the water level falls in the third test is used as the infiltration rate.

This test has the advantage of being inexpensive to conduct. Yet it is believed to be fairly reliable for screening as the dimensions of the test are similar, proportionally, to the dimensions of a typical BMP. The key limitations of this test are that it measures a relatively small area, does not necessarily result in a precise measurement, and may not be uniformly implemented.

Source: City of Portland, 2008. Stormwater Management Manual, Appendix F.2.

4.2.2 Open Pit Falling Head Test

This test is similar to the Simple Open Pit Test, but covers a larger footprint, includes more specific instructions, returns more precise measurements, and generally should be overseen by a geotechnical professional. Nonetheless, it remains a relatively simple test.

To perform this test, a hole is excavated at least 2 feet wide by 4 feet long (larger is preferred) and to a depth of at least 12 inches. The bottom of the hole should be approximately at the depth of the proposed infiltrating surface of the BMP. The hole is presoaked the by filling it with water at least a foot above the soil to be tested and leaving it at least 4 hours (or overnight if clays are present). After pre-soaking, the hole is refilled to a depth of 12 inches and allow it to drain for one hour (2 hours for slower soils), measuring the rate at which the water level drops. The test is then repeated until successive trials yield a result with less than 10 percent change.

In comparison to a double-ring infiltrometer, this test has the advantage of measuring infiltration over a larger area and better resembles the dimensionality of a typical small-scale BMP. Because it includes both vertical and lateral infiltration, it should be adjusted to estimate design rates for larger scale BMPs.

Source: County of Orange (2011)

4.2.3 Double Ring Infiltrometer Test (ASTM 3385)

The Double Ring Infiltrometer was originally developed to estimate the saturated hydraulic conductivity of low permeability materials, such as clay liners for ponds, but has seen significant use in stormwater applications. The most recent revision of this method from 2009 is known as ASTM 3385-09. The testing apparatus is designed with concentric rings that form an inner ring and an annulus between the inner and outer rings. Infiltration from the annulus between the two rings is intended to saturate the soil outside of the inner ring such that infiltration from the inner ring is restricted primarily to the vertical direction.

To conduct this test, both the center ring and annulus between the rings are filled with water. There is no pre-wetting of the soil in this test. However, a constant head of 1 to 6 inches is maintained for 6 hours, or until a constant flow rate is established. Both the inner flow rate and annular flow rate are recorded, but if they are different, the inner flow rate should be used. There are a variety of approaches that are used to maintain a constant head on the system, including use of a Mariotte tube, constant level float valves, or manual observation and filling. This test must be conducted at the elevation of the proposed infiltrating surface, therefore application of this test is limited in cases where the infiltration surface is a significant distance below existing grade at the time of testing.

This test is generally considered to provide a direct estimate of vertical infiltration rate for the specific point tested and is highly replicable. However, given the small diameter of the inner ring (standard diameter is 12 inches, but it can be larger), this test only measures infiltration rate in a small area. Additionally, given the small quantity of water used in this test compared to larger scale tests, this test may be biased high in cases where the long term infiltration rate is governed by groundwater mounding and the rate at which mounding dissipates (i.e., the capacity of the infiltration receptor). Finally, the added effort and cost of isolating vertical infiltration rate may not necessarily be warranted considering that BMPs typically have a lateral component of infiltration as well. Therefore, while this method has the advantages of being technical rigorous and well standardized, it should not necessarily be assumed to be the most representative test for estimating full-scale infiltration rates. Source: ASTM International (2009)

4.2.4 Single Ring Infiltrometer Test

The single ring infiltrometer test is not a standardized ASTM test, however it is a relatively well-controlled test and shares many similarities with the ASTM standard double ring infiltrometer test (ASTM 3385-09). This test is a constant head test using a large ring (preferably greater than 40 inches in diameter) usually driven 12 inches into the soil. Water is ponded above the surface. The rate of water addition is recorded and infiltration rate is determined after the flow rate has stabilized. Water can be added either manually or automatically.

The single ring used in this test tends to be larger than the inner ring used in the double ring test. Driving the ring into the ground limits lateral infiltration; however some lateral infiltration is generally considered to occur. Experience in Riverside County (CA) has shown that this test gives results that are close to full-scale infiltration facilities (Riverside County, 2011). This finding is also supported by King County's Surface Water Design Manual (2009).

The primary advantages of this test are that it is relatively simple to conduct and has a larger footprint (compared to the double-ring method) and restricts horizontal infiltration and is more standardized (compared to open pit methods). However, it is still a relatively small-scale test and can only be reasonably conducted near the existing ground surface.

4.2.5 Encased Borehole Tests

Encased borehole test methods are similar to single ring infiltrometer tests; however they are typically conducted using a smaller diameter encasement (typically 6 to 12 inches) driven into the native soil that is allowed to drain completely for each test. The encasement ensures that water moves primarily in the vertical direction through the soil plug in the encasement. Generally, these tests measure a smaller surface area than single ring infiltrometers and therefore have greater inherent uncertainty related to spatial heterogeneity. However, they may be less expensive to conduct and practical at greater depths below existing grade. The City of Portland *Encased Falling Head Test* is an example (City of Portland 2008). Similar methods are used in other jurisdictions.

4.2.6 Large-scale Pilot Infiltration Test (PIT)

As its name implies, this test is closer in scale to a full-scale infiltration facility. This test was developed by WADOE specifically for stormwater applications.

To perform this test, a test pit is excavated with a horizontal surface area of roughly 100 square feet to a depth that allows 3 to 4 feet of ponding above the expected bottom of the infiltration facility. Water is continually pumped into the system to maintain a constant water level (between 3 and 4 feet about the bottom of the pit, but not more than the estimated water depth in the proposed facility) and the flowrate is recorded. The test is continued until the flow rate stabilizes. Infiltration rate is calculated by dividing the flowrate by the surface area of the pit. Similar to other open pit test, this test is known to result in a slight bias high because infiltration also moves laterally through the walls of the pit during the test. WADOE requires a correction factor of 0.75 (factor of safety of 1.33) be applied to results.

This test has the advantage of being more resistant to bias from localized soil variability and being more similar to the dimensionality and scale of full-scale BMPs. It is also more likely to detect long term decline in infiltration rates associated with groundwater mounding. As such, it remains the preferred test for establishing design infiltration rates in Western Washington (WADOE 2012). In a comparative evaluation of test methods, this method was found to provide a more reliable estimate of full-scale infiltration rate than double ring infiltrometer and borehole percolation tests (Philips and Kitch 2011). King County's Surface Water Design Manual (2009) states that large single ring infiltrometer and PIT tests have proven more effective than smaller test methods at matching as-built performance of infiltration facilities.

The difficulty encountered in this method is that it requires a larger area be excavated than the other methods, and this in turn requires larger equipment for excavation and a greater supply of water. However, this method should be strongly considered when less information is known about spatial variability of soils and/or a higher degree of certainty in estimated infiltration rates is desired. WADOE (2012) incentivizes the use of this test by allowing a lower safety factor to be applied to testing results in comparison to the safety factors that must be applied to the results of smaller-scale tests.

Source: Washington State Department of Ecology, WADOE (2012)

4.2.7 Smaller-scale Pilot Infiltration Test (PIT)

The smaller-scale PIT is conducted similarly to the large-scale PIT but involves a smaller excavation, ranging from 20 to 32 square feet instead of 100 square feet for the large-scale PIT, with similar depths. The primary advantage of this test compared to the full-scale PIT is that it requires less excavation volume and less water. It may be more suitable for small-scale distributed infiltration controls where the need to conduct a greater number of tests outweighs the accuracy that must be obtained in each test, and where groundwater mounding is not as likely to be an issue. WADOE establishes a correction factor of 0.5 (factor of safety of 2.0) for this test in comparison to 0.75 for the large-scale PIT to account for a greater fraction of water infiltrating through the walls of the excavation and lower degree of certainty related to spatial variability of soils.

4.3 Deeper Subsurface Tests

4.3.1 Well Permeameter Method (USBR 7300-89)

Well permeameter methods were originally developed for purposes of assessing aquifer permeability and associated yield of drinking water wells. This family of tests is most applicable in situations in which infiltration facilities will be placed substantially below existing grade, which limits the use of surface testing methods.

In general, this test involves drilling a 6 inch to 8 inch test well to the depth of interest and maintaining a constant head until a constant flow rate has been achieved. Water level is maintained with down-hole floats. The Porchet method or the nomographs provided in the USBR Drainage Manual (U.S. Department of the Interior, Bureau of Reclamation, 1993) are used to convert the measured rate of percolation to an estimate of vertical hydraulic conductivity. A smaller diameter boring may be adequate, however this then requires a different correction factor to account for the increased variability expected.

While these tests have applicability in screening level analysis, considerable uncertainty is introduced in the step of converting direct percolation measurements to estimates of vertical infiltration. Additionally, this testing method is prone to yielding erroneous results cases where the vertical horizon of the test intersects with minor lenses of sandy soils that allow water to dissipate laterally at a much greater rate than would be expected in a full-scale facility. To improve the interpretation of this test method, a continuous soil core can be extracted from the bore hole to determine whether thin lenses of material may be biasing results at the strata where testing is conducted. This boring should also be extended below the depth of the test, following the completion of the test.

Source: (U.S. Department of the Interior, Bureau of Reclamation, 1990, 1993)

4.3.2 Borehole Percolation Tests (Various Methods)

Borehole percolation tests were originally developed as empirical tests to estimate the capacity of on-site sewage disposal systems (septic system leach fields), but have more recently been adopted into use for evaluating stormwater infiltration. Similar to the well permeameter method, borehole percolation methods primarily measure lateral infiltration into the walls of the boring and are designed for situations in which infiltration facilities will be placed well below current grade. The percolation rate obtained in this test should be converted to an infiltration rate using a technique such as the Porchet method.

This test is generally implemented similarly to the USBR Well Permeameter Method. Per the Riverside County Borehole Percolation method, a hole is bored to a depth at least 5 times the borehole radius. The hole is presoaked for 24 hours (or at least 2 hours if sandy soils with no clay). The hole is filled to approximately the anticipated top of the proposed infiltration basin. Rates of fall are measured for 6 hours, refilling each half hour (or 10 minutes for sand). Tests are generally repeated until consistent results are obtained.

The same limitations described for the well permeameter method apply to borehole percolation tests, and their applicability is generally limited to initial screening. To improve the interpretation of this test method, a continuous soil core can be extracted from the hole and below the test depth, following testing, to determine whether thin lenses of material may be biasing results at the strata where testing is conducted.

Sources: Riverside County Percolation Test (2011), California Test 750 (1986), San Bernardino County Percolation Test (1992); U.S. EPA Falling Head Test (1980).

Porchet Method (aka Inverse Auger Hole Method) for Estimating Saturated Hydraulic Conductivity from Borehole Test Results

The Porchet Method (or Inverse Auger Hole Method) is used to estimate one-dimensional saturated hydraulic conductivity of soil based on measurements of the rate of fall of water in a borehole, collected during a borehole percolation test (van Hoorn, 1979, Food and Agriculture Organization of the United Nations, 2007). Data should be recorded after the borehole test has stabilized (i.e., drawdown rates do not vary considerably between sequential trials). When the drop in head is relatively small compared to the total height of water, the following simplified conversion equation can be used (Orange County, 2011):

$$I_t = \frac{\Delta H(60r)}{\Delta t(r + 2H_{avg})}$$

Where:

"ΔH" is the change in height over the time interval ($H_o - H_f$). inches

"H_o" is the initial height of water at the selected time interval, measured from the bottom of the hole (inches)

"H_f" is the final height of water at the selected time interval, measured from the bottom of the hole (inches)

“H_{avg}” is the average height of water in the hole over the time interval, measured from the bottom of the hole, inches

“Δt” = test interval, minutes

“r” is the test hole radius, inches

“I_t” is the resulting infiltration rate, inches per hour.

When the ΔH is more than 25 percent of the H_{avg}, more detailed equations may be necessary. See Food and Agriculture Organization of the United Nations (2007) for more detailed methods.

4.3.3 Tube Permeameter Test

The Tube Permeameter Test provides an option for estimating infiltration rates in cases where the infiltration surface is well below the ground surface and in-situ borehole testing cannot be conducted.

The tube permeameter test is a falling head test in which a core sample, 6 inches long between 1.5 and 3 inches in diameter, is taken from the ground to be tested. The sample is presoaked in de-aired water, and then a column of water is allowed to drain down through the sample.

While this test may be reliable for planning-level screening, it has a number of limitations. First, this test is subject to bias introduced by localized variability as a result of the narrow diameter and relatively small dimensions of the core. Second, the removal of the sample can disturb the sample and change its infiltration properties. Finally, this method is not practical for non-cohesive samples due to difficulties in obtaining an intact core sample. For non-cohesive soils, it may be more appropriate to obtain a soil sample and remold it for testing. Furthermore, it is less likely that cohesive soil, which is generally fine grained, will provide suitable infiltration characteristics. Note that some jurisdictions, such as Maryland, explicitly prohibit laboratory methods from being used to establish design infiltration rates.

Source: New Jersey Department of Environmental Protection (2009)

Table 2. Summary Matrix of Infiltration Rate Estimation and Testing Methods

Test	Spatial Scale	Dimensionality	Vertical Strata	Suitable for Planning-Level Screening	Suitable for Design
Infiltration Rate Estimation Methods					
NRCS Soil Survey Maps	Geographic/ landscape scale (regional maps)	NA	NRCS soil maps generally provide information for multiple strata; generally within 10 to 20 feet of surface. Geologic maps may be more reliable for deeper subsurface properties.	Potentially suitable, but should be interpreted with caution. Should be confirmed with site observations if possible.	Not generally suitable for design unless a large factor of safety is applied.
Grain Size Analysis	Point measurement; however it is relatively simple to obtain a large number of measurements.	Correlations provide estimates of vertical Ksat.	Samples can be obtained from any strata.	Potentially suitable for unconsolidated soils only. Reliability can be improved if correlations are derived and validated based on local soil types.	Accepted in WA State for small-scale design in unconsolidated soils, with appropriate factor of safety. Other locations should confirm that correlations are applicable to local soils.
Cone Penetrometer Testing	Point measurement; however relatively simple to obtain a large number of measurements.	Correlations provide estimates of vertical Ksat.	Continuous through vertical strata.	Potentially suitable; reliability improved if correlations are derived and validated locally.	Not generally acceptable for design.

Test	Spatial Scale	Dimensionality	Vertical Strata	Suitable for Planning-Level Screening	Suitable for Design
		Infiltration Testing Methods			
Simple Open Pit Test	Point measurement (~4 sq-ft)	Has a greater component of horizontal flow than expected in most BMPs. Correction should be applied.	Applicable for near-surface only.	Generally suitable.	Not generally accepted as a design-level test. Can be acceptable if conducted as an *Open Pit Falling Head* test with professional oversight.
Open Pit Falling Head Test	Larger point measurement (~8 sq-ft; greater preferred)	Greater component of horizontal flow than expected in most BMPs. Correction should be applied.	Applicable for near-surface only.	Suitable with correction for dimensionality.	Suitable with correction for dimensionality.
Double Ring Infiltrometer Test (ASTM 3385)	Point measurement (~1 sq-ft)	Relatively true estimate of vertical rates.	Applicable for near-surface only.	Suitable, but may be cost prohibitive for preliminary screening of a large area.	Generally suitable.
Single Ring Infiltrometer Test	Larger point measurement (~10 sq-ft)	Primarily vertical direction, but with some horizontal direction.	Applicable for near-surface only.	Suitable, but may be cost prohibitive for preliminary screening of a large area.	Suitable; generally a preferred option in jurisdictions using this test.
Encased Borehole Tests	Smaller point measurement (0.5 to 2.0 ft diameter)	Primarily vertical direction, but with some horizontal direction.	May be more applicable for deeper tests than similar methods.	Suitable, but may be cost prohibitive for preliminary screening of a large area.	Generally suitable.
Large-scale Pilot Infiltration Test (PIT)	Extensive measurement (~100 sq-ft)	Dimensionality resembles to lateral proportions expected in typical infiltration BMPs; correction still needed.	Applicable for near-surface only.	Generally cost prohibitive for preliminary screening of a large area.	Suitable; generally a preferred option in jurisdictions using this test.

Test	Spatial Scale	Dimensionality	Vertical Strata	Suitable for Planning-Level Screening	Suitable for Design
Smaller-scale Pilot Infiltration Test (PIT)	Smaller-scale extensive measurement (~20 to 32 sq-ft)	Dimensionality resembles proportions of vertical to lateral expected in typical small-scale BMPs; requires greater correction than large-scale PIT.	Applicable for near-surface only.	Suitable, but may be cost prohibitive for preliminary screening of a large area.	Suitable; generally preferred in jurisdictions using this test.
Well Permeameter Method (USBR 7300-89)	Point measurement (3 to 8 inch diameter bore)	Primarily lateral infiltration; correction required.	Subsurface strata.	Generally suitable; reliability of this test can be improved by obtaining a continuous core where tests are conducted.	May be appropriate in areas of proposed cut where other tests are not possible; ideally should be confirmed with a more direct measurement following excavation.
Borehole Percolation Tests (various methods)	Point measurement (6 to 12 inch diameter bore)	Primarily lateral infiltration; correction required.	Can be conducted from shallow sub-surface to deeper subsurface; most applicable for deeper subsurface.	Generally suitable; reliability of this test can be improved by obtaining a continuous core where tests are conducted.	May be appropriate in areas of proposed cut where other tests are not possible; ideally should be confirmed with a more direct measurement following excavation.
Tube Permeameter Test	Point measurement (1.5 to 3 inch diameter bore)	Vertical only; conducted within tube in laboratory.	Samples can be collected from any strata of a boring; most applicable for deep subsurface investigations where in-situ tests are not possible.	Limited reliability, should be used only when other methods are not feasible.	Not generally accepted.

5 Specific Considerations for Infiltration Testing

The following subsections are intended to address specific topics that commonly arise in characterizing infiltration rates.

5.1 Hydraulic Conductivity versus Infiltration Rate versus Percolation Rate

A common misunderstanding is that the “percolation rate” obtained from a percolation test is equivalent to the “infiltration rate” obtained from tests such as a single or double ring infiltrometer test which is equivalent the “saturated hydraulic conductivity”. In fact, these terms have different meanings. Saturated hydraulic conductivity (Ksat) is an intrinsic property of a specific soil sample under a given degree of compaction. It is a coefficient in Darcy’s equation (Darcy 1856) that characterizes the flux of water that will occur under a given gradient. The measurement of Ksat in a laboratory test is typically referred to as “permeability”, which is a function of the density, structure, stratification, fines, and discontinuities of a given sample under given controlled conditions. In contrast, infiltration rate is an empirical observation of the rate of flux of water into a given soil structure under long term ponding conditions. Similarly to permeability, infiltration rate can be limited by a number of factors including the layering of soil, density, discontinuities, and initial moisture content. These factors control how quickly water can move through a soil. However, infiltration rate can also be influenced by mounding of groundwater, and the rate at which water dissipates horizontally below a BMP – both of which describe the “capacity” of the “infiltration receptor” to accept this water over an extended period. For this reason, an infiltration test should ideally be conducted for a relatively long duration resembling a series of storm events so that the capacity of the infiltration receptor is evaluated as well as the rate at which water can enter the system. Infiltration rates are generally tested with larger diameter holes, pits, or apparatuses intended to enforce a primarily vertical direction of flux. Permeability can be considered to be synonymous with infiltration rate when the infiltration rate is primarily vertical below the BMP and the capacity of the infiltration receptor is not a limiting factor.

In contrast to Ksat, permeability, and infiltration rate, percolation is an empirical observation of the flux of water into a certain soil structure, primarily in the lateral direction. Percolation is tested with small diameter holes, and it mostly a lateral phenomenon. The direct measurement yielded by a percolation test tends to overestimate the infiltration rate, except perhaps in cases in which a BMP has similar dimensionality to the borehole, such as a dry well. Adjustment of percolation rates may be made to an infiltration rate using a technique such as the Porchet Method.

5.2 Cut and Fill Conditions

Where the proposed infiltration BMP is to be located in a cut condition, the infiltration surface level at the bottom of the BMP might be far below the existing grade. For example, if the infiltration surface of a proposed BMP is to be located at an elevation that is currently beneath 15 feet of planned cut, how can the proposed infiltration surface be tested to establish a design infiltration rate prior to beginning excavation? The question can be addressed in two ways: First, one of the deeper subsurface tests described above can be used to provide a planning-level screening of potential rates at the elevation of the proposed infiltrating surface. These tests can be conducted at depths exceeding 100 feet, therefore are applicable in most cut conditions. Second, the project can commit to further testing using more reliable methods following bulk excavation to refine or adjust infiltration rates, and/or apply higher factors of safety to borehole methods to account for the inherent uncertainty in these measurements and conversions.

If the bottom of a BMP (infiltration surface) is proposed to be located in a fill location, the infiltration surface may not exist prior to grading. How then can the infiltration rate be determined? For example, if a proposed infiltration BMP is to be located with its bottom elevation in 10 feet of fill, how could one reasonably establish an infiltration rate prior to the fill being placed? Because of uncertainty in material properties as well as concerns regarding geotechnical issues, it is common for guidance manuals to prohibit infiltration into fill. However, if the design process allows for a more detailed understanding of fill properties (potentially through a phased approach, discussed below) and includes consideration of potential geotechnical design impacts, it may be possible to identify locations on a project in which infiltration into fill could be safe and effective.

There are two types of fills – those that are engineered or documented, and those that are undocumented. Undocumented fills are fills placed without engineering controls or construction quality assurance and are subject to great uncertainty. On the other hand, engineered fill properties can be very well understood, as they are generally placed using construction quality assurance procedures and may have criteria for grain size and fines content. However, for these types fills, infiltration rates may still be quite uncertain due to layering and heterogeneities introduced as part of construction that cannot be precisely controlled.

Where possible, infiltration BMPs should be designed such that their infiltrating surface extends into native soils. Additionally, for shallow fill depths, fill material can be selectively graded (i.e., high permeability granular material placed below proposed BMPs) to provide reliable infiltration properties until the infiltrating water reaches native soils. However, in some cases, due to considerable fill depth, the extension of the BMP down to natural soil and/or selective grading of fill material may prove infeasible. In additional, fill material will result in some compaction of now buried native soils potentially reducing their ability to infiltrate. In these cases, because of the uncertainty of fill parameters as described above as well as potential compaction of the native soils, an infiltration BMP may not be feasible.

However, if the source of fill material is defined and this material is known to be of a granular nature and that the native soils below is permeable and will not be highly compacted, infiltration through compacted fill materials may still be feasible. In this case, a project phasing approach could be used including the following general steps, (1) collect samples from areas expected to be used as borrow sites for fill activities, (2) remold samples to approximately the proposed degree of compaction and measure the Ksat of remolded samples using laboratory methods, (3) if infiltration rates appear adequate for infiltration, then apply an appropriate factor of safety and use the initial rates for preliminary design, (4) following placement of fill, conduct in-situ testing to refine design infiltration rates and adjust the design as needed; the infiltration rate of native soil below the fill should also be tested at this time to determine if compaction as a result of fill placement has significantly reduced its infiltration rate. The project geotechnical engineer should be involved in decision making whenever infiltration is proposed in the vicinity of engineered fill structures so that potential impacts of infiltration on the strength and stability of fills and pavement structures can be evaluated.

5.3 Effects of Direct and Incidental Compaction

It is widely recognized that compaction of soil has a major influence on infiltration rates (Pitt et al. 2008). However, direct (intentional) compaction is an essential aspect of roadway construction, and indirect compaction (such as by movement of machinery, placement of fill, stockpiling of materials, and foot traffic) can be difficult to avoid in some parts of the project site. Infiltration testing strategies should attempt to measure soils at a degree of compaction that resembles anticipated post-construction conditions.

Ideally, infiltration systems should be located outside of areas where direct compaction will be required and should be staked off to minimize incidental compaction from vehicles and stockpiling. For these conditions, no adjustment of test results is needed.

However, in some cases, infiltration BMPs will be constructed in areas to be compacted. For these areas, it may be appropriate to include field compaction tests or prepare laboratory samples and conducting infiltration testing to approximate the degree of compaction that will occur in post-construction conditions. Alternatively, testing could be conducted on undisturbed soil, and an additional factor of safety could be applied to account for anticipated infiltration after compaction. To develop a factor of safety associated with incidental compaction, samples could compacted to various degrees of compaction, their hydraulic conductivity measured, and a “response curve” developed to relate the degree of compaction to the hydraulic conductivity of the material.

5.4 Temperature Effects on Infiltration Rate

The rate of infiltration through soil is affected by the viscosity of water, which in turn is affected by the temperature of water. As such, infiltration rate is strongly dependent on the temperature of the infiltrating water (Cedergren, 1997). For example, Emerson (2008) found that wintertime infiltration rates below a BMP in Pennsylvania were approximately half their peak summertime rates. As such, it is important to consider the effects of temperature when planning tests and interpreting results.

If possible, testing should be conducted at a temperature that approximates the typical runoff temperatures for the site during the times when rainfall occurs. If this is not possible, then the results of infiltration tests should be adjusted to account for the difference between the temperature at the time of testing and the typical temperature of runoff when rainfall occurs. The measured infiltration can be adjusted by the ratio of the viscosity at the test temperature versus the typical temperature when rainfall occurs (Cedergren, 1997), per the following formula:

$$K_{Typical} = K_{Test} \times \left(\frac{\mu_{Test}}{\mu_{Typical}} \right)$$

Where:

$K_{Typical}$ = the typical infiltration rate expected at typical temperatures when rainfall occurs

K_{Test} = the infiltration rate measured or estimated under the conditions of the test

$\mu_{Typical}$ = the viscosity of water at the typical temperature expected when rainfall occurs

μ_{Test} = the viscosity of water at the temperature at which the test was conducted

5.5 Number of Infiltration Tests Needed

The heterogeneity inherent in soils implies that all but the smallest proposed infiltration facilities would benefit from infiltration tests in multiple locations. Indeed, several of the jurisdictions surveyed in this study provide requirements for the number of infiltration tests they require. The number of infiltration tests specified varies considerably by jurisdiction and is generally a matter that is left to the discretion of the designer and plan reviewer. For example, Orange County (2011) and Portland (2008) have adopted the following requirements for land development:

- A total of 2 infiltration tests for every 10,000 square feet of lot area available for new or redevelopment (minimum 2 tests per priority project). An additional test for every 10,000 square feet of lot area available for new or redevelopment.
- At least one test for any potential street facility.
- One test for every 100 lineal feet of infiltration facility.
- In general no more than 5 valid tests are required per development, unless more tests would be valuable or necessary *(at the discretion of the qualified professional assessing the site, as well as the reviewing agency).*
- Infiltration testing should be conducted at each proposed facility.
- Testing at multiple strata is recommended.

These types of criteria are typical of municipal guidance, particularly where there is an emphasis on rigorously identifying infiltration opportunities as well as establishing design infiltration rates, however specific numbers may vary regarding spacing and frequency of testing.

Jurisdictions that do not require a rigorous evaluation of opportunities may only require testing at the locations where BMPs are proposed. Western Washington (2012) requires that tests be conducted at each facility, and at each unique strata of soil, but does not specify a minimum number. However, their guidance allows a lower factor of safety to be used if a greater number of tests are conducted. This incentive is also provided by Orange County’s guidance.

Caltrans (2003) evaluated selected sites by conducting a minimum of four borehole percolation tests per facility. Their report recommended conducting additional tests at some locations to improve confidence in estimates, specifically where variability in test results was greater.

This white paper has not attempted to provide general recommendations regarding the number of tests needed. As a general rule, more tests are needed for sites with higher variability in soil properties and situations in which a higher degree of certainty is desired. The number of samples should be at the discretion of qualified professional assessing the site, as well as the reviewing agency, based on the number of samples needed to characterize the site for its intended use.

6 Selecting a Safety Factor

Should I use a factor of safety for design infiltration rate?

Monitoring of actual facility performance has shown that the full-scale infiltration rate can be much lower than the rate measured by small-scale testing (King County Department of Natural Resources and Parks, 2009). Factors such as soil variability and groundwater mounding may be responsible for much of this difference. Additionally, the infiltration rate of BMPs naturally declines between maintenance cycles as the BMP surface becomes occluded and particulates accumulate in the infiltrative layer. Gulliver et al. (2010) provide the following summary:

> *In the past, infiltration structures have been shown to have a relatively short lifespan. Over 50 percent of infiltration systems either partially or completely failed within the first 5 years of operation*

(U.S. EPA. 1999a). In a Maryland study on infiltration trenches (Lindsey et al. 1991), 53 percent were not operating as designed, 36 percent were clogged, and 22 percent showed reduced filtration. In a study of 12 infiltration basins (Galli 1992), none of which had built-in pretreatment systems, all had failed within the first 2 years of operation.

Given the known potential for infiltration BMPs to fail over time, an appropriate factor of safety applied to infiltration testing results is strongly recommended.

However, under the evolving regulatory context that increasingly requires infiltration to be used where feasible, the concern has been raised that an "artificially" high factor of safety could be inappropriately used by project proponents to demonstrate that infiltration is infeasible where it actually may be feasible.

It is recognized that there are competing objectives in the selection of a factor of safety. There is an initial economic incentive to select a lower factor of safety to yield smaller BMP designs. A low factor of safety also allows a broader range of systems to be considered "feasible" in marginal conditions. However, there are both economic and environmental incentives for the use of an appropriate factor of safety to prevent premature failure and substandard performance. The use of an artificially low factor of safety to demonstrate feasibility in the design process is shortsighted in that it does not consider the long-term feasibility of the system. For these reasons, we recommend that careful consideration be given to the selection of a factor of safety.

What factors should be considered in selecting and applying a factor of safety?

Local jurisdictions generally take the approach of either prescribing factors of safety that must be used or allowing for the discretion of the engineer and plan reviewer. While this white paper is not intended to supplant local regulations or replace good professional judgment, this section presents a recommended thought process for selecting a safety factor. This method was adapted from technical guidance prepared in Orange County and Ventura County, California. This method considers factor of safety to be a function of:

- Site suitability considerations, and
- Design-related considerations.

These factors and the method for using them to compute a safety factor are discussed below. Importantly, this method encourages rigorous site investigation, good pretreatment, and commitments to routine maintenance to provide technically-sound justification for using a lower factor of safety.

When selecting a factor of safety, attention should also be given to factors of safety that may be implicit in other aspects of the design such as the precipitation event used, the runoff coefficient of the tributary area, and other factors, as factors of safety can have compounding effects. Additionally, regenerative processes such as the ability of deeper rooted plants to maintain infiltration pathways should be considered. If other design factors include an implicit or explicit factor of safety and/or if significant regenerative processes are provided in the design, then a lower factor of safety may be warranted for infiltration rate. Using an overly conservative factor of safety may result in over-design and associated excessive costs or a risk in rejecting what would be a suitable condition for infiltration.

6.1 Site Suitability Considerations for Selection of an Infiltration Factor of Safety

Considerations related to site suitability include:

- Soil assessment methods – the site assessment extent (e.g., number of borings, test pits, etc.) and the measurement method used to estimate the short-term infiltration rate.
- Predominant soil texture/percent fines – soil texture and the percent of fines can influence the potential for clogging. Finer grained soils may be more susceptible to clogging.
- Site soil variability – site with spatially heterogeneous soils (vertically or horizontally) as determined from site investigations are more difficult to estimate average properties for resulting in a higher level of uncertainty associated with initial estimates.
- Depth to seasonal high groundwater/impervious layer – groundwater mounding may become an issue during excessively wet conditions where shallow aquifers or shallow clay lenses are present.

Table 3. Suitability Assessment Related Considerations for Infiltration Facility Safety Factors

Consideration	High Concern – 3 points	Medium Concern – 2 points	Low Concern – 1 point
Assessment methods (see explanation below)	Use of soil survey maps or simple texture analysis to estimate short-term infiltration rates Use of well permeameter or borehole methods without accompanying continuous boring log Relatively sparse testing with direct infiltration methods	Use of well permeameter or borehole methods with accompanying continuous boring log Direct measurement of infiltration area with localized infiltration measurement methods (e.g., infiltrometer) Moderate spatial resolution	Direct measurement with localized (i.e., small-scale) infiltration testing methods at relatively high resolution[1] or Use of extensive test pit infiltration measurement methods[2]
Texture Class	Silty and clayey soils with significant fines	Loamy soils	Granular to slightly loamy soils
Site soil variability	Highly variable soils indicated from site assessment, or Unknown variability	Soil borings/test pits indicate moderately homogeneous soils	Soil borings/test pits indicate relatively homogeneous soils
Depth to groundwater/ impervious layer	<5 ft below facility bottom	5-15 ft below facility bottom	>15 below facility bottom

1 - Localized (i.e., small scale) testing refers to methods such as the double-ring infiltrometer and borehole tests)

2 - Extensive infiltration testing refers to methods that include excavating a significant portion of the proposed infiltration area, filling the excavation with water, and monitoring drawdown. The excavation should be to the depth of the proposed infiltration surface and ideally be at least 30 to 100 square feet.

6.2 Design-Related Considerations for Selection of an Infiltration Factor of Safety

Design-related considerations include:

- Expected influent sediment loads and the level of pretreatment – Sediment loading to the infiltration system is a major factor in the rate at which infiltration rates decline and the potential for failure of the facility increases. For areas with expected sediment in runoff, well designed pretreatment should be included to reduce the probability of clogging from high sediment loading. Infiltration facilities in high sediment loading potential areas should be designed with a higher factor of safety. Infiltration facilities designed to capture runoff from relatively clean surfaces such as rooftops are likely to see low sediment loads and therefore may be designed with lower safety factors. In particular, the amount of landscaped area and its vegetation coverage characteristics tributary to an infiltration facility should be considered. For example in arid areas with more soils exposed, open areas draining to infiltration systems may contribute excessive sediments. Also to be considered is whether sanding is employed for winter traction and the type of truck traffic and materials being transported, both of which can contribute to sediment loading.
- Compaction during construction – Proper construction oversight is needed during construction to ensure that the bottoms of infiltration facility are not impacted by significant incidental compaction. Facilities that use proper construction practices and oversight need less restrictive safety factors.
- Redundancy/resiliency – Does the design include provisions that would allow the system to continue to operate adequately if conditions are different than design? For example, is there an elevated underdrain system to provide a relief for extended surface ponding should underlying infiltration rates be less than designed for? Can the VRA be designed to allow for maintenance to restore lost infiltration capacity? Are plants with deep/active root systems provided to help maintain infiltration rates and soil health?
- Storage depth - The storage depth of the VRA is the total equivalent water depth stored, after accounting for pore spaces. VRAs with deeper storage depths tend to have a higher sediment loading per unit area compared shallower VRAs, which may lead to greater clogging potential. They also may experience more significant issues with extended drain times if clogging does occur.

In addition to these factors, the influence of dry weather flows should be considered. For intermittent dry weather flows (say periodic irrigation return flows) where algae or biofilm growth would not be an issue, then infiltration of these flows are typically acceptable, except where they would create a nuisance or potentially impact groundwater quality. For continuous dry weather flow infiltration, one would have to consider that in design and consider potential clogging associated with biofilm.

Table 4 describes how to evaluate a given project in these areas.

Table 4. Design-Related Considerations for Infiltration Facility Safety Factors

Consideration	High Concern – 3 points	Medium Concern – 2 points	Low Concern – one point
Level of pretreatment/ expected influent sediment loads	Limited pretreatment using gross solids removal devices only, such as hydrodynamic separators, racks and screens AND tributary area includes landscaped areas, steep slopes, high traffic areas, road sanding, or any other areas expected to produce high sediment, trash, or debris loads.	Good pretreatment with BMPs that mitigate coarse sediments such as vegetated swales AND influent sediment loads from the tributary area are expected to be moderate (e.g., low traffic, mild slopes, stabilized pervious areas, etc.).	Excellent pretreatment with BMPs that mitigate fine sediments such as bioretention or media filtration OR sedimentation or facility only treats runoff from relatively clean surfaces, such as rooftops/non-sanded road surfaces.

Table 4. Design-Related Considerations for Infiltration Facility Safety Factors

Consideration	High Concern – 3 points	Medium Concern – 2 points	Low Concern – one point
Compaction during construction	Construction of facility on a compacted site or increased probability of unintended/ indirect compaction.	Medium probability of unintended/ indirect compaction.	Equipment traffic is effectively restricted from infiltration areas during construction and there is low probability of unintended/ indirect compaction.
Redundancy/ resiliency	No “backup” system is provided; the system design does not allow infiltration rates to be restored relatively easily with maintenance.	The system has a backup pathway for treated water to discharge if clogging occurs or infiltration rates can be restored via maintenance.	The system has a backup pathway for treated water to discharge if clogging occurs and infiltration rates can be relatively easily restored via maintenance.
Effective Storage Depth of VRA	Relatively deep profile (>4 feet)	Moderate profile (1 to 4 feet)	Shallow profile (< 1 ft)

6.3 Determining Factor of Safety

The following procedure can be used to estimate an appropriate factor of safety to be applied to the infiltration testing results. When assigning a factor of safety, care should be taken to understand what other factors of safety are implicit in other aspects of the design to avoid incorporating compounding factors of safety that may result in significant over-design.

1. For each consideration shown above, determine whether the consideration is a high, medium, or low concern.
2. For all high concerns in Table 3, assign a factor value of 3, for medium concerns, assign a factor value of 2, and for low concerns assign a factor value of 1.
3. Multiply each of the factors in Table 3 by 0.25 and then add them together. This should yield a number between 1 and 3.
4. For all high concerns in Table 4, assign a factor value of 3, for medium concerns, assign a factor value of 2, and for low concerns assign a factor value of 1.
5. Multiply each of the factors in Table 4 by 0.5 and then add them together. This should yield a number between 1 and 3.
6. Multiply the two safety factors together to get the final combined safety factor. If the combined safety factor is less than 2, then 2 should be used as the safety factor.
7. Divide the tested infiltration rate by the combined safety factor to obtain the adjusted design infiltration rate for use in sizing the infiltration facility.

Note: The minimum combined adjustment factor should not be less than 2.0 and the maximum combined adjustment factor should not exceed 9.0.

Worksheet 1 provides a form for documenting this method.

6.4 Implications of a Factor of Safety in BMP Feasibility and Design

The above method will provide safety factors in the range of 2 to 9. From a simplified practical perspective, this means that the size of the facility will need to increase in area from 2 to 9 times relative to that which might be used without a safety factor. Clearly, numbers toward the upper end of this range will make all but the best locations prohibitive in land area and cost.

In order to make BMPs more feasible and cost effective, steps should be taken to plan and execute the implementation of infiltration BMPs in a way that will reduce the safety factors needed for those projects. A commitment to thorough site investigation, use of effective pretreatment controls, good construction practices, and restoration of the infiltration rates of soils that are damaged by prior compaction should lower the safety factor that should be applied, to help improve the long term reliability of the system and reduce BMP construction cost. While these practices decrease the recommended safety factor, they do not totally mitigate the need to apply a factor of safety. The minimum recommended safety factor of 2.0 is intended to account for the remaining uncertainty and long-term deterioration that cannot be technically mitigated.

For projects being designed under a regulatory mandate to conduct a rigorous infiltration feasibility screening and to select infiltration BMPs where feasible, it may be necessary to put an upper cap on the factor of safety that may be used as part of infeasibility screening. For example, in Orange County (CA), a factor of safety of 2.0 must be used for infiltration feasibility screening such that an artificially high factor of safety cannot be used to inappropriately rule out infiltration. If the site passes the feasibility analysis at a factor of safety of 2.0, then infiltration must investigated, but a higher factor of safety may be selected at the discretion of the design engineer. A similar approach may be useful for DOTs under similar regulatory conditions.

Worksheet 1: Factor of Safety and Design Infiltration Rate Worksheet

Factor Category		Factor Description	Assigned Weight (w)	Factor Value (v)	Product (p) $p = w \times v$
A	Suitability Assessment	Soil assessment methods	0.25		
		Predominant soil texture	0.25		
		Site soil variability	0.25		
		Depth to groundwater / impervious layer	0.25		
		Suitability Assessment Safety Factor, $S_A = \Sigma p$			
B	Design	Level of pretreatment/ expected sediment loads	0.25		
		Redundancy/resiliency	0.25		
		Compaction during construction	0.25		
		Design infiltration depth	0.25		
		Design Safety Factor, $S_B = \Sigma p$			
Combined Safety Factor, $S_{total} = S_A \times S_B$					
Observed Infiltration Rate, inch/hr, $K_{observed}$ (corrected for test-specific bias)[1]					
Design Infiltration Rate, in/hr, $K_{design} = K_{observed} / S_{total}$					

Supporting Data

Briefly describe infiltration test and provide reference to test forms:

7 References

ASTM International. (2009). ASTM Standard D3385-09. Retrieved from http://www.astm.org/Standards/D3385.htm.

Caltrans (1986). Method for Determining the Percolation Rate of Soil Using a 6-inch-diameter Test Hole. California Test 750. http://www.dot.ca.gov/hq/esc/sdsee/wwe/documents/Test_750.pdf.

Caltrans (June 2003). Infiltration Basin Site Selection Study, Volume I. Report No. CTSW-RT-03-025. http://www.dot.ca.gov/hq/env/stormwater/special/newsetup/_pdfs/new_technology/CTSW-RT-03-025/IFB_Final_Report.pdf.

Cedergren, H.R. (1997). *Seepage, Drainage, and Flow Nets.* John Wiley & Sons, 496 p.

Colwell, S. and T. Tackett (undated). Ballard Roadside Rain Gardens, Phase 1 – Lessons Learned. http://water.epa.gov/infrastructure/greeninfrastructure/upload/gi_ballardproject.pdf.

Orange County (2011). Technical Guidance Document for the Preparation of Conceptual/Preliminary and/or Project Water Quality Management Plans (WQMPs). http://www.ocwatersheds.com/DocmgmtInternet/Download.aspx?id=638.

County of San Bernardino (August 1992). Suitability of Lots and Soils for Use of Leachlines or Seepage Pits, Soil Percolation (PERC) Test Report Standards, On-Site Waste Water Disposal System.

Darcy, H. (1856). Les fontaines publiques de la Ville de Dijon (The public fountains of the City of Dijon). Trans. Patricia Bobeck. Dalmont, Paris, France. (Kendall/Hunt, 2004) 506 p.

Emerson, C.H. (May 2008). *Evaluation of Infiltration Practices as a Means to Control Stormwater Runoff.* Doctoral dissertation, Villanova University.

Food and Agriculture Organization of the United Nations (2007). Guidelines and computer programs for the planning and design of land drainage systems, (W.H. van der Molen, J. Martínez Beltrán, and W.J. Ochs, eds.) Irrigation and Drainage Paper 62, Annex 3 Field methods for measuring hydraulic conductivity. ftp://ftp.fao.org/docrep/fao/010/a0975e/a0975e01.pdf.

Federal HighwayAdministration (August 2001). Urban Drainage Design Manual. Hydraulic Engineering Circular No. 22, Second Edition Publication No. FHWA-NHI-01-021. U.S. Department of Transportation. http://isddc.dot.gov/OLPFiles/FHWA/010593.pdf.

Galli, J. (1992). Analysis of urban stormwater BMP performance and longevity in Prince George's County, Maryland. Metropolitan Washington Council of Governments, Washington, D.C.

Gulliver, J.S., A.J. Erickson, and P.T. Weiss (eds.). (2010). "Stormwater Treatment: Assessment and Maintenance." University of Minnesota, St. Anthony Falls Laboratory. Minneapolis, MN. http://stormwaterbook.safl.umn.edu/

Hazen, A. (1892). Some Physical Properties of Sands and Gravels, With Special Reference to their Use in Filtration. 24th Annual Rep., Massachusetts State Board of Health, Pub. Doc. No. 34, 539–556.

Hazen, A. (1911). Discussion of Dams on Sand Foundations' by A.C. Koenig. Trans. *Am. Soc. Civ. Eng.*, 73, 199–203.

Hinman, C. (2009). Bioretention Soil Mix Review and Recommendations for Western Washington. Puget Sound Partnership. http://www.psparchives.com/publications/our_work/stormwater/BSMResultsGuidelines%20Final.pdf.

King County Department of Natural Resources and Parks (2009). King County, Washington Surface Water Design Manual. Retrieved from http://your.kingcounty.gov/dnrp/library/water-and-land/stormwater/surface-water-design-manual/SWDM-2009.pdf.

Lindsey, G., L. Roberts, and W. Page (1991). Storm Water Management Infiltration. Maryland Department of the Environment, Sediment and Storm Water Administration.

Lunne, T., Robertson, P. & Powell, J. (1997). *Cone Penetration Testing in Geotechnical Practice*, E & FN Spon, London, 312 p.

Maryland Department of the Environment (2009). Maryland Stormwater Design Manual, Volumes I & II (Effective October 2000, Revised May 2009) http://www.mde.state.md.us/programs/Water/StormwaterManagementProgram/MarylandStormwaterDesignManual/Pages/programs/waterprograms/sedimentandstormwater/stormwater_design/index.aspx.

Massmann, J., C. Butchart, and S. Stolar, (2003). *Infiltration Characteristics, Performance, and Design of Stormwater Facilities*, Final Research Report, University of Washington, WSDOT, and in cooperation with FHWA. http://depts.washington.edu/trac/bulkdisk/pdf/483.1.pdf.

Massmann, J.W. and L. Johnson (July-August 2001) A set of exercises illustrating flow in porous media, Ground Water, Volume 34(4), 499–503.

New Jersey Department of Environmental Protection, Division of Watershed Management (2009). New Jersey Stormwater Best Management Practices Manual (Appendix E: Soils Testing). Retrieved from http://www.njstormwater.org/bmp_manual2.htm.

Philips C. and W. Kitch (2011). *A review of methods for characterization of site infiltration with design recommendations.* California State Polytechnic University-Pomona. http://www.csupomona.edu/~wakitch/arts/Philips_&_Kitch_2011.pdf.

Pitt, R., S. Chen, S. Clark, J. Lantrip, and C. Ong (January 2008). Compaction's Impacts on Urban Stormwater Infiltration, *J. Irr. and Drainage Eng*.

U.S. Department of the Interior, Bureau of Reclamation (1990). "Procedure for Performing Field Permeability Testing by the Well Permeameter Method (USBR 7300-89)" in Earth Manual, Part 2. Materials Engineering Branch Research and Laboratory Services Division, Denver, Colorado.

U.S. Department of the Interior, Bureau of Reclamation. (1993). Drainage Manual: A Water Resources Technical Publication. Retrieved from http://www.usbr.gov/pmts/wquality_land/DrainMan.pdf.

U.S. EPA (1980). Onsite Wastewater Treatment and Disposal Systems (EPA No. 625/1-80-012). Retrieved from nepis.epa.gov/Exe/ZyPURL.cgi?Dockey=300043XO.txt

U.S. EPA (1999). Preliminary data summary of urban storm water best management practices. EPA-821-R-99-012, U. S. Environmental Protection Agency, Washington, D.C.

Van Hoorn, J.W. (1979). Determining hydraulic conductivity with the inversed auger hole and infiltrometer methods. In J. Wesseling, ed. Proceedings of the International Drainage Workshop, ILRI Publication 25, Wageningen, The Netherlands, 150–154.

Washington State Department of Ecology (2012). Stormwater Management Manual for Western Washington - Volume 3: Hydrologic Analysis and Flow Control BMPs. Retrieved from https://fortress.wa.gov/ecy/publications/summarypages/1210030.html.

Wisconsin (2012). Storm Water Post-construction Technical Standards. Dates of individual sections vary. Accessed October 5, 2012. http://dnr.wi.gov/topic/stormwater/stormwater_manual.html.

Appendix D

Potential Impacts of Highway Stormwater Infiltration on Water Balance and Groundwater Quality in Roadway Environments (White Paper #2)

1 Introduction

Infiltration of stormwater from urban highways has many potential benefits, but also has the potential to result in environmental and infrastructure impacts associated with the volume of water infiltrated (i.e., water balance impacts) and the introduction and/or mobilization of pollutants into groundwater (i.e., groundwater quality impacts). Therefore, a thorough assessment of potential negative impacts of infiltration is recommended as part of evaluating the feasibility and desirability of employing infiltration techniques for a given project site and within watersheds in general. The purpose of this white paper is to provide guidance for identifying potential impacts related to these factors, particularly in the urban roadway environment, and provide recommendations for project planners and designers with respect to assessing and avoiding and/or mitigating these potential impacts. Two key factors should guide the user's interpretation and application of this material:

First, it is critical to balance the benefits of stormwater infiltration with its risks. This white paper is intended as resource for understanding a wide range of potential issues. However, the purpose of this white paper is not to discourage or introduce unnecessary barriers to stormwater infiltration where it makes sense. Not all of the issues identified would necessarily apply to a given site. In most cases, the groundwater-related risks associated with stormwater infiltration can be mitigated or avoided once they are identified and given careful consideration.

Additionally, it is important to consider the watershed-scale context of potential issues. Some of the potential issues identified in this white paper would be of limited concern for stormwater infiltration at isolated sites, but cumulative effects could lead to a significant issue with widespread application. Analogously, the most effective way to mitigate these issues and balance benefits of stormwater infiltration may be the watershed and regional planning scale. Indeed, we recommend that many of these factors should be considered, and appropriate studies done, as part of developing regulations that would mandate infiltration.

The remainder of this white paper is organized as follows:

- **Section 2** introduces and identifies potential impacts related to changes in the natural water balance.
- **Section 3** introduces and identifies potential impacts related to groundwater quality.
- **Section 4** discusses the factors that should be considered in in evaluating whether infiltration of roadway runoff is infeasible or undesirable from the perspective of site water balance or groundwater quality. This section also provides recommendations for classifying the relative risk that a given project poses and how impacts can be potentially avoided and/or mitigated as part of the project development process.
- **Section 5** provides guidance for consulting with local agencies, such as water suppliers and resource agencies, with respect to potential water balance or groundwater quality impacts associated with stormwater infiltration.
- **Section 6** provides a brief summary of this white paper.

This white paper is based on a synthesis of published literature, experiences of the research team, and selected stormwater guidance documents. This white paper is not intended to provide a comprehensive set of criteria for evaluating infiltration feasibility that are applicable in all cases. Project planning and design professionals should exercise appropriate judgment in considering potential water balance and groundwater quality impacts associated with infiltration of stormwater from highways.

2 Potential Impacts of Stormwater Infiltration on Water Balance

The "water balance" refers to the fate of precipitation that falls on a given area of land over a given period of time. The major components of the water balance include (1) direct runoff to surface waters, (2) evapotranspiration (ET), and (3) deeper infiltration, including water that recharges a groundwater aquifer and/or discharges as baseflow to stream channels (Gobel et al. 2004). A water balance can be computed at a wide range of scales, from the scale of a site or small subwatershed up to the scale of a major river basin or even continent. In the context of highway project development, the water balance at a site-scale or small watershed-scale is typically the most meaningful, as a project may have the greatest potential to cause negative impacts at this scale. A water balance can also be computed for a range of timescales, from very short (i.e., minutes, hours) to long term (i.e., decades, or longer). Analysis at the scale of days or weeks may be most appropriate for assessing acute impacts such as localized groundwater mounding, while analysis of long term averages (i.e., years, decades) may be most appropriate timescale for evaluating long term changes in watershed hydrologic regime, such as changes in baseflow or long term subsurface soil wetting, that may result in chronic or acute issues.

The "natural" or "undeveloped" water balance varies greatly by region, watershed conditions, and the scale of the system that is being considered. To illustrate this variability, annual fluxes in water balance components were compared between several case studies of mostly undeveloped watersheds throughout the United States (Table 1). While studies reported fluxes in different combinations, the variability in conditions is evident. In several studies, ET represents the largest component of the water balance, ranging from 30 to more than 90 percent. This trend is especially prevalent in warmer locations that receive less rainfall (e.g., California, Texas). While it was infrequently separated from baseflow in the studies reviewed, direct surface runoff is typically a minor element in natural water balances, particularly in the arid west. At a site level, water balance may differ substantially from regional averages as a function of soil properties, local surface geology and hydrogeology, vegetation properties, presence of impervious surfaces, relative magnitudes and patterns of potential ET and rainfall, and other factors.

Table 1. Evaluation of Variability in Undeveloped Water Balance by Region based on Selected Studies

Source:	**Location:**	**Annual Fluxes (as percent of precipitation)**:
Church et al. 1995	North Eastern US	Runoff and Baseflow = 55% ET and Recharge = 45%
Jefferson et. al. 2008	Northwest (Cascade Mountains); The two study watersheds adjoin each other in the upper McKenzie River watershed on the west side of the Oregon Cascades	Runoff and Baseflow = 70%; ET = 30%; Water Storage Change = 0%
Milly 1994	East of the Rocky Mountains	Runoff and Baseflow = 27%; ET = 73%; Water Storage Change = 0%
Mohseni & Stefan 2001	The Baptism River watershed in northern Minnesota. The watershed is heavily timbered with both deciduous and coniferous trees.	Runoff and Baseflow = 55% ET and Recharge = 45%
Mohseni & Stefan 2001	The Little Washita River watershed in Oklahoma. One third of the watershed is cultivated and the rest	Runoff and Baseflow = 7%

Source:	Location:	Annual Fluxes (as percent of precipitation):
	is either pasture or wooded pasture.	ET and Recharge = 93%
Najjar 1999	Susquehanna River Basin	Runoff and Baseflow = 49% ET = 51% Water Storage Change = 0%
Ng & Miller 1980	Southern California Chaparral (average of 2 years of monitoring)	South facing: Runoff and Baseflow = 3%; ET = 97%, Storage change negligible. North facing: Runoff and Baseflow = 9%, ET = 83% and storage change = 8%.
Rose 2009	South Eastern US: five-state study area (Georgia, South Carolina, North Carolina, Virginia, and Maryland)	Runoff and Baseflow = 37% ET and Recharge = 63%
Ward 1993	Texas	Runoff = 12.5 %; ET = 86 %; Recharge = 1.5%

Sanford and Selnick (2013) estimated the estimated long term fraction of precipitation lost to evapotranspiration at the county scale for the conterminous United States (Figure 1). This graphic illustrates the variability in the relative proportions of different fluxes in of the water balance. It also demonstrates that ET is at or above 40 percent of the long-term fraction of precipitation for the majority of the United States. Given that the post-development ET fraction is approximately proportional to the amount of vegetation remaining, the ET fraction could be reduced to less than 10 to 20 percent for moderate to dense development. With this change in ET, the potential to impact water balance is significant for both runoff and infiltration approaches.

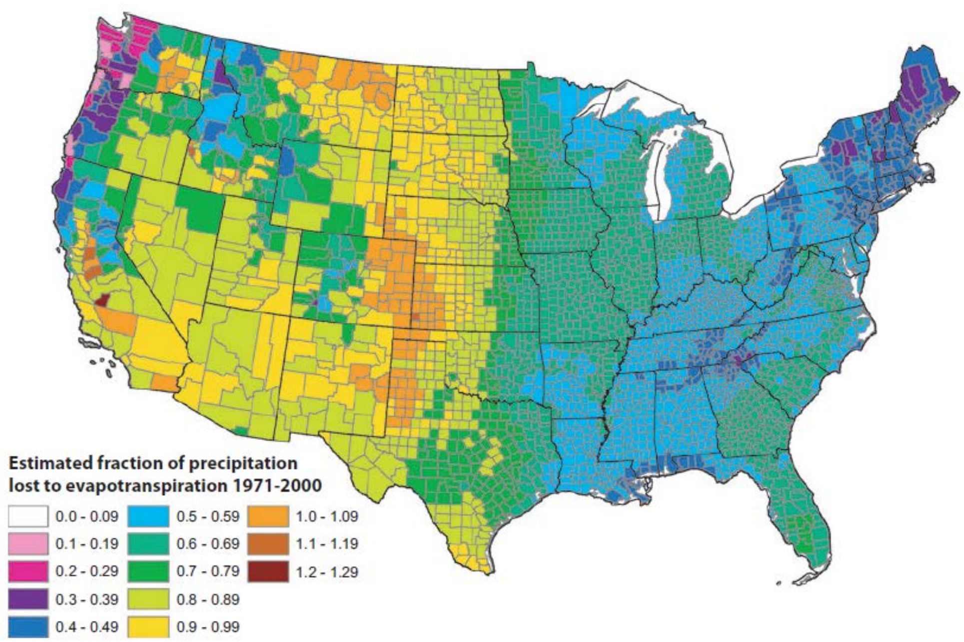

Figure 1. Estimated Long Term Fraction of Precipitation Lost to Evapotranspiration (Sanford and Selnick, 2013)

2.1 Potential Changes in the Water Balance

Project activities have the potential to alter the water balance of a site (and smaller watersheds) as a result of changes in land cover (i.e., addition of impervious surface, compaction of pervious areas) and/or as a result of the addition of stormwater controls (i.e., infiltration systems). Project changes together with other watershed developments may impact overall watershed characteristics and should be considered in concert with cumulative impact analyses.

In what ways can stormwater infiltration result in changes in the water balance?

How are roadways unique in their impacts on the water balance?

Many transportation improvements include the addition of impervious surfaces, either as a result of construction of a new roadway or the addition of lanes. In addition adjacent roadside unpaved right-of-way is compacted. An increase in impervious surface (and compacted soils) typically results in an increase in the surface runoff component of the water balance and tends to decrease the amount of water that enters the ground, resulting in reductions in deeper infiltration (groundwater recharge and/or baseflow). Where vegetation is removed and replaced by impervious surface, ET is also generally reduced due to a reduction in the amount of rainfall intercepted in plant leaves and in the upper soil layer that contains plant roots (i.e., the root zone) and mulches – these storage elements hold water during a rain storm and make it available for subsequent ET. While evaporation occurs from residual ponded water on impervious surfaces as well, the relative storage provided from ponding on impervious surfaces tends to be significantly less than the storage provided in the root zone below vegetated areas (Gobel et al. 2004) or even in unvegetated soils. The result is that project development, without stormwater controls, tends to result in an increase in surface runoff with corresponding decreases in both ET and deeper infiltration.

Stormwater regulations are increasingly emphasizing management approaches based on "mimicking pre-development hydrology". By "hydrology" the regulations typically narrow hydrology to what is really surface runoff hydrology (i.e. groundwater and ET components of hydrology are not addressed). This goal is generally accomplished by using volume reduction practices that rely on infiltration, ET, or consumptive uses to reduce the amount of runoff discharged directly to surface waters (Dietz 2007; U.S. EPA 2010). In most cases, compliance with these regulations is demonstrated based on the volume of direct surface runoff, without reference to or direct consideration of the other elements of the water balance or overall hydrology (SARWQCB 2009; WADOE 2012; U.S. EPA 2012c). Infiltration BMPs can be effective in mitigating increases in direct surface runoff volume and the corresponding reductions in the amount of water infiltrated. However infiltration BMPs may result in proportions of ET and deeper infiltration that are different than natural conditions. When surface runoff volume is held fixed between natural and proposed conditions and ET is reduced (as discussed above), an increase in deeper infiltration elements of the water balance (recharge and/or baseflow) must occur.

A study conducted in Recklinghausen, Germany demonstrated that an "average density" development without infiltration caused the area-averaged groundwater recharge to decrease from 221 mm per year (28 percent of rainfall) in the natural condition to 163 mm year (20 percent of rainfall) in the developed condition. When runoff from the developed impervious area was infiltrated, the area-averaged groundwater recharge nearly doubled from the proposed condition without controls (163 mm per year) to the proposed condition with controls (245 mm per year, 31 percent of rainfall), which also exceeded the natural condition recharge of 221 mm per year (Gobel et al. 2004). In semi-arid climates, natural recharge may be significantly less than this study, which could result in a more substantial change in groundwater recharge as a result of the use of infiltration BMPs. For example, Ng and Miller (1980) found that in

Southern California chaparral, ET made up 83 to 97 percent of the water balance, and deeper infiltration was less than 10 percent. This is consistent with the estimates made by Sanford and Selnick (2013) shown in Figure 1. In evaluations that we have conducted in Southern California, deeper infiltration has been estimated to increase by as much as three times over pre-project conditions when stormwater in infiltrated to meet water quality design requirements. Also, sites that develop with a greater impervious cover (such as roadways), tend to result in a greater reduction in ET and therefore a more substantial shift toward increased infiltration when project goals include matching pre-development volume to the post-project volume of direct surface discharge.

The proportional split between deeper infiltration and ET that occurs in a BMP is a function of the underlying infiltration rate of site soils, soil moisture retention properties, plant root depths, rainfall intensity, and facility design characteristics, specifically the footprint and the depth of the BMP (Clark et al. 2006). When shallow BMPs with larger surface areas are used, the level of ET tends to increase due to the additional retained moisture content in the top layer of soils in closer contact with the atmosphere (Strecker and Poresky, 2009). In contrast, when deeper BMPs with smaller footprints are used, or when BMPs do not contain amended soil and vegetation elements a greater portion of the water balance is associated with deeper infiltration; ET plays a more minor role.

Roadway environments, particularly in urban areas, represent a unique category of developed areas because of the typically high degree of “connectedness” of impervious area with hardened drainage systems and relatively small footprints that are typically available for infiltration BMPs. Directly connected impervious areas (DCIAs) have been shown to generate a significant amount of the total runoff from developed areas (Lee and Heaney 2003). The level of connectedness is important for water balance, as runoff from DCIAs, by definition, is not routed across pervious surfaces before being routed to the stormwater drainage system, which limits opportunities for ET and infiltration. Limited space in typical roadway environments also limits the amount of ET that may occur from BMPs. While relatively small footprints may be sufficient to achieve infiltration where soil conditions allow (i.e., infiltration rates are high, other constraints do not exist), ET is primarily driven by surface area, therefore the ET element of the water balance generally decreases as the BMP footprint decreases. As a result, where infiltration is used in the roadway environment to mimic pre-development surface runoff volumes, a substantial increase in deeper infiltration compared to natural conditions would be expected; this change may be particularly acute in semi-arid and arid areas where deeper infiltration tends to be a smaller element of the water balance in the natural condition and ET a larger element in the natural condition.

2.2 Potential Benefits and Impacts Associated with Increases in Deeper Infiltration

When considering potential impacts of stormwater infiltration on water balance, the most significant consideration is the increase in deeper infiltration that may occur as part of projects that are designed to mimic natural surface water discharge (as introduced in Section 2.1). An increase in deeper infiltration may have both positive and negative impacts. An increase in deeper infiltration (when done safely) may have the important positive benefit of augmenting groundwater supplies, improving the quantity and temperature of baseflow in streams, especially in areas where baseflows have already been depleted due to previous development and/or groundwater extraction, and/or improving the groundwater quality. However, there are also potential negative impact related to elevated groundwater tables and changes to dry weather hydrologic regimes in stream channels.

How do the changes in the water balance express themselves in terms of positive and negative impacts?

Potential benefits. In regions where aquifers are actively managed for water supply, an increase in infiltration may be desirable and even encouraged. For example, the Los Angeles (CA) region is underlain by a number of productive aquifers that are actively managed for water supply. Based on model estimates conducted by the Los Angeles and San Gabriel River Watershed Council (2010), if the first 0.75 inches of each rainstorm within the Los Angeles Region were captured and infiltrated into the regional aquifer system, the natural percolated volume of 194,000 acre-feet could be increased to 578,000 acre-feet. This addition could potentially supply enough additional water for 1.5 million people, which amounts to approximately $311 million in new water supply for a region that continually faces water supply challenges. The study identified a range of other benefits associated with infiltration in these conditions. In some areas including the Central Valley in California and the Ogallala aquifer in South Dakota, Nebraska, Wyoming, Colorado, Kansas, Oklahoma, New Mexico, and Texas, where groundwater elevations have been severely reduced, such augmentation over natural infiltration recharge rates may be extremely beneficial.

In regions where streams are perennial, urbanization has often been found to result in a reduction in baseflow and associated water quality issues during dry weather (for example, Spinello and Simmons 1992). Note that in some locations with wastewater discharges or significant irrigation return flows this is not the case. Implementing stormwater measures that increase deeper infiltration compared to natural conditions in selected locations may be an important part of regional strategies to augment baseflow and address dry weather water quality considerations. Where localized increases in infiltration volume do not cause negative impacts (such as those discussed below), this localized increase in infiltration may help contribute to regional improvements in watershed health by offsetting regional decreases in infiltration volume caused by historic urbanization.

Another potential positive impact of increased infiltration volume is the potential to improve groundwater quality through dilution of groundwater contaminants, provided that the stormwater itself does not result in water quality impacts (as discussed in Section 3). In some cases, dilution of contaminants is not desirable, as groundwater contamination plumes can be more efficiently addressed if they remain localized and concentrated. However, in some cases, dilution of groundwater contaminants is an acceptable management strategy that can be supported through practices that result in an increased infiltration component of the water balance (Fischer et al. 2003). Nightingale (1987) found that infiltration of stormwater in the Fresno, California, area was actually diluting nitrate levels in groundwater that were elevated due in large part to surrounding agricultural activities.

Potential negative consequences. Potential negative consequences of increases in deeper infiltration are generally most acute in areas where local subsurface conditions have limited ability to accept additional infiltrated volumes or where minor changes in hydrogeologic conditions would result in potential impacts.

On a subwatershed scale, an increase in deeper infiltration volume has the potential to change the local hydrogeologic regime, which can have significant adverse impacts on streams. For example, in the arid southwest, many channels are naturally ephemeral, meaning that they flow during and after storm events or for some period during the wet season, but are normally dry for much of the year. The riparian ecosystems in these areas are specifically adapted for these conditions. An increase in deeper infiltration in these areas has the potential to extend the duration of surface baseflows which can result in a "type change" of the stream from ephemeral to intermittent or perennial channels. This can result in colonization of the channel with different vegetative and terrestrial species, changes in hydrologic regime, and other changes that threaten the functions and values of a riparian area as well as the species present therein. As a specific example, environmental clearance documents for the Rancho Mission Viejo planned community in Orange County, California (California FEIR No 589) identified potential adverse impacts to the endangered Arroyo Toad (which prefers dry wash habitats) if the baseflow regime of ephemeral intermittent creeks within the project were significantly altered as a result of development. This project included carefully planned stormwater management features to manage the overall water balance of the site to avoid significant reductions or increases in deeper infiltration volumes so that natural stream baseflows are maintained.

On a more localized basis, the volume of infiltrated water can result in groundwater “mounding” – localized increases in the elevation of the groundwater table below infiltration BMPs. The potential for groundwater mounding is increased where there is shallow groundwater, a shallow restricting layer (bedrock, clay lens), poor soils for infiltration, a relatively shallow groundwater gradient (i.e., slope of the groundwater surface), and/or the BMP footprint is relative large (Carleton 2010). Elevated groundwater levels can lead to a number of severe problems, including flooding and damage to structures and utilities through buoyancy and moisture intrusion, increase in inflow and infiltration into municipal sanitary sewer systems, and flow of water through existing utility trenches, including sewers, potentially leading to formation of sinkholes (Gobel et al. 2004). Groundwater mounding also has the potential to impact groundwater quality as a result of a reduction in the separation between BMPs and the groundwater table and/or mobilization of contaminants as a result of submergence of contaminated soils, as discussed in Section 3. Infiltration may also increase the risk of geotechnical hazards such as subsidence, liquefaction, slope instabilities, foundation and subbase issues, and infrastructure damage associated with expansive clays (OCPW 2011; Oregon State University et al. 2006). White Paper No. 3 provides more information regarding potential geotechnical impacts and hazards related to stormwater infiltration.

Section 4 provides guidance for assessing the potential for water balance impacts recommended measures to help mitigate these potential impacts.

3 Potential Impacts of Stormwater Infiltration on Groundwater Quality

While infiltration of stormwater has the potential for improvements in surface water quality, it also has the potential for unintended consequences for groundwater quality (Clark et al. 2006). Infiltration of stormwater has the potential to impact groundwater quality as a result of an influx of pollutants contained in stormwater and/or mobilization of pollutants that are present in soils or groundwater. This section identifies the groundwater contaminants of concern that are typically encountered in runoff from urban roadways and discusses their fate and transport relative to potential impacts on groundwater quality (Section 3.1). It also discusses the potential for stormwater infiltration to mobilize and spread soil and groundwater contaminants (Section 3.2) and the potential for contaminant spills to impair groundwater quality (Section 3.3).

3.1 Sources, Fates, and Transport of Groundwater Contaminants of Concern in Roadway Runoff

Potential for stormwater infiltration to contaminate groundwater has been studied extensively (Weiss et al. 2008). Pitt et al. (1999) and Pitt and Clark (2007) have characterized the risk of groundwater contamination from infiltration of stormwater as a function of the following factors:

- Pollutant mobility – pollutants that are more mobile in the vadose zone (the unsaturated zone between the groundwater table and the ground surface) have a higher potential to contaminate groundwater than those which do not move through the vadose zone as readily.
- Pollutant abundance – pollutants that are highly abundant in terms of concentration and detection frequency in stormwater have higher potential to impact groundwater quality.
- Pollutant partitioning – pollutants that are present primarily in soluble fractions tend to have a higher potential to contaminate groundwater.

This section identifies the main categories of contaminants which may potentially pose a risk to groundwater quality as a result of stormwater infiltration.

What are the most common contaminants of concern for groundwater in roadway environments?

How are roadways unique in their impacts on groundwater quality?

In what ways can stormwater infiltration impact groundwater quality?

The fate and transport of individual groundwater contaminants varies significantly depending on the pollutant, characteristics of water and soil, treatment facility type, and other factors. Although the composition and concentration of pollutants found in stormwater runoff are highly site-specific, the following categories of pollutants have been frequently detected in urban stormwater runoff and may pose concerns for groundwater quality:

- Nutrients
- Pesticides
- Organic compounds
- Pathogenic microorganisms
- Heavy metals
- Salts

Note that particulates and particulate-bound pollutants are not included on this list because infiltration BMPs are generally highly effective at removing particulates from stormwater prior to stormwater reaching groundwater. With the exception of conditions with direct connections between surface water and groundwater (such as Karst topography, discussed in Section 4.7 or injection wells with little or no pretreatment), there is limited potential for particulates to migrate to groundwater and impair groundwater resources.

The sections below provide a discussion of the potential for groundwater contamination for the six primary pollutants categories listed above based on a synthesis of literature sources and evaluation of unit processes provided in stormwater BMPs. Table 2, at the end of this section, provides a summary of the pollutants of concern and the relative risk to groundwater quality that each is believed to possess.

3.1.1 Nutrients

The two nutrients with the most significant potential to contaminate groundwater are nitrogen and phosphorus. Nitrogen compounds, specifically nitrate, are the most commonly encountered nutrient contaminants in groundwater due to their application as fertilizers on developed landscaped areas and agricultural land uses and the deposition of vehicular exhaust on roadways and surrounding soils (Pitt et al. 1994, 1996); they also occur from the breakdown of organic debris. Naturally occurring sources of nitrogen, the atmosphere and soils, can also lead to groundwater contamination if environmental conditions are conducive. Compost amendments in shoulder treatments and/or stormwater controls can also be sources of nutrients at levels of concern if not appropriately specified and sourced.

Nitrogen compounds in stormwater may be removed through a number of processes, such as uptake by plants and microbes, microbially-mediated nitrification/denitrification, and volatilization, which are highly dependent on soil composition and hydrologic properties (Weiss et al. 2008). Nitrate is the nitrogen species of greatest concern for groundwater quality as it is highly mobile in the vadose zone and groundwater, and it is not readily converted to other nitrogen species, except where biological processes are active and cyclical changes in redox conditions occur (Pitt et al. 1994, 1996). The leaching of nitrogen into groundwater and soils is most common during cool, wet seasons because lower temperatures reduce the rates of denitrification, ammonia volatilization, microbial immobilization, and plant uptake. An accumulation of nitrate in groundwater may result in health risks associated with groundwater consumption and may contribute to detrimental nutrient loadings in surface waters that receive inputs from contaminated groundwater. Nitrite and ammonia are also mobile in soils, but tend to be present in relative low levels because in most cases, they undergo oxidation to nitrate relatively rapidly in the aerobic vadose zone. Fertilizer sources of nitrate can be controlled by limited and careful application of fertilizers.

Phosphorus is also of concern for groundwater contamination in some cases. Sources of phosphorus in the urban highway environment include detergents in gasoline, motor oil, fertilizer, bird droppings, animal remains, and compost amendments. The most common form of dissolved, mobile phosphorus present in stormwater, orthophosphate, can be removed from infiltrating water through precipitation or chemical adsorption onto soils (Weiss et al. 2008). As this is a sorption process, the relative phosphorus saturation of native soils can be used as an indicator of the level of potential removal of dissolved phosphorus that will result as water percolates through the soil. Soils that have been historically used for agricultural purposes may contain relatively high levels of phosphorus and may provide relatively little capacity to sorb additional phosphorus or may contribute to phosphorus loadings. Like nitrate, elevated dissolved phosphorus levels in groundwater may contribute to detrimental nutrient loadings in surface waters that receive groundwater discharges.

Overall, nutrients in highway runoff pose relatively limited potential to impact groundwater quality in most cases due to relatively low concentrations in urban stormwater in comparison to groundwater concentrations and groundwater quality objectives (Pitt et al. 1999). Risks are elevated when stormwater includes runoff from industrial and/or agricultural land uses that may periodically contain very high levels of nutrients or where aquifers or receiving waters have limited capacity for additional nutrients.

3.1.2 Pesticides

Pesticides contamination of groundwater, such as 2,4-D, lindane and chlordane, usually originate from weed, pest or fungus control in landscaped areas. Sources in the urban highway environment may include roadside maintenance of landscaping, medians, and other landscaped or managed areas. Concentrations of pesticides found in groundwater vary significantly based on pesticide usage (quantity and type), underlying soil texture, total organic carbon of the soil, contaminant persistence, and the depth to groundwater (Pitt et al. 1994, 1996, 1999). Pesticides with high solubility and low affinity for organic matter (low partitioning coefficients) tend to pose the greatest risk. Pesticides tend to have the highest mobility in coarse-grained or sandy soils without a hardpan layer, with low clay and organic matter content and high permeability.

Decomposition is possible in both soil media and water, but the time frame can range between days and years depending on the conditions and the specific compounds (Pitt et al. 1994, 1996). Studies conducted on the half-life of most pesticides are generally applicable to surface and near-surface conditions and do not generally account for the reduced microbial activity deeper in the vadose zone.

Where there is concern regarding pesticide contamination, risks can generally be mitigated by careful selection and application of pesticides. Pesticides with low solubility and high affinity for organic matter tend to pose limited risk of groundwater contamination. Where soils have limited organic matter, the use of amended media in an infiltration BMP has the potential to limit transport of pesticides.

3.1.3 Organic Compounds

Some organic compounds in groundwater are naturally occurring, such as those originating from decomposing animal wastes, leaf litter, vegetation, and organisms in the soil, but many are man-made and originate from sources like landfills, sewage systems, agricultural runoff, and urban stormwater runoff, including highway runoff (Pitt et al. 1999). The most common organics detected in groundwater are phthalate esters, such as bis(2-ethylhexyl)phthalate, and phenolic compounds, such as phenol and 2,4-dimethyl phenol. Volatile organic compounds, such as benzene, chloroform, methylene chloride, trichloroethylene (TCE), tetrachloroethylene, toluene, and xylene, are also detected in groundwater. Finally, polycyclic aromatic hydrocarbons (PAHs, a class of semi-volatile organic compounds), such as fluoranthene and pyrene, are commonly detected in urban stormwater, particularly from roadway and parking lot runoff due to their production via combustion processes, and have been detected in groundwater (Pitt et al. 1999). Uncombusted petroleum products (oils, lubricants, etc.) are also sources of some types of PAHs. For example, testing in New Jersey demonstrated that groundwater receiving runoff from roadways and parking lots contained elevated concentrations of petroleum hydrocarbons, such as benzene and toluene, when compared to background groundwater levels (Fischer et al. 2003).

The majority of organic compounds can be removed via volatilization, sorption and degradation to levels that would not affect groundwater quality. Treatment processes found in organically-active soils (sorption, microbial degradation) generally trap hydrocarbons within the first few centimeters of soil and do not allow them to percolate far enough to contaminate groundwater (Weiss et al. 2008). PAHs, in particular, have been successfully removed from infiltrating stormwater through degradation by naturally developed microbial communities where there is a prolonged aerobic, sulfate reducing, and denitrifying environment (Miklas and Grabowiecki 2007). Volatilization is another pathway that has been observed to remove organics, but the rate of volatilization for many hydrocarbons decreases with lower temperatures by nature, resulting in higher detectable rates in colder months (Fischer et al. 2003). The presence of sandy soils and a high water table have been found to be correlated with greater prevalence of groundwater contamination by organic compounds (Pitt et al. 1994, 1996).

While roadways are a known source of organic compounds, particularly petroleum hydrocarbons and combustion byproducts, the risk posed by infiltration of organic compounds in stormwater is relatively low due to relatively low stormwater concentrations and relatively low mobility of most organic compounds in the vadose zone. There is an elevated level of contamination risk where specific sources of organic compounds are present, where there is a shallow depth to groundwater, where soils are sandy, and where subsurface injection/infiltration is used – each of these factors reduces the effectiveness of removal processes in the vadose zone that would protect groundwater quality (Pitt et al. 1994, 1996).

3.1.4 Pathogenic Microorganisms

Pathogens, including bacteria and viruses, are ubiquitous in urban stormwater runoff. They originate from anthropogenic sources, such as human waste, dog waste, and failing septic systems, as well as natural sources, such as bird and animal droppings. Pathogenic contamination of groundwater has been linked to stormwater infiltration: for example, pathogens, most commonly enteroviruses, have been found to occur more frequently in groundwater where there is a high groundwater table and near MS4 outfalls (Pitt et al. 1999). Contamination depends on a number of site-specific environmental factors and pathogen characteristics, but it is believed that infiltration through BMPs may create a pathway for groundwater contamination (Weiss et al. 2008).

The survival and persistence of pathogens in the vadose done and in groundwater is sensitive to a number of factors, including pH, interactions with soil microflora, moisture content, temperature, dissolved oxygen content, and concentrations of organic matter. Viral pathogens, specifically enteroviruses have a high likelihood for groundwater contamination if they are present in the stormwater runoff as they are highly mobile and are less sensitive to environmental factors than bacterial pathogens (Pitt et al. 1999). Bacterial pathogens are often removed through sedimentation or sorption to soils during percolation through the upper layer of soils due to their size. The pathogens that are successfully filtered and sorbed are inactivated and killed as soil dries and necessary survival factors are eliminated (Pitt et al. 1999). For pathogens reaching the groundwater table, the distribution and movement of pathogens in groundwater is controlled by convection, sorption, and dispersion in the liquid phase.

Pathogen contamination from stormwater infiltration, particularly by enteroviruses, may be of significant concern for groundwater quality in many urban roadway areas due to the potential for elevated loadings from human waste, pet waste, and garbage. Like other contaminants, the risk of pathogen contamination is elevated where soils are coarser and depth to groundwater is less. However, where groundwater is not used for human consumption or where groundwater is already disinfected prior to use, the material consequences of bacterial contamination may be limited. Additionally, for controlled access facilities, such as limited access highways and freeways, the source of human pathogens would be very limited, with associated reduced concern for groundwater contamination. When highways include runoff from adjacent areas, there is more potential for human pathogen sources.

3.1.5 Heavy Metals

Heavy metals, such as copper, lead, and zinc, are commonly found in stormwater runoff. Sources of heavy metals in urban runoff include automobile parts, building materials, exposed metal products, fertilizers, fungicides, atmospheric deposition from industrial and vehicle exhausts and other sources. In the roadway environment, specific sources included brake pads and treated woods (copper), tires (zinc), and galvanized guardrails (zinc). Leaded gasoline was historically a major source of lead and arsenic loading, however allowable levels of lead have been greatly reduced in gasoline in the United States and, in the last 20 years, lead is not commonly detected in runoff at levels of concern. Lead may still be a concern as a legacy pollutant in roadside soils in some case and/or if there are sensitive receptors that could be impacted by lead at low levels. While roadways are recognized to be an elevated source of metals, concentrations in urban runoff tend to be much less than drinking water standards, therefore risks of human health issues from stormwater infiltration are minor.

During the infiltration process, when infiltrated through soils, metals are typically removed via filtration of particulate-bound metals, adsorption of dissolved metals to soil particles, chemical precipitation, diffusion into solid particles, and biological uptake (Weiss et al. 2008), decreasing their likelihood of groundwater contamination. However, metals are of increased concern when infiltration facilities are located in rapidly-infiltrating inert materials, such as sand or gravels, or when infiltration occurs in close proximity to sites with elevated stormwater concentrations, such as industrial sites and maintenance yards. Among the heavy metals of potential concern to groundwater quality, zinc has the highest solubility in stormwater and tends to be the most mobile in the vadose zone (Pitt et al. 1999). Soils that have a high cation exchange capacity (CEC) and organic content tend to provide greater potential for removing heavy metals through sorption and other processes, however metals have been found to be well retained in systems without organic content, such as permeable pavements (Dierkes et al. undated). The use of organic materials in soil media for infiltration systems can improve the performance of removal of heavy metals.

3.1.6 Salts and Dissolved Minerals

Inorganic dissolved minerals, including chloride, sulfate, and sodium, have been detected in groundwater at concentrations of concern (Pitt et al. 1999). Sources of salts in groundwater include natural salts in soils, addition of fertilizers to agricultural fields, evaporation of irrigation water (leaving salts behind), addition of salts to waste streams through consumptive uses specifically water softening, and application of salts to roads and other surfaces during cold weather. Among stormwater-related sources, road salting is the most significant source of salt loading and has been found to significantly affect chloride content and salinity of groundwater (Pitt et al. 1994, 1996). The potential for long-term accumulation of salts in groundwater is a function of the nature of the aquifer and the loading of saline water versus fresh water – aquifers that exist in closed or relatively closed basins are more susceptible to long term increases in salts. Reclaimed water (treated wastewater) is also a source of salts and may be a consideration in areas where reclaimed water is used to irrigate roadside vegetation or other landscaping tributary to roadway systems.

Roadway runoff in cold climates, where salt is used, has a high potential for contaminating groundwater because salts are water soluble, non-filterable, not readily sorbed to solids, and can leach into groundwater as infiltration occurs (Weiss et al. 2008; Pitt et al. 1994, 1996). Because conventional treatment methods are not effective at removing salts, the potential for salt contamination of groundwater may be an overriding factor in determining the feasibility of stormwater infiltration. In areas where salts are not applied to roadways, the infiltration of stormwater may improve groundwater quality through dilution, as stormwater typically has relative low dissolved mineral concentrations (Pitt et al. 1994, 1996).

3.1.7 Summary of Stormwater Contaminants of Concern for Groundwater Quality Impacts

Table 2 provides a summary of the potential risks to groundwater quality posed by the stormwater contaminants. The level of risk posed by each category of contaminant is discussed, as well as the degree to which risks can potentially be addressed through mitigation measures such source controls, pretreatment, and separation to groundwater. Specific mitigation measures are identified in Section 4.

Table 2. Summary of Potential Stormwater Contaminants of Concern for Groundwater Quality

Constituent	Roadway-related Sources	Relative Abundance in Roadway Stormwater Runoff in Soluble Phase	Mobility through Vadose Zone	Relative Stormwater-Related Contamination Risk	Potential for Remaining Risk after Mitigation Measures (See Section 4.4)
Nitrogen	• Fertilizers • Vehicle exhaust • Petroleum products • Plant materials • Animal droppings and remains • Compost amendments	Low to Moderate	Moderate to High, as nitrate Greatest during cooler weather	Low to moderate; while nitrate is has a high potential for leaching to groundwater, relatively low concentrations are typically observed in stormwater runoff which tend to be well below groundwater quality objectives. Agricultural or industrial land uses in the tributary watershed are an indicator of elevated risk.	Limited risk, except where groundwater is very sensitive to nitrogen inputs, or where natural sources of nitrogen exist in soils that may be mobilized by stormwater infiltration.
Phosphorus	• Fertilizers • Detergents in gasoline • Motor oil • Plant materials • Animal droppings and remains • Compost amendments	Low to Moderate	Low to Moderate Greatest where soils have limited potential to sorb dissolved phosphorus.	Low; phosphorus concentrations tend to be relatively low in highway runoff and mobility in subsurface environments tends to be limited. Where soils have been used for historic agricultural uses, risks may be elevated.	Limited risk with appropriate mitigation measures, except where soils have received historic phosphorus loadings and may be a source of phosphorus.
Pesticides	• Pesticide use in landscaped areas	Low to Moderate	Low to High Greatest in sandy soils, high water table, and pesticides with high solubility and low affinity for organic matter.	Low to moderate; concentrations in highway stormwater runoff tend to be low, and pesticide concentrations can be managed through application, however some combinations of pesticide types and soil conditions are conducive for pesticides to migrate to and persist in groundwater.	Limited risk with appropriate mitigation measures.
Organic Compounds	• Oils, gasoline • Asphalt, coal tar • Agricultural runoff • Other	Moderate to High	Low to Moderate Greatest in sandy soils and where there is a high water table.	Low to moderate; most organics are removed in soils prior to reaching groundwater.	Limited risk with appropriate mitigation measures.
Pathogens	• Human waste • Animal droppings and remains • Septic systems	Highly variable; may be highest in urban areas due to human waste and pet waste	Low for bacterial pathogens Moderate to High for enteroviruses	Moderate to high; pathogens are commonly present in stormwater runoff. While roadways do not have elevated risks compared to other land uses, pathogens may be present at elevated levels. Enteroviruses are highly mobile and have a high risk of groundwater contamination if infiltrated.	Limited risk for bacterial pathogens with appropriate mitigation measures Enteroviruses may pose an unavoidable risk where they are present and their introduction into groundwater is not tolerable.
Metals (copper, zinc, lead	• Vehicles • Roadway debris • Roadway paint • Roadway materials	Moderate to High relative to surface water toxicity criteria Generally Low relative to drinking water criteria	Low, removed through sorption and filtration	Low to moderate; metals are generally removed through sedimentation, filtration, sorption and precipitation prior to reaching groundwater. Indicators of elevated risk include industrial land uses, inorganic soils, and shallow depth to groundwater. Zinc appears to exhibit the highest risk.	Limited risk with appropriate mitigation measures.
Salts	• Roadway salting • Fertilizers • Natural mineral leaching • Reclaimed water irrigation	Seasonally High	High	Seasonally high; where colder temperatures and roadway freezing requires salting, added loading of salts increases the risk of groundwater contamination.	Risks of salt loadings may be difficult to mitigate, except where it is feasible to: (1) divert runoff from infiltration BMPs during winter months, (2) avoid using salt (risks of alternative approaches should be evaluated), or (3) coordinate stormwater management approach with a local salt management plan.

3.2 Potential for Groundwater Impacts Associated with Existing Soil and Groundwater Contamination

Legacy contamination of soil and groundwater is common in areas that have previously been used for urban development or agricultural uses. The most common constituents of concern are volatile organic compounds; especially chloroform, solvents like PCE and TCE, and the gasoline oxygenate MBTE. These compounds have been observed to persist in groundwater and have been detected in aquifers throughout the United States (Moran, et al. 2006). Subsurface contamination may also take the form of septic systems, cemeteries, and informal municipal solid waste disposal sites, with similar considerations.

Where soil or groundwater contamination exists below a project site or in the project vicinity, stormwater infiltration may have a number of negative consequences that should be carefully evaluated (Gobel et al. 2004):

1. Infiltration of stormwater can mobilize pollutants in contaminated soils and provide a mechanism to transport these pollutants to groundwater. This can threaten groundwater quality and complicate soil cleanup efforts.
2. Infiltration of stormwater can influence the behavior of existing groundwater pollutant plumes through the addition and/or concentration of infiltrating water over current conditions. This can result in a plume shifting its direction of movement, accelerating movement and/or spreading and becoming more diffuse. Where cleanup efforts are underway, this can complicate these efforts and make isolation of the plume from groundwater sources more challenging. Where cleanup activities have not started, the spreading of a plume generally makes cleanup efforts more challenging and costly and may threaten drinking water supplies.
3. Where stormwater infiltration results in a localized increase in groundwater table, there may be potential for the water table to intersect with contaminated soil and/or utilities that would otherwise be “high and dry”. Finally, infiltration may also disrupt the natural degradation process of contaminants in soil or groundwater by introducing additional waters, nutrients, or limiting reagents to the environment.

Soil and groundwater contaminants are not solely anthropogenic. In some cases, historic geologic deposits may contain elevated levels of natural contaminants such as selenium and arsenic which can be mobilized through stormwater infiltration. For example, the central portion of Orange County (California) is underlain by a selenium plume that originated from selenium contained in natural sediments deposited over geologic time, which has been released as a result of changes in water table with agricultural development and urbanization. Infiltration of stormwater in this vicinity has the potential to further mobilize selenium from soils as well as increase the volume of contaminated groundwater discharged to creeks.

It is possible that in some cases, stormwater infiltration could be used as part of a solution to address groundwater or soil contamination, either through dilution or by strategically influencing the movement of a plume. Such an approach should generally be implemented in close coordination with groundwater management and/or cleanup authorities. Soil and groundwater contamination are highly site-specific considerations that should be evaluated carefully for each project site where historical data suggests that contamination is present or may be present.

3.3 Potential Risks to Groundwater Quality from Roadway Contaminant Spills

The U.S.DOT estimates that 7 percent of all trucks travelling the nation's roadways are carrying hazardous material (Federal Highway Administration 2009). As such, contaminant spills are a constant risk in the roadway environment, with the potential to deposit high concentrations and volumes of pollutants onto the roadway and into the roadway drainage system within a short period of time. While most state DOTs have procedures defined for reporting and handling spills, contaminants may begin to infiltrate before responders are able to contain and remove the source. Infiltration of spilled contaminants may occur whether there are infiltration BMPs present or not, however, in theory, the use of infiltration BMPs may accelerate the rate at which the contaminants are able to percolate into groundwater and also may concentrate these contaminants into small areas such that the natural ability of soils to attenuate and remove contaminants are more limited. Conversely, the presence of infiltration BMPs may help prevent spilled pollutants from draining to surface waters where they may have impacts on the downstream environment and receiving waters. Much of the spill may be contained within the infiltration system if runoff is not present at the time of the spill.

Based on a review of 10 years of roadway spill records in Alabama, Becker et al. (2001) found that hydrocarbons (including diesel oil, road tar, gasoline, fuel oil, asphalt, liquefied petroleum gas, jet fuel, hydraulic oil, and creosote) were the most commonly spilled constituents in highway incidents by a large margin and were also released in the greatest volumes. Ammonia and ammonium nitrate were also released, but at much less frequency and lower quantities. Other types of spills are possible; however data suggests that spills other than petroleum hydrocarbons tend to be quite rare.

Per the discussion in Section 3.1.3, petroleum hydrocarbons (organic compounds) are generally retained within a relatively thin layer of surficial soils and would not be expected to pose a significant risk to groundwater quality in most cases where infiltration occurs through soils and/or media. BMPs that are underlain by a thick layer of soil or that have amended media filtration processes would theoretically provide a high level of control for petroleum hydrocarbon spills and limit the potential for groundwater contamination. Higher risk of contamination may occur when groundwater table is high and soils are sandy or gravelly with limited organic content. Where these conditions prevail, it may be desirable to include BMP design components isolate groundwater contamination pathways such as including a containment vault with isolation valves upstream of underground infiltration systems to contain inflow of contaminants. However, the general measures that are used to address potential impacts from chronic stormwater loadings in sandy soils and high groundwater conditions (e.g., pretreatment and amended soils) would also tend to be effective for controlling and containing most spills.

Miklas and Grabowiecki (2007) evaluated the potential for permeable pavements to capture and degrade petroleum hydrocarbons and concluded that relatively large hydrocarbon spills can be contained in permeable pavement systems. They found that the gravel, sand, and soil that make up permeable pavements function as hydrocarbon traps and in-situ bioreactors under some conditions. Similar findings would likely apply to bioretention areas and similar systems that have amended soils and plant roots. In the event of any significant spills, the systems would need to be remediated to remove contaminated soil and media.

Literature was not identified that specifically considered the potential risks posed by spills of solvents or other mobile contaminants; however these types of spills are believed to be very uncommon and highway designs may not be able to provide risk mitigation measures for very infrequent and unpredictable occurrences.

4 Assessing and Mitigating Potential Stormwater Infiltration Impacts on Water Balance and Groundwater Quality

This section is intended to provide guidance for assessing the potential for impacts of stormwater infiltration related to water balance and groundwater quality. This section also provides recommendations for mitigating these potential impacts to potentially improve the level of infiltration that can safely be achieved in the highway environment.

What and how should factors be considered in evaluating whether infiltration of roadway runoff is infeasible or undesirable related to site water balance and groundwater quality?

How can potential impacts be mitigated to reduce risks and improve feasibility and desirability of infiltration?

This section is divided into seven key factors that should be considered in assessing the feasibility and desirability of infiltration related to water balance and groundwater quality:

- Localized groundwater mounding,
- Water balance impacts on streamflow,
- Contamination from stormwater runoff pollutants,
- Soil and groundwater contamination,
- Wellhead and spring protection,
- Special considerations for Karst aquifers, and
- Local groundwater management objectives and criteria.

Key to summary tables in this section: The summary tables included within each subsection below contain guidance for evaluating the potential level of risk associated with each feasibility factor and provides recommendations for additional analysis and mitigation measures where site-specific factors suggest that risks may be elevated. The intent of these tables is to help users develop planning level classifications of potential risk. Each of these factors may warrant more extensive analysis than described in this planning level document, but the tables and guidance here will assist in project planning by focusing attention where the potential for impacts is greatest. Where elevated risk indicators are not present, a simplified assessment may be adequate. The greater number of elevated risk indicators present for a given site indicates a higher level of risk, which may require a more extensive assessment to demonstrate that infiltration can be safely done or may represent a technical basis for infiltration to be considered infeasible. These tables are intended for planning level screening only -- the final determination of whether to implement infiltration in an urban roadway environment should be based on site specific information and analysis, coordination with other applicable agencies, and best professional judgment.

4.1 Localized Groundwater Mounding

The use of infiltration BMPs has the potential to result in elevated local groundwater tables that exceed the natural seasonal high groundwater table or extend the duration of ponding at a site. This can have a number of negative consequences, as introduced in Section 2.2, including:

- Infrastructure damage,
- Geotechnical hazards,
- Inflow and infiltration into the sanitary sewer,
- Conveyance of water in utility trenches,
- Mobilization of groundwater contaminants,
- Reduced separation to groundwater, resulting in greater risk of contamination (See Section 4.3), and
- Undesirable surface ponding.

The mounding potential beneath a particular BMP is a function of the facility design, the infiltration rate, the precipitation rate, and the hydrogeologic conditions at the site. Table 3 provides indicators for assessing whether groundwater mounding may be of specific concern for a project.

Table 3. Risk Indicators for Localized Groundwater Mounding

	Lower Risk Indicators	↔	Elevated Risk Indicators
Risk Factors	• Depth to seasonally high groundwater relatively large (greater than 10 feet) (Pitt et al. 2004) • Relatively steep groundwater gradient; groundwater mounding quickly dissipates • Relatively simple geology; roughly uniform infiltration expected • Smaller footprint and linear BMPs • Storm events are well-distributed throughout the year		• Depth to seasonally high groundwater is low to moderate (less than 10 feet) (Pitt et al. 2004) • Relatively little groundwater gradient • Complex geology with impermeable lenses, potential faults or other barriers to vertical or lateral dissipation • Larger footprint BMPs • Storm events tend to arrive in clusters, creating critical periods for localized groundwater mounding
Potential Additional Studies and Mitigation Measures for Sites with Elevated Risk	• Conduct more detailed investigation of groundwater depths and gradients to refine qualitative classification of the potential for mounding. • Conduct computational analysis of groundwater mounding or conduct large scale pilot infiltration testing (See White Paper 1) to assess capacity of subsurface geology to safely accept water. • Use BMPs that are more distributed (smaller footprint area per unit) or are more linear in nature to help reduce mounding potential. Smaller facilities are those located at intervals within the normal right-of-way width, whereas linear facilities are those that are continuous over much of the length of the right-of-		

	way. These BMPs include bioretention areas (a smaller footprint area), or permeable shoulders (a linear facility). • Develop a stormwater management strategy involving less infiltration; infiltration of the full water quality or hydromodification control volume may not be feasible or desirable in conditions with unavoidable potential for groundwater mounding.

Where predominantly lower risk indicators exist, the use of a simplified method is generally appropriate. For example, USGS has developed a spreadsheet tool for rapid assessment of mounding potential (Carleton 2010). The tool (available at http://pubs.usgs.gov/sir/2010/5102/) estimates the maximum expected mounding below an infiltration BMP by solving simplified groundwater flow equations that incorporate a number of simplifying assumptions about the hydrogeology of the site. For more complex or critical conditions, it may be appropriate to utilize a more robust modeling framework, such as the USGS MODFLOW model, which is a finite-difference groundwater flow model (http://water.usgs.gov/nrp/gwsoftware/modflow.html). It is appropriate for the civil and geotechnical engineers to use best professional judgment regarding the selection of the methods used to assess this factor.

4.2 Water Balance Impacts on Stream Flows

As discussed in Section 2.2, the use of infiltration systems to reduce surface water discharge volumes may result in additional volume of deeper infiltration compared to natural conditions, which may result in impacts to receiving channels associated with change in dry weather flow regimes. This is a function of the local climate, but also the local hydrogeologic conditions. To assist in project planning, a number of indicators may be used to help evaluate the potential for impacts, as summarized in Table 4. A relatively simple survey of hydrogeologic data (piezometer measurements, boring logs, regional groundwater maps) and downstream receiving water characteristics is generally adequate to determine whether there is potential for impacts and whether a more rigorous assessment is needed. Since linear highway projects typically make up a relatively minor portion of any watershed, a single project is unlikely to significantly change the overall watershed water balance. More typically, the practical question is whether infiltration of highway runoff would exacerbate an existing water balance condition rather than necessarily create a new condition of concern. A review of local watershed conditions may be adequate to determine if this factor warrants further considerations. If water balance concerns do exist in the watershed, it may be most appropriate to follow the lead of local jurisdictions in terms of watershed-scale priorities and approaches.

Where water balance conditions appear to be sensitive to development impacts and there is an elevated risk of impacts, a computational analysis may be warranted to evaluate the feasibility/desirability of infiltration. Such an analysis should account for precipitation, runoff, irrigation inputs, soil moisture retention, ET, baseflow, and change in groundwater recharge on a long-term basis. Because water balance calculations are sensitive to the timing of precipitation versus ET, it is most appropriate to utilize a continuous model simulation rather than basing calculations on average annual or monthly normal conditions. The USGS Soil Water Balance Model (http://pubs.usgs.gov/tm/tm6-a31/), the EPA Stormwater Management Model (SWMM, http://www.epa.gov/nrmrl/wswrd/wq/models/swmm/), or other models may be appropriate for these calculations. Where cumulative watershed impacts are of concern, other studies prepared by local jurisdictions may be informative to understand the feasibility and desirability of infiltrating highway runoff.

Table 4. Risk Indicators for Water Balance Impacts on Stream Flows

	Lower Risk Indicators	↔	Elevated Risk Indicators
Risk Factors	• Actively managed aquifer with adequate capacity for additional infiltrated volume • Perennial streams in vicinity that naturally receive base flow from groundwater discharge year round • Relatively simple geology; predictable vertical migration of water • Higher order (larger) streams • Existing proportion of precipitation lost to ET is lower (See Figure 1) • Proposed BMPs provide substantial ET losses; less potential to increase deeper infiltration compared to natural conditions • Current conditions to do not show signs of water balance issues		• Perched groundwater or shallow, local aquifer • Streams in the vicinity are ephemeral or intermittent and have sensitive species/habitat issues • More complex and problematic geology; e.g., sloping strata with potential for lateral migration to channel banks • Lower order (smaller) streams • Existing proportion of precipitation lost to ET is higher (See Figure 1) • Future development is projected over a significant proportion of watershed (potential for cumulative effects) • Current conditions show signs of water balance issues
Potential Additional Studies and Risk Mitigation Measures for Sites with Elevated Risk	• Conduct more detailed investigation of hydrogeology and water fluxes. • Conduct computational analysis of water balance on a long-term basis. • Select BMPs with greater quantity of amended soil media, greater vegetation, and shallower depth to achieve a more natural balance between infiltration and ET. • Develop a stormwater management strategy involving less infiltration; infiltration of the full water quality or hydromodification control volume may not be feasible or desirable in cases where water balance impacts are significant and cannot be avoided. • If cumulative development impacts are of concern, potentially collaborate with local jurisdictions to evaluate potential water balance impacts, or adhere to findings of studies prepared by local jurisdictions.		

4.3 Pollutants in Stormwater Runoff

As discussed in Section 3.1, the concentration of stormwater pollutants is highly dependent on the land uses and activities present in the area tributary to an infiltration BMP. Likewise, the potential for groundwater contamination is a function of pollutant abundance, speciation of pollutants in soluble forms, and the mobility of the pollutant in the subsurface soils. Therefore, an evaluation of a large number of potential conditions that may be encountered with a range of associated risks and a site-specific assessment is recommended. To assist in project planning, Table 5 provides risk indicators to help evaluate the potential for impacts and identify pollutants that may warrant greater analysis. This table also provides potential mitigation measures to help reduce risks – these include general mitigation measures, as well as pollutant-specific mitigation measures.

Table 5. Potential for Contamination from Stormwater Runoff Pollutants

	Lower Risk Indicators	↔	Elevated Risk Indicators
Risk Factors	• Lower traffic volumes, lower heavy truck traffic and contaminant sources[1] • Soils have substantial pollutant attenuation capacity[1] • Depth to mounded seasonal high groundwater exceeds 10 feet • Climate does not necessitate significant roadway salting • Anthropogenic sources of pathogens are limited in tributary area		• Higher traffic volumes and contaminant sources[1] • Soils have limited pollutant attenuation capacity[1] • Depth to mounded seasonal high groundwater is less than 10 feet • Climate necessitates significant roadway salting • Sources of anthropogenic pathogens are present in tributary area • Karst topography (see Section 4.7)

Potential Additional Studies and Risk Mitigation Measures for Sites with Elevated Risk	• Characterize depth to groundwater – locate and design BMPs to maintain separation of 10 to 20 feet to seasonal high groundwater – greater separation appears to result in lower risk for all constituents of concern.
	• Review data and/or conduct monitoring to characterize expected runoff quality for the project; evaluate expected stormwater quality vs. local groundwater quality objectives.
	• Utilize BMPs that provide treatment to runoff prior to deeper infiltration to address pollutants of concern, such as amended soil media to augment removal capacity of native soil – effective for pesticides, metals, many organics and bacterial pathogens; limit subsurface infiltration/injection.
	• Utilize practices to reduce pollutant loadings, such as permeable friction course overlays, which result in less splashing and lesser quantity of pollutants.
	• Utilize pollutant-specific source controls, as applicable:
	  o Nutrients - limit fertilizer applications, use of slow-release fertilizers.
	  o Pesticides - limit application of pesticides; utilize pesticides with lower mobility and shorter half-life.
	  o Pathogens – provide sanitary facilities; provide animal waste disposal; implement program to clean up animal remains.
	  o Salts – limit application, consider alternative methods, and potentially divert runoff from salted roadways in cold weather.
	• Identify expected high source areas, such as maintenance yards and gas stations, and design drainage system to hydrologically isolate these areas from infiltration BMPs.
	• Develop a stormwater management strategy involving less infiltration; infiltration of the full water quality or hydromodification control volume may not be feasible or desirable in cases where potential contamination cannot be mitigated.		

1 – Indicators of loading and soil pollutant attenuation capacity are included in the following bullet lists.

The following list includes indicators of elevated sources that may be applicable in the urban highway environment (SARWQCB 2009; WADOE 2012; U.S. EPA 2009):

- Average daily traffic volume - A traffic volume threshold of 25,000 or 30,000 average daily traffic (ADT) has been applied in a number of permits and guidance documents as an indicator of conditions that could potentially threaten groundwater quality. Note that some researchers attributed potential water quality concerns in these areas as general urban sources vs. the specific traffic volumes (i.e. with ADTs of this level, monitoring sites were located in more dense urban land use areas.)
- Commercial or industrial sites subject to an expected average daily traffic count (ADT) ≥100 vehicles/1,000 ft^2 gross building area (trip generation). Particularly those with heavy truck traffic.
- Other areas with potential high threat to water quality, such as industrial or light industrial activities; fleet maintenance, storage, and/or wash yards; and nurseries.
- Material storage areas, such as asphalt or salt storage areas.
- Other locally-applicable guidance at the discretion of the project engineer.

The following list includes indicators of the pollutant attenuation capacity of soils (WADOE 2012):

- Cation exchange capacity (CEC) of the treatment soil should be at least 5 milliequivalents CEC/100 g dry soil (U.S. EPA Method 9081). CEC values of greater than 5 meq/100g are expected in loamy sands, according to Rawls, et al. Lower CEC content may be considered if it is based on a soil loading capacity determination for the target pollutants that is accepted by the local jurisdiction.
- Organic content of the treatment soil (ASTM D 2974): Organic matter can increase the sorptive capacity of the soil for some pollutants. A minimum of 1.0 percent organic content is recommended.
- Depth of organically-active soils. An assessment of CEC and organic content should encompass all distinct layers below the base of the facility to a depth of at least 2.5 times the maximum design water depth, but not less than 6 feet.
- Other locally-applicable guidance at the discretion of the project engineer.

4.4 Existing Soil and Groundwater Contamination

Section 3.2 describes the potential impacts that may result when stormwater is infiltrated in the vicinity of existing groundwater plumes, contaminated soils, septic systems, cemeteries, municipal solid waste disposal sites, and other potential sources of groundwater pollution. When conducting a site assessment to assess the feasibility of stormwater infiltration, the following sources may contain information that is useful in determining the potential risks associated with existing contamination:

- Records of previous uses of the site.
- Locations of current or historic septic systems, cemeteries, historic (informal) refuse dumps.

- Mapping of contaminated groundwater plumes and soils, which may be available from local groundwater management agencies or state environmental quality agencies. For example, spatial databases and layers containing contaminated sites were identified in California, New Jersey, Connecticut, and Alaska (see References) as part of this effort; similar resources likely exist or are under development in other states.
- U.S. EPA Envirofacts database, which contains locations and information regarding RCRA, CERCLA, brownfields, and cleanup sites (U.S. EPA 2012a). Individual states may also have more detailed information available.

Table 6 summarizes risk factors that may indicate an elevated degree of risk and provides recommendations for potential mitigation measures related to infiltrating near groundwater plumes or contaminated soils.

Table 6. Groundwater Contaminant Plumes and Soil Contamination

	Lower Risk Indicators	Moderate Risk Indicators ↔	Elevated Risk Indicators
Risk Factors	• Contamination not suspected at site • Local data confirms no contamination • Historic land uses do not include industrial, agricultural or other uses suggesting potential for contamination • Septic systems, cemeteries, municipal solid waste sites not present in project vicinity	• Historic site uses included industrial, agricultural, or other uses suggesting potential contamination, but contamination is unknown • Historic contamination is documented but has been fully remediated	• Known contamination at the site • Groundwater plumes (natural or unnatural) exist in site vicinity • Septic systems, cemeteries, municipal solid waste sites present in site vicinity and hydrogeologic connections with these sources are possible
Potential Additional Assessments and Risk Mitigation Measures for Sites with Elevated Risk	• Consult with regulatory agencies responsible for site cleanup and groundwater protection regarding potential benefits or consequences associated with stormwater infiltration at the site (see Section 5). • Conduct more detailed investigation of potential contamination to reduce uncertainties and better quantify potential risk. • Site infiltration BMPs to avoid infiltration of stormwater where there is significant unavoidable risk of mobilizing of pollutants or spreading of groundwater plumes. • Develop a stormwater management strategy involving less or no infiltration; infiltration of the full water quality or hydromodification control volume may not be feasible or desirable in cases where potential for mobilization or spreading of existing contamination cannot be avoided.		

4.5 Wellhead and Spring Protection

Wellheads and springs, natural and man-made, are water resources that may potentially be adversely impacted by stormwater infiltration through the introduction of contaminants or alteration in water supply and levels. It is recommended that the locations of wells and springs be identified early in the design process and site design be developed to avoid infiltration in the vicinity of these resources. Setbacks of 100 to 200 feet from springs and wellheads are typical in existing guidance (OCPW 2011, VCWPD 2011; WADOE 2012). In Washington State, a wellhead protection zones are also defined by the 1-year, 5-year, and 10-year travel times. It may be appropriate to develop site-specific setbacks. Table 7 summarizes the risk indicators associated with infiltration near wellheads and springs and recommends potential mitigation measures.

Table 7. Risk Indicators for Wellhead and Stream Protection

	Lower Risk Indicators	↔	Elevated Risk Indicators
Risk Factors	• No springs or wellheads in close proximity to infiltration BMPs (greater than 100 to 200 feet; and greater than 1-year travel time, as applicable) • Low levels of contaminants expected in stormwater runoff after effective treatment • Infiltration rates are low and soils have substantial pollutant attenuation capacity • Groundwater is relatively deep (greater than 10 to 20 feet)		• Springs and/or wellheads in close proximity to infiltration BMPs (less than 100 to 200 feet; or within 1-year travel time, as applicable) • High levels of contamination in stormwater runoff even after effective treatment • Infiltration rates are high, and soils have more limited pollutant attenuation capacity • Groundwater is relatively shallow (less than 10 to 20 feet) • Karst topography (see Section 4.7)
Potential Risk Mitigation Measures for Sites with Elevated Risk	• Locate infiltration BMPs to maintain recommended setbacks. • Coordinate with local water provider or health department responsible for wellhead protection. • Follow mitigation measures for stormwater pollutants identified in Section 4.4. • Follow mitigation measures for existing soil and groundwater contamination in Section 4.3. • Develop a stormwater management strategy involving less infiltration; infiltration of the full water quality volume may not be feasible or desirable if recommended setbacks cannot be preserved and/or risks to drinking water supplied or springs cannot be avoided.		

4.6 Contaminant Spills

As discussed in in Section 3.3, contaminant spills are a constant risk on roadways, with the potential to deposit high concentrations and volumes of pollutants before responders can control the source of the spill. However, the most common spills are petroleum hydrocarbons, and the risk of these spills contaminating groundwater can generally be mitigated with appropriate measures. Table 8 provides indicators of elevated risk and recommendations for potential mitigation measures to reduce these risks.

Table 8. Risk Indicators for Contaminant Spills

	Lower Risk Indicators	↔	Elevated Risk Indicators
Risk Factors	• Lower traffic roadways with lower heavy truck traffic • Groundwater is relatively deep (greater than 10 to 20 feet) • Tributary area does not contain fueling stations, warehouses, storage tanks, or similar potential sources of spills		• Higher traffic roadways, particularly for truck traffic • History of spills in nearby area may be indicative of higher risk • Groundwater is relatively shallow (less than approximately 10 feet) • Tributary area contains fueling stations, warehouses, storage tanks, or similar potential sources of spills • Karst topography (see Section 4.7)
Potential Risk Mitigation Measures for Sites with Elevated Risk	• Include amended media treatment layer below the infiltration media, as needed when inert sandy or gravely soils are present, to improve sorption capacity to retain spills and improve bioremediation potential. • Consider treatment train options: oil-water separator, oil contamination boom, media filters, biofiltration systems, vegetated filter strip, upstream of infiltration system. • Develop spill response plan for project if a general spill response plan is not already administered by the project owner; include provisions for protection and remediation of stormwater BMPs in the event of a spill. • Include isolation/diversion mechanism that can be activated by emergency responders in the event of a spill to prevent contamination from either entering the BMP if adequate storage is available and/or capturing the spill within the BMP prior to discharge to either groundwater or a surface receiving water; note it may not be practicable to isolate or divert flow from some BMP types, such as permeable pavement shoulders or filter strips which receive runoff as sheet flow.		

4.7 Special Considerations for Karst Topography

Karst topography refers to a specific geologic formation that has been shaped by the dissolution of soluble bedrock elements. Karst topography is most frequently associated with limestone or dolomite rock, but may be present in other types of rock as well. The Karst landscape is characterized by containing sinkholes, underlying caves, and springs. Karst topography is present in various locales throughout the United States and is not characteristic of any one region.

Karst topography has a number of unique considerations from the perspective of BMP implementation, including elevated potential for groundwater contamination from stormwater infiltration. This elevated risk is a result of thin or non-existent surface soil layers beneath the surface prior to the Karst formation, which results in little to no natural filtration of runoff. Karst aquifers also provide the potential for direct hydraulic connections from the surface to groundwater via sinkholes, springs, and caves (Donaldson 2004; Weiss et al. 2008).

The EPA NPDES *Menu of BMPs* states that infiltration BMPs may not be used in regions with Karst topography due to the potential to create sinkholes or cause groundwater contamination (http://cfpub.epa.gov/npdes/stormwater/menuofbmps/index.cfm?action=factsheet_results&view=specific &bmp=69). Unless site-specific analyses determine that infiltration can be safely achieved without impacts through the use of robust treatment and spill containment methods, infiltration of roadway runoff should be avoided in Karst areas.

5 Consultation with Local Agencies

As introduced above, infiltration may play an important role in improving groundwater resources, and projects may be able to demonstrate multiple benefits as a result of careful application of infiltration approaches. However infiltration may pose significant risks to groundwater resources and environments, and infrastructure that can be impacted by changes in groundwater levels. In general, it is a best practice for DOTs to coordinate with agencies responsible for local groundwater management and underground utilities and resource agencies whenever infiltration is considered for a project. These agencies have a vested interest in protecting groundwater supplies and underground infrastructure and usually have extensive knowledge about these resources.

5.1 Local Groundwater Suppliers

Consulting with applicable groundwater supply agencies early in the project development process can help simplify the process of evaluating feasibility and desirability of infiltration. These agencies may be able to provide information to assist in evaluating the feasibility of infiltrating stormwater and may have locally-applicable criteria, maps, or other resources that have already been developed. Groundwater supply agencies with groundwater management authority may include local governments, special water supply districts, or others.

A potential model for inter-agency coordination was developed in Orange County (CA) as part of development of the Orange County Technical Guidance Document (Orange County Public Works 2011). The Technical Guidance Document was developed by Orange County Public Works (OCPW), and includes guidance of evaluating the feasibility of infiltration stormwater, with consideration of groundwater quality, among other factors. The Orange County Water District (OCWD) is a public agency responsible for providing water to more than 20 cities and more than 2 million residents of Orange County. OCWD is responsible for management of the water quality and basin yield of the groundwater basin that underlies the Santa Ana River in northern Orange County. OCWD was actively involved in the development of the Technical Guidance Document, as they have an interest in enhancing groundwater supplied through infiltration and have significant concerns about the potential for groundwater contamination due to stormwater infiltration. Through a collaborative approach, OCPW and OCWD developed criteria acceptable to both agencies, including the use of groundwater levels and plume locations provided from OCWD records to help develop screening tools. The agencies also established a process by which projects identified as having an elevated risk of groundwater impacts would be submitted for review by OCWD.

5.2 Water Resources Protection Agencies

Groundwater protection requirements are typically administered by state environmental quality agencies or regions of U.S. EPA. These agencies are commonly the same agencies responsible for administering stormwater and surface water regulations; however different departments may be responsible for surface water quality versus groundwater quality. Consulting with local resource agencies responsible for groundwater quality protection can also help streamline the process of evaluating feasibility and desirability of stormwater infiltration, for many of the same reasons introduced in Section 5.1. Water resources protection agencies are typically responsible for establishing groundwater quality objectives and developing plans to protect or improve groundwater quality, where needed. These agencies are also typically responsible for administering the cleanup of contaminated sites. Because of their multiple-resource purview, these agencies may be able to help provide input and strike a balance between the surface water quality benefits of stormwater infiltration versus potential impacts to water balance or groundwater quality. Consultation can also help identify if stormwater infiltration would have the potential to exacerbate existing problems that resource protection agencies are working to address.

Additionally, depending on the location and design, infiltrating facilities may be considered “Class V Injection Wells” under the federal Underground Injection Control (UIC) program. Class V wells are defined as systems that are used to inject non-hazardous fluids underground, which could include dry wells or deep infiltration basins. The minimum requirements for compliance are that fluid injected underground may not endanger underground drinking water sources and that the owners/operators must submit the required inventory information to their permitting authority. Some areas may have additional requirements and more information is available on the EPA website (U.S. EPA 2012b). UIC programs are generally administered by state environmental quality agencies in most states.

5.3 Sanitation Districts and Other Underground Utilities

Infiltration of stormwater from roadway projects has the potential to elevate local groundwater tables, which can increase the amount of inflow and infiltration (I&I) to the sanitary sewers and/or cause impacts to other utilities. This risk is particularly high where groundwater levels are already relatively close to the elevation of sewer and/or utility elevations. Infiltrated stormwater can migrate into sewer lines, or flow within utility trenches.

The local operators of sewer collection and treatment systems should be consulted early in the planning process to evaluate whether stormwater infiltration may pose a concern related to increases in inflow and infiltration. An increase in inflow and infiltration may place an additional burden on these agencies, with respect to hydraulic conveyance capacity and waste water treatment plant treatment capacity, potentially resulting in increased incidence of sanitary sewer overflows. As a result, these agencies may have locally-applicable criteria, maps, or other resources that have already been developed to assist in identifying areas where increased stormwater infiltration may be undesirable.

Similarly, agencies or companies that operate other underground utilities may have specific concerns regarding infiltration of stormwater and/or resources to assist in evaluating feasibility and desirability.

6 Summary

The potential risks of stormwater infiltration on water balance and groundwater quality are influenced by many site-specific factors, as introduced in this white paper. While careful attention should be paid to any site-specific actors that indicate elevated risks, in general the risks are relatively low and can be effectively mitigated in a number of ways, including via pretreatment, soil amendments, selection of appropriate infiltration sites, and observing minimum separation criteria between infiltration BMPs and the groundwater table. Additional, watershed-scale plans prepared by local jurisdictions may provide guidance for how cumulative risks can be mitigated to an acceptable level. In cases where elevated risks cannot be mitigated and/or specific local prohibitions are in place, stormwater infiltration may not be feasible and/or desirable.

7 References

Alaska Department of Environmental Conservation (2012). Spill Prevention and Response, http://dec.alaska.gov/spar/guidance.htm.

Becker, S., R. Pitt, and S. Clark (July 2001). Environmental Health, Public Safety, and Social Impacts Associated with Transportation Accidents Involving Hazardous Substances. University Transportation Center of Alabama and US Dept. of Transportation. UTCA Project No. 00214. Tuscaloosa, AL.

California State Water Resources Control Board (2012a). EnviroStor, http://www.envirostor.dtsc.ca.gov/public/.

California State Water Resources Control Board (2012b). GeoTracker, http://geotracker.waterboards.ca.gov/.

Carleton, G.B. (2010). Simulation of Groundwater Mounding Beneath Hypothetical Stormwater Infiltration Basins, United States Geological Survey, Scientific Investigations Report 2010-5012.

Church, M.R., G.D. Bishop and D.L. Cassell (1995). Maps of regional evapotranspiration and runoff/ precipitation ratios in the northeast United States. *Journal of Hydrology*, 168, 283–298.

Clark, S.E., and R. Pitt (2007). Influencing factors and a proposed evaluation methodology for predicting groundwater contamination potential from stormwater infiltration practices. *Water Environment Research*, 79, 29–36.

Clark, S.E.; K.H Baker, J.B. Mikula, and C.S. Burkhardt (2006). Infiltration vs. Surface Water Discharge: Guidance for Stormwater Managers, Water Environment Research Fund Report 04-SW-3.

Connecticut Department of Energy and Environmental Protection, List of Contaminated or Potentially Contaminated Sites in Connecticut, http://www.ct.gov/dep/cwp/view.asp?A=2715&Q= 325018, 2012.

Dierkes, C.; A. Holte, and W.F. Geiger (undated). Heavy Metal Retention within a Porous Pavement Structure, Department of Civil Engineering, Urban Water Management, University of Essen.

Dietz, M.E. (2007). Low impact development practices: A review of current research and recommendations for future directions, *Water Air Soil Pollut*, 186, 351–363.

Donaldson, B.M. (2004). Highway Runoff in Areas of Karst Topography, Virginia Transportation Research Council, VTRC 04-R13.

Federal Highway Administration (FHWA) (2009). Traffic Incident Management in Hazardous Materials Spills in Incident Clearance FHWA-HOP-08-058, January 2009 http://ops.fhwa.dot.gov/publications/fhwahop08058/default.htm.

Fischer, D., E.G. Charles, and A.L. Baehr (2003). Effects of Stormwater Infiltration on Quality of Groundwater Beneath Retention and Detention Basins, *Journal of Environmental Engineering*. 129(5), 464–471.

Göbel, P., H. Stubbe, M. Weinert, J. Zimmermann, S. Fach, C. Dierkes, H. Kories, J. Meßer, V. Mertsch, W.F. Geiger, and W.G. Coldewey. (2004). Near-natural stormwater management and its effects on the water budget and groundwater surface in urban areas taking account of the hydrogeological conditions. - *Journal of Hydrology*, 299, 267–283.

Jefferson, A., A. Nolin, S. Lewis and C. Tague (2008). Hydrogeologic controls on streamflow sensitivity to climate variation. *Hydrol. Process.*, 22, 4371–4385.

Lee, J.G.; and J.P. Heaney (2003). Estimation of Urban Imperviousness and Its Impacts on Storm Water Systems, *Journal of Water Resources Planning and Management*, 129(5), 419–426.

Los Angeles and San Gabriel Rivers Watershed Council (LASGRWC 2010), Water Augmentation Study-Research, Strategy and Implementation Report, Los Angeles and San Gabriel Rivers Watershed Council. http://www.usbr.gov/lc/socal/reports/LASGwtraugmentation/report.pdf.

Miklas, S. and P. Grabowiecki (2007). Review of permeable pavement systems, *Building and Environment*, 42, 3830–3836.

Milly, P.C.D. (1994). Climate, soil water storage, and the average annual water balance. *Water Resources Research*, 30(7), 2143–2156.

Mohseni, O. and H.G. Stefan (2001). Water budgets of two watersheds in different climate zones under projected climate warming. *Climatic Change*, 49, 77–104.

Moran, M.J., P.A. Hamilton, and J.S. Zogorski (2006). Volatile Organic Compounds in the Nation's Ground Water and Drinking-Water Supply—A Summary, United States Geological Survey. USGS Fact Sheet 2006-3048. http://pubs.usgs.gov/fs/2006/3048/.

Najjar, R.G. (1999). The water balance of the Susquehanna River Basin and its response to climate change. *Journal of Hydrology* 219, 7–19.

Oregon State Univeristy, GeoSyntec Consultants, University of Florida, and The Low Impact Development Center, Inc. (2006), *NCHRP Report 565: Evaluation of Best Management Practices for Highway Runoff Control*, Transportation Research Board of the National Academies, Washington, D.C.

New Jersey Department of Environmental Protection (NJDEP) (2012), Site Remediation Program, http://www.nj.gov/dep/srp/kcsnj/.

Ng, E. and P.C. Miller (1980). Soil Moisture Relations in the Southern California Chaparral. Ecology, 61(1), 98–107.

Nightingale, H.I. (1987). Water Quality Beneath Urban Runoff Management Basins. Water Resources Bulletin, 23(2), 197–205.

Orange County (CA) Public Works (OCPW) (2011). Technical Guidance Document for Preparing WQMPs, http://www.ocwatersheds.com/DocmgmtInternet/Download.aspx?id=638.

Pitt, R., S. Clark, and K. Parmer (1994). Protection of Groundwater from Intentional and Nonintentional Stormwater Infiltration, U.S. Environmental Protection Agency, EPA/600/SR-94/051. PB94-165354AS, Storm and Combined Sewer Program, Cincinnati, OH. 187 pp.

Pitt, R., S. Clark, K. Parmer, and R. Field (eds.) (1996). Groundwater Contamination from Stormwater Infiltration, Ann Arbor Press, Ann Arbor, MI, 219 pp.

Pitt, R.; S. Clark, and R. Field (1999). Groundwater contamination potential from stormwater infiltration practices, *Urban Water*, 1, 217–326.

Rose, S. (2009). Rainfall–runoff trends in the south-eastern USA: 1938–2005. *Hydrol. Process.*, 23, 1105–1118.

Sanford, W.E. and D.L. Selnick (2012). Estimation of Evapotranspiration Across the Conterminous United States Using a Regression with Climate and Land-Cover Data. Journal of the American Water Resources Association (JAWRA) 49(1), 217–230. DOI: 10.1111/jawr.12010

Santa Ana Regional Water Quality Control Board (SARWQCB) (2009). Orange County Municipal NPDES Storm Water Permit (Order No. R8-2009-0030, Amended by Order No. R8-2010-0062). http://www.waterboards.ca.gov/rwqcb8/water_issues/programs/stormwater/oc_permit.shtml.

Spinello, A.G. and D.L. Simmons (1992). Base flow of 10 South-Shore streams, Long Island New York, 1976-85, and the effects of urbanization on base flow and flow duration. USGS Water-Resources Investigations Report 90-4205.

Stephenson, J.B., and B.F. Beck (1995). Management of the discharge quality of highway runoff in karst areas to control impacts to groundwater–A review of relevant literature. Multidisciplinary Conference on Sinkholes and the Engineering and Environmental Impacts of Karst, 297–321.

Strecker E.W. and A.L. Poresky (2009). Stormwater Retention On-Site, The Water Report, 2009.

U.S. EPA (2010). MS4 Permit Improvement Guide U.S. Environmental Protection Agency, Office of Water, Office of Wastewater Management, Water Permits Division, April 2010, EPA 833-R-10-001. http://www.epa.gov/npdes/pubs/ms4permit_improvement_guide.pdf.

U.S. EPA (2012a), Envirofacts Database, http://www.epa.gov/enviro/.

U.S. EPA (2012b). Class V Wells, http://water.epa.gov/type/groundwater/uic/class5/index.cfm.

U.S. EPA (2012c). U.S. EPA Permit No. DC0000221. Authorization to Discharge Stormwater Under the National Pollutant Discharge Elimination System, Municipal Separate Storm Sewer System (MS4) Permit. U.S. EPA Region 3, November 9, 2012.

Ventura County (CA) Watershed Protection Division (2011). Technical Guidance Manual for the Preparation of Stormwater Quality Management Plans. http://vcstormwater.org/index.php?option=com_content&view=article&id=32&Itemid=45.

Washington Department of Ecology (WADOE) (August 2012). Stormwater Management Manual for Western Washington. Publication Number 12-10-030.

Ward, G.H. (1993). A water budget for the state of Texas with climatological forcing. *The Texas Journal of Science*, 45(3), 249–264.

Washington State Department of Transportation (WSDOT) (2011). Highway Runoff Manual. M 31-16.03. http://www.wsdot.wa.gov/publications/manuals/fulltext/M31-16/HighwayRunoff.pdf.

Weiss, P.T., G. LeFevre, and J.S. Gulliver (2008). Contamination of Soil and Groundwater Due to Stormwater Infiltration Practices, University of Minnesota, St. Anthony Falls Laboratory, Project Report No.515.

Appendix E

Geotechnical Considerations in the Incorporation of Stormwater Infiltration Features in Urban Highway Design (White Paper #3)

1 Introduction

Design of infiltration systems includes a feasibility evaluation to determine areas where infiltration might be suited, as well as site-specific evaluations to quantify potential environmental and geotechnical impacts. Addition of water to the subsurface can result in minor to significant impacts on existing conditions and infrastructure within the vicinity of the infiltration feature, ranging from reduced infiltration to settlement or slope failures. This White Paper is intended to be used to help identify the technical factors that should be considered in conducting a site-specific geotechnical evaluation of the potential impacts of stormwater infiltration, and to help guide the development of geotechnical designs that safely allow for an enhanced degree of volume reduction in the urban highway environment. This White Paper is intended to be used in conjunction with the comprehensive feasibility screening process described in Section 5 the Guidance Manual, which includes a number of feasibility screening factors beyond geotechnical factors. This White Paper is primarily intended for stormwater engineers to understand the potential geotechnical impacts that stormwater infiltration may have on surrounding features (i.e., after water leaves these systems). This White Paper is organized as follows:

Section 2 provides a discussion on how infiltration feasibility factors into the geotechnical investigation and design process. In other words, what types of investigations and analysis are appropriate at each design phase?

Section 3 identifies several key forms of geotechnical failure that may be associated with stormwater infiltration, including discussion of how these failure mechanisms should be assessed, analyzed, and potentially mitigated in the planning and design process.

Section 4 provides more detailed information about certain "mitigation measures" that may be incorporated into planning and/or design to help mitigate elevated risks posed by stormwater infiltration and potentially allow for greater volume reduction.

Section 5 provides a synthesis of geotechnical feasibility factors and general guidance regarding how they may influence different types of volume reduction approaches.

Section 6 provides a summary of recommended contents of project-specific reports related to investigation of geotechnical feasibility of stormwater infiltration and associated design parameters**.**

The geotechnical factors and potential mitigation measures presented in this White Paper were based on the following range of stormwater VRAs:

1. Direct infiltration into roadway subgrade – designs include permeable pavement with direct infiltration and collection of runoff within the roadway shoulder and routing back beneath the roadway for infiltration;
2. Infiltration in shoulders – designs include collecting drainage from the roadway and infiltrating within permeable pavement strips or other features along roadway shoulders;
3. Compost amended slopes/filter strips adjacent to roadway – designs incorporate compost or other decompaction approaches to increase absorption and encourage subsurface infiltration as water sheet flows away from the roadway;
4. Channels, trenches, and other linear depressions parallel to roadway - designed to achieve direct and incidental infiltration, tend to be set back from the roadway at the toe of slope where space allows;

5. Basins and localized depressions – design incorporating basins or localized depressions, ranging in size from relatively small (with footprint less than 200 square feet) to larger (with footprint on the order of 1 acre or more), located at intervals along the roadway or in open spaces such as interchanges and other wide spots in the right-of-way (ROW).

Description of specific VRAs are provided in Section 4 of the Guidance Manual. Additional geotechnical design issues not presented in this White Paper may result from design features and infiltration mechanisms other than those stated above.

2 Assessment of Infiltration Feasibility in the Geotechnical Investigation and Design Process of Roadways

2.1 Overview

2.1.1 Goals of Infiltration Assessment in Planning and Design Phases

Successful stormwater infiltration inherently results in an increase in soil moisture in subsurface soil and/or rock and a potential rise in the local and/or regional groundwater table, which in turn may have geotechnical impacts on local features. For a successful project, these potential geotechnical impacts must be identified and considered during the planning phases, and fully evaluated (including design and evaluation of mitigation measures, if appropriate) during the design phases of the VRA.

Planning level design. At the planning level design phase of a project, information about the site may be limited, the proposed design features may be conceptual, and there may be an opportunity to adjust project plans to adapt to project goals, including volume reduction, if applicable. At this phase, geotechnical practitioners are typically responsible for conducting explorations of geologic conditions, performing preliminary analyses, and identifying particular aspects of design that require more detailed investigation at later phases. As part of this process, the role of a planning level infiltration feasibility assessment is to help planners reach early tentative conclusions regarding where infiltration is likely feasible, possibly feasible if done carefully, or clearly infeasible. This determination can help guide the design process by influencing project layout, informing the selection of VRAs, and identifying more detailed studies, if needed.

Detailed design. During the detailed design phase of project, geotechnical practitioners typically prepare detailed calculations and specifications based on the proposed design to support the preparation of final designs and construction documents. When a decision has been made to use VRAs that infiltrate stormwater, additional analyses may be required to be performed or existing geotechnical analyses may require modification. The purpose of design level infiltration analysis is to ensure that infiltration is incorporated appropriately, and as a byproduct, to refine planning level estimates of the feasible level of infiltration, as appropriate.

Guidance for geotechnical assessment of infiltration feasibility is provided in this document for both the planning level and detailed design phases. General guidance for each phase is provided in the sections that follow. Guidance on assessing and accounting for specific geotechnical hazards is provided in Section 3.

2.1.2 Consideration of Total versus Incremental Risk

In standard roadways designs, here are various pathways for water to enter the road base and shoulder material in the absence of intentional infiltration (e.g., seepage through cracks and joints in the pavement, lateral drainage). As such, a certain degree of wetting of base materials is commonly anticipated in assessment of material properties and the design of the roadway. Therefore, the risk posed by stormwater infiltration should be considered to be the incremental risk posed by the addition of greater quantities of water into the subsurface. Practically, this incentivizes stormwater designers to coordinate with geotechnical practitioners and pavement engineers to understand the soil moisture assumptions that are implicit in their standard design analysis. For example, where a standard design analysis includes an assumption of saturated subgrade to assess the risk of a certain form of failure, then the incremental risk posed by stormwater infiltration may be negligible.

2.2 General Guidance for Planning Level Feasibility Assessment

Potential geotechnical impacts from stormwater infiltration must be determined early in the planning phase to reduce the potential for late-stage design changes and unanticipated project costs. Fortunately, information commonly collected as part of project planning level investigations can be used as part of planning level geotechnical feasibility screening. The goal of the geotechnical assessment in the planning and feasibility phase is to identify potential geotechnical impacts and to determine which impacts may be considered fatal flaws and which impacts may be possible with design features to mitigate risks.

In order to identify and assess the potential geotechnical impacts, the designer must first understand the area of impact of the proposed stormwater infiltration system. The “impact area” is the area within which stormwater infiltration would have a non-negligible effect on geotechnical conditions. The extent of the impact area will depend on the type of infiltration system, volume and duration of infiltration anticipated, subsurface geology and other site-specific features. In general, the impact area can range from several feet to hundreds of feet or more and can extend beyond property boundaries and right-of-ways

To assess the area of impact, the designer may find it necessary to answer the following questions regarding the subsurface conditions:

- How deep is the water table?
- What are the typical subsurface soil or rock conditions?
- What is the permeability/hydraulic conductivity of the subsurface materials?
- Is subsurface water flow controlled by percolation, flow in joints, or along an aquitard?

The subsurface geology may have a significant impact on the determination of the area of impact. For example, if a high permeability sand layer is present above a deep groundwater table, the area of impact may be relatively limited because the infiltration would be generally vertical and the potential for groundwater mounding would be limited. In contrast, for conditions with lower permeability soils and a shallow groundwater table, the area of impact may be greater as the groundwater table would tend to mound to a greater degree and promote lateral migration of water. Other factors, such as dipping geologic strata (i.e., geologic layers that are inclined from horizontal), buried riverbeds, or jointing (i.e., directional cracks in rock layers) can have significant effects on the area of impact.

In addition to the hydrology of the subsurface, the designer must also consider other aspects of the geology and subsurface conditions to be able to access the types of potential geotechnical impacts that stormwater infiltration. The designer should be able to provide answers to the following questions, in addition to those above:

- What is the potential impact on the groundwater table from stormwater infiltration?
- Are there expansive or collapsible soils present?
- Are there compressible or liquefiable soils present?
- Are there any slopes nearby?
- What are the potential impacts to existing structures?

Further, the designer should catalog the existing infrastructure features within the potential impact area. This catalog should include utilities (present and abandoned), above- and underground structures, retaining walls and abutments, and hardscape and structure road subgrade features. The proximity to existing infrastructure and structures may help the designer identify early in the planning and feasibility phase areas where infiltration may be considered more favorable and areas where it has the potential to have significant geotechnical impacts.

The designer must consider potential impacts from stormwater infiltration as well as the risks associated with that impact. For example, is reduction of the factor of safety of a slope acceptable? Is potential settlement in a public park area acceptable? Is potential infiltration into subsurface structures acceptable and at what cost to repair or mitigate is the VRA worth incorporating? Are there regulations (local, state or federal) controlling the design – such as minimum factors of safety for slopes? While these questions may be more appropriately answered in the design phase, the considerations should be taken into account in the planning level phase.

Lastly, the VRA designer must be in communication with other design leaders on a development project to make sure the impacts of their design are taken into account in the early planning stages of concurrent designs. For example, if a new highway is proposed, the civil and geotechnical engineers must be aware of future planned VRAs located near slopes when determining the road alignment and designing the future cut or fill slopes.

Guidance for planning level assessment of specific geotechnical hazards is provided in Section 3.

2.3 Considerations in Design Phase Analyses

During the design phase, potential geotechnical impacts must be fully considered and evaluated and mitigation measures should be incorporated in the VRA design, as appropriate. In this context, mitigation measures refer to design features or assumptions intended to reduce risks associated with stormwater infiltration. While rules of thumb may be useful, if applied carefully, for the planning level phase, the analyses conducted in the detailed design phase require the involvement of a geotechnical professional familiar with the local conditions.

One of the first steps in the design phase should be determination if additional field and/or laboratory investigations are required (e.g., borings, test pits, laboratory or field testing) to further assess the geotechnical impacts of stormwater infiltration. As the design of infiltration systems are highly dependent on the subsurface conditions, coordination with the stormwater design team may be beneficial to limit duplicative efforts and costs.

Additional resources, such as design and as-built information regarding existing structures should be obtained, as appropriate. When representative conditions and input parameters are compiled, final design analyses and calculations can be performed to evaluate the geotechnical impacts. Design may also include evaluation of mitigation measures to reduce potential geotechnical impacts to acceptable levels, as appropriate.

Determination of acceptable risks and/or mitigation measures may involve adjacent land owners and/or utility operators, as well as coordination with other projects under planning or design in the project vicinity. Early involvement of potentially impacted parties is critical to avoid late-stage design changes and schedule delays and to reduce potential future liabilities.

3 Guidance for Evaluating Specific Geotechnical Hazards Associated with Stormwater Infiltration

This section gives an introduction to the types of geotechnical conditions and/or hazards that could be impacted by stormwater infiltration. This section is not intended to cover all possible conditions and additional regional- or site-specific conditions may need to be evaluated. Each section contains (1) an overview to introduce the key concepts and technical elements that underlie the potential hazard, (2) guidance for evaluating the potential hazard as part of the planning level feasibility analysis, and (3) considerations that should be reviewed and/or analyzed by the project geotechnical practitioner as part of designing to allow safe infiltration. Where available, rules of thumb are provided for planning level feasibility analyses; however these are not intended to replace the need for project-specific analysis and exercise of professional judgment by the project design team.

This White Paper is not intended to provide geotechnical guidance on design of VRAs or pavements. Designers should consider the impact of VRAs on the drainage and performance of roadway subgrade systems and may refer to NCHRP Report Nos. 499 or 583 (Hall and Correa, 2003; Hall and Crovetti, 2007) or other design references for guidance.

Section 4 provides greater information about the potential mitigation measures identified in this section, and Section 5 provides a summary of the relative risk posed by different failure mechanisms for different categories of stormwater infiltration facilities.

3.1 Utility Considerations

3.1.1 Overview

Utilities are either public or private infrastructure components that include underground pipelines and vaults (e.g., potable water, sewer, stormwater, gas or other pipelines), underground wires/conduit (e.g., telephone, cable, electrical) and above ground wiring and associated structures (e.g., electrical distribution and transmission lines). Below ground utilities are conveyed in trenches, typically located at depths of 1 to 10 feet below the ground surface and often within roadway right-of-ways. Culverts, storm drains, and sanitary sewers may exist well deeper than 10 feet where dictated by adjacent grades. This section will focus on underground utilities; impacts of stormwater infiltration on foundations for above ground structures and utilities are addressed in Sections 3.3 and 3.4.

Utility considerations are typically within the purview of a geotechnical site assessment and should be considered in assessing the feasibility of stormwater infiltration. Infiltration has the potential to damage subsurface utilities and/or underground utilities may pose geotechnical hazards in themselves when infiltrated water is introduced.

3.1.2 Planning Level Feasibility Screening Recommendations

At the planning phase, the designer should identify underground utilities (including abandoned, existing and proposed) within the area of impact. Impacts related to stormwater infiltration in the vicinity of underground utilities are not likely to cause a fatal flaw in the design, but the designer must be aware of the potential cost impacts to the design during the planning stage. The following paragraphs present typical impacts that stormwater infiltration may have on underground utilities.

When located within the area of impact of a stormwater infiltration facility, an underground utility trench has the ability to become a preferential pathway for the infiltrated stormwater. This can result in a larger than anticipated area of impact of stormwater infiltration that could lead to additional geotechnical or other types of impacts that were not considered. This is likely a concern if the utility trench backfill is more permeable (i.e., higher hydraulic conductivity) than the surrounding soil. The practice of bedding and shading the pipe or conduits with granular materials to reduce damage is a common practice during utility construction. If granular backfill (sand and gravel) is present, the infiltrated water may travel within and along the axis of the utility trench causing unanticipated flow patterns. This is particularly of concern when working in areas with older existing infrastructure.

Additionally, localized groundwater table increases may impact the underground conduits and underground vaults or manholes. Possible localized buoyancy of pipes may damage pipes and/or result in changes in pipeline gradients that could impact the effectiveness and flow rates of pipelines designed for gravity drainage. Localized increases in water table may impact significant buoyant forces on sealed underground vaults that could result in uplift of the vault. Differential uplift forces may impact the integrity of utility connections, such as conduits connecting to underground vaults. Further, underground vaults or access ports such as manholes may become submerged or flooded as a result of the increased groundwater table. This would impact the accessibility of the vault for maintenance and repair and may impact performance of aged electrical utilities with degraded coatings.

While not specifically a geotechnical concern, the presence of septic system leach fields within the area of impact should be identified early in the design process. As discussed in White Paper No. 2 (Appendix D), localized increases in groundwater tables may cause the groundwater table to intersect with septic leach fields reducing the effectiveness of leach fields and potentially contaminating groundwater. Lastly, many of our country's underground utilities are old and are in need of repair. The infiltration designer must consider the potential for aged utilities (specifically sanitary sewer or petroleum pipelines, pipes with gravel bedding that do not have cut off wall) to impact the water quality of the stormwater that may flow in the vicinity of the trench, increase the inflow and infiltration (I&I) into sanitary sewers, and/or create preferential pathways where water could migrate longitudinally and potentially cause issues such as sinkhole formation.

Local stormwater design manuals may provide rule of thumb guidance for setbacks from utilities. For example, it is common for a minimum setback of 100 feet from septic system drain fields to be observed (LACDWP 2011, WADOE 2012). These setbacks should be supplemented by site-specific information and professional judgment.

3.1.3 Design Considerations and Potential Mitigation Measures

During the design stage, the impacts of stormwater infiltration on utilities should be further evaluated. Design drawings and as-built records of utilities should be obtained and reviewed, if available. Detailed design efforts may include the following:

- Determination of likelihood of preferential flow within utility corridor based on comparison of permeability of existing trench backfill with typical subsurface soil/rock permeability. This may require sampling of subsurface materials for laboratory or field testing and in complex conditions detailed modeling of flow patterns;
- Calculation of uplift loads and associated factor of safety for uplift of underground vaults and evaluation of impacts resulting from potential uplift;
- Survey assessment of as-built gravity flow lines that could be impacted by uplift and calculation of pipe flow conditions in the event of uplift; and
- Determination of impacts to leach fields based on rises in the groundwater table.

Mitigation measures to control impacts to utilities primarily consist of methods to keep the potential impact area away from underground trenches or vaults, such as cut off walls/membranes running parallel to trenches, and measures to help prevent flow within the trenches, such as cutoff walls (low permeability backfill like concrete or bentonite grout) within the trenches.

Mitigation measures to reduce impacts of uplift on underground vaults may include anchors or adding additional weight to the vault, or measures that would limit the local rise in the groundwater table, such as deep infiltration or additional drains surrounding the vaults. These potential mitigation measures discussed further in Section 4.

3.2 Slope Stability

3.2.1 Overview

Infiltration of water has the potential to increase risk of slope failure of nearby slopes and this risk should be assessed as part of both the feasibility and design stages of a project. There are many factors that impact the stability of slopes, including, but not limited to, slope inclination, soil and unit weight and seepage forces. Increases in moisture content or rising of the water table in the vicinity of a slope, which may result from stormwater infiltration, have the potential to change the soil strength and unit weight and to add seepage forces to the slope, which in turn, may reduce the factor of safety of the stability of the slope.

3.2.2 Planning Level Feasibility Screening Recommendations

The first step in a planning level or feasibility assessment for slope stability impacts from stormwater infiltration is to identify existing or planned slopes that are located within the area of impact (see Section 2.2 for discussion of area of impact). For preliminary planning purposes, the designer may consider slopes as any area with a ground inclination steeper than approximately 20 percent or 5H:1V (horizontal:vertical). Typically slopes that are greater than 4 feet should be reviewed for geotechnical impacts, however, if potential impacts from movement or failure of a shorter slope are significant, they should be reviewed as well.

The designer should understand the typical subsurface conditions in the slope areas:

- How deep is the groundwater table?
- Are there existing seeps or springs in the slope?
- Are there joints or bedding layers in the slope that could be impacted by introduction of water?
- What are the subsurface conditions near the slope?
- Is the soil/rock prone to strength loss or weakening if wetted?
- Is the area prone to landslides?

A review of existing site conditions, including available geotechnical investigations for the area and published geologic maps, may indicate whether slope stability is an existing concern. Indications of slope movement may include presence of surface cracking or scarps at or near the crest of a slope and slumping at or near the toe of a slope. A review of geotechnical reports for development in the vicinity can also provide information regarding slope stability in the region and in particular subsurface conditions/formations. Further, a qualified geologist or geotechnical practitioner may review aerial photographs of the area to identify existing landslide features.

When evaluating the effect of infiltration on the design of a slope, the designer working with the geotechnical engineer must consider all types of potential slope failures, including but not limited to:

Figure 1: Slope slumping failure along roadway (Wikipedia)

- Deep seated failures – these failures are often rotational or block-like and the failure surface is typically at a depth below 5 feet;
- Toppling failures – these failures can result from movement along joint sets in subsurface soil or rock;
- Surficial slumping or debris slide– these failures are often in the surficial soils and often related to seepage along the slope face;
- Surficial erosion failures – these failures are often initiated by erosion of the slope surface and propagate due to over-steepened areas of erosion; and
- Creep/lateral fill extension – slopes which experience long term creep of fill soils down the slope;

The stability of a slope is generally evaluated by comparing the sum of the destabilizing forces and moments on a slope to the stabilizing forces and moments:

$$\text{Factor of Safety} = \frac{\text{Stabilizing Forces and Moments}}{\text{Destabilizing Forces and Moments)}}$$

A slope that has equal stabilizing and destabilizing forces has a factor of safety of 1.0 and failure is considered imminent. Slopes with factors of safety in the range of 1.1 to 1.3 may be considered stable for short term conditions but may experience minor to significant slope movement. Long term and permanent slopes are often designed for a minimum factor of safety of 1.5. While evaluating the effect of potential infiltration of the factor of safety, the designer working with the geotechnical engineer must determine what factor of safety would be considered acceptable. The acceptable factor of safety will be dependent on the duration of the impacts of the stormwater infiltration and the whether some slope displacement would be considered acceptable. For example, for conditions where settlement sensitive developments (i.e., buildings, hardscape, or utilities) are present at the top or toe of the slope, any measureable slope movement may be considered unacceptable. Whereas in undeveloped areas with no development near the slope, minor slope creep or surface erosion that does not lead to more significant failures may be acceptable. Local or state regulations may stipulate minimum factors of safety for various slope conditions.

Once an understanding of existing conditions is gained, the designer should evaluate, possibly in conjunction with a geotechnical practitioner, whether an increase in moisture content, seepage forces and/or rising of the water table may reduce the stability of the slope by:

- Softening of clay resulting in lower shear strengths and a reduction in the stabilizing forces/moments;
- Increasing soil unit weight thereby increasing the destabilizing forces/moments;
- Increasing potential for ice formation within joints/cracks resulting in increased destabilizing forces/moments;
- Increasing seepage forces within the slope, particularly seepage moving parallel to the slope face which increase destabilizing forces/moments; and
- Raising the groundwater table at the toe of the slope reduces the stabilizing forces/moments.

Several tools are available to provide simplified solutions for stability of a slope. All simplified methods should be used with caution as they do not take into account site-specific conditions or alternative mechanisms of failure. Simplified chart solutions for slope stability of homogenous slopes have been developed for use as a preliminary planning tool (Taylor, 1934; and Michalowski, 2002). Chart solutions for slopes with cohesive soil (clays and silts) as well as submerged slopes are available in design manuals such as NAVFAC (1986).

Local stormwater design manuals may provide rule of thumb guidance for maximum slopes suitable for infiltration systems and setbacks from slopes. For example, WADOE (2012) recommends a setback of at least 50 feet from slopes that are greater than 15 percent slope, while LACDPW (2011) recommends setbacks of at least 5 feet or half the height of the slope for any slope. Clearly, rules of thumb have not converged in all areas, and setback recommendations found in local guidance should be supplemented by site-specific information and professional judgment.

3.2.3 Design Considerations and Potential Mitigation Measures

In the design stage, detailed analyses of the infiltration impacts on slope stability should be performed by the geotechnical engineer as part of overall slope stability calculations. These analyses will likely include two-dimensional modeling with programs such as SLIDE (Rocscience, 2012) or SLOPE/W (Geo-Slope, 2012). To perform these analyses, the geotechnical professional needs a thorough understanding of the topography, subsurface conditions (soil stratigraphy, strength, and unit weight) and groundwater conditions. Additionally, the stormwater designer may need to provide the geotechnical practitioner with an estimate of the expected peak and long-term infiltration volumes.

The results of the detailed stability analyses should be evaluated with respect to allowable risks (see discussion in 3.2.2) and regulatory guidelines. Various federal, state and local agencies require minimum factors of safety as part of the grading permit approval process. If an acceptable factor of safety is not achieved considering the anticipated level of infiltration, mitigation measures may be considered. The most direct mitigation for slope stability is to limit the potential for water in the slope by providing a minimum setback from the top of the slope. Other options include encouraging deeper infiltration into the slope that extends below the depth of the calculated failure surface. Methods for encouraging deeper infiltration include french drains, dry wells, or wick drains. The effects of the deeper infiltration must be modeled for their impacts to the slope.

Other methods to increase slope stability may include overexcavating soft or weak layers in the slope subsurface that impact stability, decreasing the inclination of the slope, providing drainage, adding a soil buttress to the toe of the slope or inclusion of soil/rock anchors or tie-backs. Surficial stability may be increased by planting vegetation with a significant root mass at depth. These measures may allow infiltration to be accommodated while maintaining an acceptable factor of safety, however may add considerably to project costs.

3.2.4 Consideration of Existing versus Proposed Slopes

It is common for roadway projects to create new slopes via project grading activities or work in the vicinity of slopes that have been previously constructed by earlier projects. Three general categories of slopes are typically found in the highway environment: (1) natural slopes formed by natural topography that are not created or substantially modified by the roadway project or prior projects, (2) cut or fill slopes created by grading activities as part of a previous project, and (3) proposed cut or fill slopes that will be created by the excavation or placement of fill as part of roadway project. While slope stability analysis is necessary for each of these categories, investigation and analysis methods may differ.

Natural and existing slopes. For natural slopes and existing constructed slopes that will not be substantially modified, field exploration can establish baseline information about these slopes. In the absence of infiltration, a slope stability analysis may still be conducted to verify that slopes are stable, or a more approximate method may be used if it is clear that the slope is currently stable and will not be modified by the project. The addition of infiltration in the vicinity of these slopes may warrant a more detailed analysis than would otherwise be done. Additionally, modification of these slopes to accommodate infiltration may expand the project footprint and increase costs considerable and therefore may not be feasible.

Proposed slopes. For slopes that are proposed as part of the project, slope stability calculations are generally performed based on the project plans, the existing geologic characteristics, and characteristics of the anticipated fill material. For these slopes, the consideration of potential infiltration impacts may be incorporated as part of the design of the slopes in terms of a change in the design inputs, such as change in moisture content, unit weight (bulk density) or strength, and/or modification of other parameters. For fill slopes, the question of infiltration feasibility should be answered in terms of how much additional design cost would be required to maintain a stable slope with stormwater infiltration versus the case where only incidental infiltration is assumed. White Paper #1 (Appendix C) provides more guidance on infiltration into fill areas, including challenges associated with estimating infiltration rates, as well as reduction in infiltration capacity as a result of compaction.

For all types of slopes, the potential for surficial erosion should be considered in drainage design. If slopes are allowed to erode, either from lack of surface stabilization or from unstabilized drainage pathways, this can increase the potential for slope instabilities by creating weak spots in the slope face. However, this factor is inherent in all drainage design, regardless of whether stormwater infiltration is proposed or not.

3.3 Settlement and Volume Change

3.3.1 Overview

Settlement refers to the condition when soils decrease in volume. Heave refers to expansion of soils or increase in volume. Upon considering the impacts of an infiltration design, the designer must identify areas where soil settlement or heave is likely and whether these conditions would be unfavorable to existing or proposed features within the area of impact. Changes in volume, and particularly differential changes in volume, can result in the following impacts:

- Damage to pavement structures, sidewalks, and other rigid structures;
- Changes in surface drainage patterns;
- Reduction of structural integrity and/or serviceability of structures or retaining walls; and
- Impacts to utility gravity drainage and utility connections.

There are several different mechanisms that can induce volume change due to water infiltration that the designer must be aware of, including:

- Hydrocollapse and calcareous soils;
- Expansive soils;
- Frost heave;
- Consolidation;
- Dispersive soils and piping; and
- Liquefaction.

The following sections discuss these various mechanisms. Many of these forms of failure may already be evaluated in a standard roadway design process to evaluate the suitability of soils for subgrade and determine subgrade strength properties. Soils subject to volume change are typically not used in road base material or are remediated as part of construction. However, unremediated soils subject to volume change may still exist outside of the mainline roadway section, in areas where infiltration facilities are planned. Therefore, these forms of failure remain important for infiltration feasibility assessment.

3.3.2 Hydrocollapse and Calcareous Soils

Overview

Collapsible soils are typically loose and cemented soil and soils with low moisture content that may experience a large and sudden reduction in volume upon wetting. Calcareous soils are soils that have significant components of calcium carbonate or other salts (e.g., gypsum, calcite, halite). Calcareous soils are typically sedimentary and were deposited in a shallow marine environment. Grains of sand or clay are cemented together by the calcium carbonate. For both collapsible and calcareous soil, the introduction of moisture can dissolve or soften the cementation or structure of the soil, causing rapid and possibly extensive settlement.

Planning Level Feasibility Screening Recommendations

Impacts from soil collapse may include damage to hardscapes, utilities and foundations as well as changes in site drainage patterns that may lead to additional impacts. Early in the planning phase, the designer must identify potentially collapsible and calcareous soils as well as potentially impacted features, as settlement impacts can be significant and mitigations typically require intrusive actions that may not be feasible or cost-effective.

Collapsible soils tend to be geologically young and are often found in alluvial (water deposited), aeolian (wind deposited), and colluvial (gravity deposited) deposits. In addition, residual soils formed by extensive weathering of parent materials, such as weathered granite, can be loose collapsible soils. These soils are common in the upper 10 to 15 feet of the ground surface but can extend to depths greater than 100 feet. Calcareous soils are typically sedimentary and were deposited in a shallow marine environment. Grains of sand or clay are cemented together by the calcium carbonate or other salts.

If collapsible soils are present within the area of impact, a preliminary estimate of anticipated settlement should be performed based on available information such as existing geotechnical reports and typical soil behavior in the area.

Design Considerations and Potential Mitigation Measures

If collapsible soils may be left intact within areas where settlement would be considered undesirable, undisturbed samples of the soils may be taken and tested for collapse potential, with a test such as ASTM D4546 (ASTM, 2012a) to estimate the magnitude of the potential settlement. The vertical and lateral extent of the potentially collapsible soils should be investigated so a thorough understanding of the potential impacts of the introduction of water into the subsurface may be evaluated.

Options for mitigation of risks associated with collapsible and calcareous soils include prewetting of the soil prior to construction of settlement sensitive features, moisture conditioning and recompaction of the collapsible soils to break down the sensitive soil structure and treatment with chemical grouting (e.g., sodium silicate or calcium chloride solutions) to encourage cementation that is not significantly affected by water, or compaction grouting. To the detriment of infiltration feasibility, treatment may result in a significant reduction in soil permeability.

3.3.3 Expansive Soils

Overview

The designer must consider the presence of potentially expansive soils in and around structures and improvements when considering infiltration in design. Expansive soils are soils that experience volume changes with changes in moisture content. In particular, increases in moisture content result in expansion or swelling and decreases in moisture content result in shrinkage and cracking. The forces imparted by expansive soils can be large, causing significant differential movement/heave in hardscapes and structures. It is estimated that damage to pavements caused by expansive soils each year in the United States is in excess of $1 billion (USDOT FHWA, 2012). Expansive soils can generate pressures in excess of 20,000 pounds per square foot and swell to more than 10 times their initial volume (Colorado Geological Survey, 2012).

Planning Level Feasibility Screening Recommendations

The first step for the designer is to evaluate if expansive soils are present within the area of impact of stormwater infiltration facilities. In accordance with the International Building Code (IBC), (2012), expansive soils are typically defined as soils that:

- Have a plasticity index of 15 or greater, determined in accordance with ASTM D4318 (ASTM, 2012a);
- More than 10 percent of the soil particles pass a No. 200 sieve (75 mm), determined in accordance with ASTM D422 (ASTM, 2012a); and
- More than 10 percent of the particles are less than 5 mm in size, determined in accordance with ASTM D422 (ASTM 2012a); or
- Expansive index greater than 20, determined in accordance with ASTM D4829 (ASTM, 2012a) or AASHTO T 258.

Expansive soils usually contain the clay minerals montmorillonite (smectite) and/or kaolin and are typically clayey or have significant components of clay. Visual cracking in the soils or areas of extended ponded water are indications of the presence of expansive soil.

Figure 2: Desiccation cracking upon drying indicates likely presence of expansive soil conditions (http://www.geology.ar.gov/images/mudcracks.jpg)

Because of the presence of clay, expansive soils tend to have relatively low hydraulic conductivity and are not ideal for vertical infiltration of stormwater. The designer must consider the potential of lateral moisture migration and its effect on expansive soils. The magnitude of the expansion/shrinkage will depend on the mineralogy of the clay, chemistry of the water, and changes in moisture content.

Expansive soils exist throughout the United States but tend to be a more significant issue in the western states. Regional maps identifying areas of expansive soils have been prepared by NOAA and may be suitable for initial screening activities (NOAA, 1978). The designer should review any geologic data available in and near the project area for the presence of clays soils with significant clay fractions. The NRCS Web Soil Survey (http://websoilsurvey.sc.egov.usda.gov) may be useful to understand if expansive soils exist in the vicinity of the projects area, however this dataset should be confirmed with local observations, as it is not intended to be accurate at the site scale.

If expansive soil is present within the limit of impact, the designer must determine if features such as structures or hardscapes are present that may be damaged by the potential expansive and desiccation of these soils. The designer should also consider that deep foundations and retaining walls may be impacted by expansive soils at depth.

Design Considerations and Potential Mitigation Measures

If, during the planning level design, the designer identifies expansive soils within the area of impact that may cause undesirable impacts, the extent of the soils should be mapped and identified within the field. Laboratory testing, such as ASTM D4829 (Standard Test Method for Expansion Index of Soils), can be performed to determine the potential swelling pressure imparted by wetting of a site-specific soil. This can be useful in evaluating potential for swelling of expansive soils at various confining pressures.

Removal and replacement of potentially expansive soils can be one of the most effective methods to reduce swell hazards. However, this method may only be economical if the expansive soils are limited in area and/or thickness and their overexcavation will not impact existing improvements. Another possible mitigation measure includes limiting contact of infiltrated water with expansive soils. Methods for limiting contact could be installation of membrane barriers (such as asphalt membranes or geomembranes) to limit lateral moisture migration into zones with expansive soils or installation of drainage systems near foundations to limit the variation in moisture conditions. Prewetting of expansive soils prior to construction has seen limited success; however, this option only applies to areas where no existing features are in place that can be impacted by potential swell.

One of the most commonly used methods to stabilize expansive clays is admixing with a chemical such as lime (calcium oxide or calcium hydroxide). Lime treatment can be performed during construction where the expansive soil is mixed directly with lime (typically 2 to 6 percent by weight of soil) or post-treatment, where the lime is introduced by pressure injection, drilling or irrigation trenches. Lime treatment chemically decreases the expansion potential while also reducing the hydraulic conductivity of the soil.

If infiltration is proposed in areas with expansive soils, existing foundations and retaining walls should be analyzed for potential impacts from soil expansion. Horizontal swelling along a retaining wall can add significant destabilizing forces (see Section 3.4) that should be addressed. If unacceptable movements are predicted, foundations and walls may be able to be retroactively strengthened to limit damage from expansive forces.

3.3.4 Frost Heave/Thaw

Overview

Upward displacement of soil resulting from formation of ice in the subsurface, called frost heave, has the potential to cause significant damage to pavements, utilities, lightly loaded structures and the proposed VRA. In addition, the cyclic nature of frost heave/thaw cycles has the potential to substantially deteriorate roadway and subgrade layers leading to eventual damage and/or failure of roadways. Frost heave is considered a hazard when the following three conditions are met:

- Water is present in the subsurface;
- Frost susceptible soils are present; and
- Weather is cold enough to freeze.

Figure 3: Formation of subsurface ice lenses and resulting frost heave. (source: Wikipedia)

Upon freezing, water increases in volume by approximately 9 percent. This increase, while not insignificant, is not the primary mechanism of frost heave. When temperatures drop below freezing, ice lenses form in the subsurface. When capillary forces are great enough, which is commonly the case in fine-grained soils (silts and fine sands), moisture is drawn to the ice lenses which increase in volume, causing upward heave of the overlying soil. Heaving can often exceed several inches or more. Frost effects on foundations, which tend to result in permanent vertical displacements, tend to be smaller in magnitude than on pavements and hardscapes, but can result in significant cumulative damage over many seasons.

Planning Level Feasibility Screening Recommendations

Because the primary mechanism for frost heave is growth of ice lenses from capillarity, only soils with significant capillarity, such as soils with loam, silt, and clay components, are typically considered frost susceptible. Further, the growth of the ice lenses occurs by movement of water in the subsurface toward the ice lenses; soils with very low permeability may not allow significant water movement during the freezing conditions to experience significant capillarity. Silts are typically considered to be the most susceptible to frost heave; however, low plasticity clays, and silty or clayey sands and gravels also have potential for frost heave.

The first step for a designer is to determine if there is potential for frost heave within the area of impact by identifying if both frost weather conditions and frost susceptible soil type(s) are present. By nature of capillarity, the types of soils that are most susceptible to frost heave do not tend to provide the most desirable conditions to promote infiltration. For example, high permeability soils tend to be coarser grained with low capillarity and low potential for frost heave.

If the three criteria for frost heave are present (freezing weather conditions, frost susceptible soil, and a source of water), the designer should then determine the frost depth is in the vicinity of the project. Frost depth can be obtained from local building codes and can be estimated based on plots provided by NOAA (1978). The designer should also determine what features are located within the frost zone that may be negatively impacted by frost heave driven by an increase in subsurface water. If potential frost heave hazards are identified, the designer should evaluate potential impacts and mitigations as early in the design phase as possible, as effects from frost heave can be significant.

Design Considerations and Potential Mitigation Measures

If potential for frost heave is identified, detailed subsurface information and foundation design of all potentially impacted features should be obtained to evaluate which feature components are located within the frost zone and whether mitigation measures can be sufficiently incorporated.

On of the most effective ways to control frost heave is to design grades to limit the access of water to frost susceptible soils (i.e., the source of water is below the capillary range of the frost susceptible soils). This can be achieved by grading (e.g., providing a large elevation change between hardscapes and drainage features) or by designing for deep infiltration (infiltration wells).

For features that have not yet been constructed, frost heave damage can be limited by placing foundations at elevations below the frost depth or removal of isolated areas of frost susceptible soils (e.g., silt pockets). Additional measures such as providing good drainage around foundations limits the potential for formation of ice lenses below and around foundations.

To limit damage to hardscapes, layers of granular soil can be placed at or near the frost depth to provide a capillary break and limit the source of water for ice lens formation. Alternatively, for fill embankments not yet constructed, frost susceptible soils may be placed and compacted at depth, beneath the frost line to limit potential for frost heave.

3.3.5 Consolidation

Overview

Consolidation settlement occurs when loading causes water from the pore space between saturated soil particles to be squeezed out, resulting in soil volume reduction. Consolidation is typically induced by an increase in overburden or loading of the subsurface soil. This additional loading can be caused by placement of fill, construction of a structure, or by increases in the bulk density (resulting from an increase in moisture content) of the overlying soil stratum. Soils that are most susceptible to consolidation settlement are typically soft silts and clays.

Planning Level Feasibility Screening Recommendations

To determine if consolidation settlement may be a potential risk, the designer should determine if saturated soft sediments exist within the area of impact. This can be determined by review of available boring logs and geotechnical reports performed in the area. The designer must also evaluate if significant changes in moisture content of subsurface soils and/or whether there may be significant long-term changes in the groundwater table. Both of which could impact consolidation settlement. The rate of consolidation settlement is dependent on the hydraulic conductivity of the soil, which for silts and clays, tends to be quite low. Hence, consolidation settlement is not typically observed for interim or intermittent conditions. However, long term changes in moisture content or groundwater table elevation resulting from increased infiltration may result in long term settlement.

The magnitude of the settlement will be dependent on the thickness and compressibility of the compressible layer and degree and duration of the subsurface moisture content variations. Minor changes in soil unit weight are not anticipated to induce significant consolidation settlement. To illustrate this point, a long-term 5 percent increase in moisture content of a 10 foot thick soil layer overlying a 20 foot thick, saturated, soft clay layer may result in settlement on the order of 1 to 2 inches. This example was computed using some standard soil parameters. The reader is cautioned that there is considerable variation in the properties of fine-grained soil which correspondingly leads to large variations in settlement.

Typically for design, differential settlement has more negative impacts than uniform settlement. Differential settlements greater than approximately 0.1% to 0.4% (depending on structure type) are typically considered undesirable for structures (Day, 2010 [Table 7.2]). The designer must determine if localized infiltration has the potential to induce differential settlement greater than allowable levels. Standard soils text books such as Holtz et al. (2010) can provide typical values for consolidation ratios and initial void ratios that can be utilized in one-dimensional consolidation equations for preliminary estimation of settlement.

Design Considerations and Potential Mitigation Measures

If there is potential for unacceptable settlements resulting from consolidation, undisturbed soil samples should be taken within the compressible soil layer for testing in accordance with ASTM D2435 (ASTM, 2012a) or similar to determine existing soil conditions (e.g., void ratio, preconsolidation ratio, compressibility index). Alternatively, correlations to typical soil parameters (such as Atterberg Limits, ASTM D4813) may be used in lieu of additional testing. Further, the location of potential drainage layers (e.g., higher permeable layers to which the excess moisture can flow) should be estimated based on subsurface investigations (e.g., test pits, boring logs, cone penetrometer tests) to estimate the duration of the anticipated settlement, if appropriate. With an understanding of the subsurface strata, material parameters, and loading conditions, a geotechnical practitioner can then predict the anticipated settlement, differential settlement and duration of settlement.

If total or differential settlements are greater than allowable tolerances, the designer may consider reducing the potential infiltration volume per unit area to reduce the impact on the soil unit weight or and/or groundwater table. This may be accomplished by reducing diversion of stormwater into the infiltration system or increasing the infiltration area of the system or a combination of the two. If the settlement sensitive features have not been constructed yet, several options exist to limit future damage from settlement. Future foundations can be designed to accommodate the potential settlement by, for example, using a mat or raft type foundation that is not as sensitive to differential settlement. Another approach is to preload the compressible soil prior to construction of the settlement sensitive structure. However, effectiveness of preloading may be reduced if the preloading is performed prior to anticipated changes in groundwater levels

3.3.6 *Dispersive Soils and Piping*

Overview

Piping can be described as subsurface erosion and typically occurs in dispersive soils or fine-grained cohesionless soils (e.g., silts or fine-grained sands) which are overlain by at least slightly cohesive soils.

Planning Level Feasibility Screening Recommendations

Internal piping is a phenomenon when subsurface movement of water induces soil particle migration. Piping has the potential to result in subsurface voids in the form of pipes or fissures, which can lead to surface settlement and/or collapse. One of the most commonly known large-scale manifestations of piping is when underground utility pipelines break, causing rapid, uncontrolled movement of water and subsurface erosion or scour, resulting in development of a ‘sinkhole’ or collapse of the overlying soils into the resulting void space. Development of piping can also occur over a longer time scale – for example, subsurface pumping of water through a silt layer could result in significant and progressive fines migration, development of subsurface pipes/fissures and possible settlement. Fine-grained cohesionless soils such as silts and very fine sands, and dispersive soils are considered the most susceptible to piping.

Figure 4: Road damage from a sinkhole in Colorado. Photo by Jon White. (http://geosurvey.state.co.us/hazards/Collapsible%20Soils/Pages/DispersiveSoils.aspx)

Dispersive soils contain clay particles that typically have a higher content of dissolved pore water sodium and upon wetting, disperse into solution. Dispersive soils cannot be identified by standard index properties. Common tests to identify dispersive soils are the pinhole test (ASTM D4647) and the double hydrometer test (ASTM D4221).

To determine if the project may be impacted by piping, the planner should determine if soils susceptible to piping are likely present, and should evaluate subsurface water gradients. Piping typically occurs where subsurface water gradients are moderate to significant and can mobilize movement of soil particles. While the factor of safety with respect to piping by subsurface erosion cannot be evaluated with practical means (Terzaghi, Peck and Mesri 1996), risks for development of piping failures can typically be reduced by providing proper filtration design in areas of subsurface gradients and by attention to detail in the design of areas of water collection and diversion.

Design Considerations and Potential Mitigation Measures

Careful design and control of subsurface flow is critical to preventing piping failures. In areas of soil transitions where subsurface flow is anticipated, a designer may include filter fabric(s) or soil filter(s) to reduce particle migration. Additionally, methods to decrease subsurface gradients may be utilized – such as increasing the flow path. Dispersive soils can be treated with lime to reduce their potential for particle suspension. The designer may also incorporate features such as anti-seep collars around piping inlets to control unanticipated flows and reduce the potential for subsurface scour.

3.3.7 Liquefaction

Overview

Liquefaction is a process by which saturated sediments temporarily lose strength and act like a fluid when exposed to rapid, cyclical loading conditions, such as an earthquake. This loss of strength can result in loss in foundation support, lateral spreading (see Figure 5), floating of underground buried tanks and utilities, slope failures, surface subsidence and cracking, and development of sand boils.

In order for liquefaction to occur, the soil must typically be:

- Saturated;
- Loose to medium dense sandy soil and fine-grained soil with a plasticity index (PI) less than 12 Bray and Sancio, 2006); and
- In a region with potential to experience rapid loading conditions (i.e., earthquakes).

The potential for stormwater infiltration to increase the risk of liquefaction hazards exists if the proposed design would increase the water table to elevations that include liquefaction susceptible soils. A change in moisture content of soils, below saturation, does not present a risk of liquefaction.

Planning Level Feasibility Screening Recommendations

Figure 5: Roadway damaged from lateral spreading in 1989 Loma Prieta earthquake (from Nakata et al., 1990)

During the feasibility evaluation of potential stormwater infiltration sites, the designer should evaluate if the potential sites are located within liquefaction susceptible zones. As discussed above, this can be evaluated be determining if the three general criteria for liquefaction are present. The United States Geological Survey (USGS) provides resources and maps that designate liquefaction susceptibility or potential in various subregions of the United States; however, comprehensive maps of all areas are not available at this time. Other sources for liquefaction susceptibility maps may be local planning agencies.

By reviewing available subsurface data, the designer may determine if sandy or silty soils are present below or near the groundwater table. Standard Penetration Tests (SPTs) performed during sampling of subsurface soils provide a general indication of the density of in-situ soils; typically sandy or silty soils with a corrected SPT blow count (N_{1-60}) greater than 30 are not typically liquefiable. Further, surface evidence of soil liquefaction typically only results from liquefaction of soils in the upper 15 meters (50 feet) from the ground surface (Idriss and Boulanger, 2008).

The designer must also evaluate if the proposed infiltration system has the likelihood of increasing the ground water table in the area. This will be dependent on the type of infiltration approach proposed, duration and volume of infiltration, and local geologic conditions (see Section 2.2).

More information regarding liquefaction can be found in Idriss and Boulanger (2008) or Kramer (1996).

Design Considerations and Potential Mitigation Measures

If the preliminary feasibility assessment indicates increased liquefaction potential at a site as a result of stormwater infiltration, the project geotechnical professional should perform a liquefaction analysis for the proposed conditions and project seismic design criteria. This analysis will assess the risks for liquefaction and potential for lateral spreading resulting from the proposed design. The geotechnical professional will assess the proposed groundwater level, design maximum ground acceleration, and soil conditions (unit weight, density, and soil type). At this point, the project team should evaluate whether the increased risks resulting from the infiltration system are considered acceptable based on the return period for the potential liquefaction-triggering earthquake, and the consequences of liquefaction.

Liquefaction susceptibility can be mitigated by densification or removal of loose sediments, or by infiltrating into deeper soil horizons that are less susceptible to liquefaction. Methods used to densify existing soils include overexcavation and recompaction, jet grouting, deep dynamic compaction (DDC), injection grouting, or stone columns. These approaches may reduce the hydraulic conductivity of the soils. Alternatively, infiltration systems can potentially be designed with drainage trenches or barriers to avoid saturation of liquefiable soils.

3.4 Retaining Walls and Foundations

3.4.1 Overview

Retaining walls, including basement walls and bridge abutments, are common features within or in close proximity to urban roadways. These structures are designed to withstand the forces of the earth they are retaining and other surface loading conditions such as nearby structures. Foundations include shallow foundations (spread and strip footings, mats) and deep foundations (piles, piers) and are designed to support overburden and design loads. All types of retaining walls and foundations can be impacted by increased water infiltration into the subsurface as a result of potential increases in lateral pressures and potential reductions in soil strength.

3.4.2 Planning Level Feasibility Screening Recommendations

Many urban highways are located in areas of dense development with frequent overpass structures, bridge abutments, and retaining walls, as well as buildings in close proximity to the right-of-way. Designers should identify foundations and retaining walls within the area of impact of stormwater facilities. For preliminary screening purposes, a horizontal setback equal to one to two times the depth of the foundation or the height of a retaining wall may be assumed to identify soils that may affect the foundation/wall.

Increases in moisture content of subsurface soil and in the groundwater table have the potential to reduce the factor of safety of these features. Similar to the calculation of factor of safety for slope stability, the factor of safety of a foundation or retaining wall is determined by comparing the stabilizing forces and moments to the destabilizing forces and moments. A factor of safety of 1.0 corresponds to a wall or foundation where failure is imminent. Typical minimum factors of safety for walls and foundations vary based on the failure mechanism but may range from 1.5 to greater than 3.0. The designer should understand the primary mechanisms in which increased infiltration can impact foundation or wall stability:

- Addition of water may reduce the strength of clay soils (clay softening), decreasing the stabilizing forces/moments;
- Addition of water increases the unit weight of soil being retained, increasing the destabilizing forces/moments and potentially causing infiltration into subsurface structures; and
- Rise in the groundwater table increases hydrostatic pressure on a wall or foundation, increasing destabilizing forces/moments, possibly decreasing the stabilizing forces of the soil (by reducing effective stresses), and potentially causing infiltration into subsurface structures;

If reductions in stabilizing forces/moments and/or increases in destabilizing forces/moments for a wall or foundation are potentially significant, a thorough investigation of the impact to the specific design of a feature is warranted. This may require an understanding of the design loads on the feature and as-built conditions (including dimensions, soil backfill conditions, concrete reinforcement, anchors or tie-backs, if applicable for retaining walls). Reductions in factor of safety can result in movement of foundations and retaining walls, and if great enough, failure. Movement can result in differential settlements which can impact the serviceability of structures and hardscapes. For example, if foundations on one side of an abutment were embedded in clay which was softened by significant increases in infiltration, settlement on one side of the abutment may occur. This would result in differential settlement that could result in cracking of structural elements and overall decrease in the structural integrity and/or serviceability of the structure.

Guidance for simplified calculation of foundation bearing capacity and retaining walls are provided in textbooks such as Bowles (2001) or circulars from the Federal Highway Administration (FHWA, 1996, 2002) that may be utilized to gain a general understanding of how changes to moisture conditions and groundwater tables may impact foundation and retaining wall stability. Stormwater planners and designers should contact a geotechnical practitioner to understand site-specific issues and potential soil strength impacts.

3.4.3 Design Considerations and Potential Mitigation Measures

If reductions in stabilizing forces/moments and/or increases in destabilizing forces/moments for a wall or foundation are potentially significant, a thorough investigation of the impact to the specific design of a feature is warranted by a geotechnical and, if appropriate, a structural practitioner. This may require an understanding of the design loads on the feature and as-built conditions (including dimensions, soil backfill conditions, drainage features, concrete reinforcement, and anchors, struts or tie-backs, if applicable for retaining walls).

The primary mitigation measure to reduce the impact of subsurface infiltration on foundations and retaining walls is to limit the area of impact away from the feature. This can be accomplished by design features with appropriate setbacks and/or by providing drainage behind retaining walls and near foundations to limit subsurface water in the vicinity of the feature. Drains can be incorporated to design of existing future features or retroactively constructed. Additional measures include addition of struts, anchors, soil nails, or tie-backs for retaining walls to increase the resisting forces of the wall. Foundations can be stiffened to accommodate differential settlement or foundation subgrades can be strengthened with procedures such as jet grouting to increase bearing strength and reduce potential settlements.

3.5 Pavement Impacts

3.5.1 Overview

One of the most prevalent causes of damage to pavements is insufficient drainage and/or excessive moisture in the pavement section and subgrade. Excess moisture commonly results in pavement damage such as rutting, bumps, depressions, potholes, fatigue cracking, roughness, etc. Many of the mechanisms for this damage have been discussed in the Section 3.3 (e.g., settlement and volume change resulting from issues such as hydrocollapse, expansive soils, consolidation, dispersive soils, and frost heave). Additional mechanisms for pavement damage resulting from excessive moisture include subgrade softening, variability in pavement properties, and fines migration.

3.5.2 Planning Level Feasibility Screening Recommendations

Pavement design in accordance with NCHRP 1-37A (A.R.A., Inc., 2004) is dependent on infiltration and drainage system design inputs, including volume and rate of infiltration, drainage system quality, and drainage path length. VRA designers should incorporate pavement engineers on the design team to evaluate the potential impacts on nearby roadways early in the project. In the case of retrofitted sites, introduction of moisture in the subgrade may significantly reduce the serviceability of the pavement. For new roadways, anticipated increases in moisture may require thicker pavement sections as well as additional drainage features. The designer should determine if the VRA has the potential to impact moisture conditions within the pavement section:

- Are existing roadways in the area exhibiting signs of moisture-related damage?
- Will the VRA increase the demand on edge drains in the roadway shoulders beyond acceptable levels?
- Will the VRA increase moisture conditions beneath the pavement, with lateral flow or increases in groundwater elevation?
- Are pavement section materials, including base, subbase and subgrade sensitive to moisture variations?
- Will increased moisture within the pavement section increase the potential of fines migration into or within the pavement section?

One of the primary design factors for pavement design is modulus or stiffness of the pavement system components. Increases in moisture can significantly reduce this modulus, particularly in soils with considerable fines (silt and clay particles), resulting in loss of pavement support. Studies indicate that modulus reductions in unbound aggregate base and subbase as well saturated fine-grained subgrades can be more than 50% (FHWA, 2006; AASTHO, 1993). Localized changes in moisture conditions can result in non-uniform subgrade conditions, which is a common cause for pavement damage such as roughness or fatigue cracking. Movement of water through the subbase/base and subgrade with improper filtration systems (such as soil or geotextile filters) can result in clogging of drainage systems and development of voids and localized loss of foundation support.

3.5.3 Design Considerations and Potential Mitigation Measures

When infiltration is being proposed in the vicinity of pavement, but not directly into pavement, the most efficient way of limiting moisture impacts on pavement systems is to reduce the likelihood of moisture migration into the pavement section. Water typically enters the pavement system by 1) capillarity and groundwater, 2) migration from roadway shoulders, and 3) infiltration through the pavement section, typically through cracks and joints. Mitigating the potential impacts for increased moisture can be achieved by controlling these sources of water, with enhanced roadway maintenance (sealing of cracks and joints) and increased drainage systems, both along the shoulder and within the pavement sections. Drainage systems should include edge drains to collect and remove drainage from the pavement system and to limit migration of water from the shoulder to the pavement section. Pavement engineers should also consider the use of free-draining base layers (with separator layers), interceptor drains (to limit run-on), and underdrains (to control groundwater and capillarity). More information on impacts on pavement design can be found in numerous publications by A.R.A., Inc. (2004), AASHTO (1998) or FHWA (2006).

However, in some cases (such as permeable shoulders), infiltration into the pavement section is part of the design. In this case, the primary mitigation methods available to designers include utilizing materials that are less sensitive to moisture variability (such as coarse grained materials with limited fines or cement treated materials) and/or developing pavement designs that are resilient to elevated moisture conditions (e.g., by increasing the depth of the base layers or using asphalt treated permeable base layer). In some cases, the additional depth of base layer needed for hydrologic design purposes can provide the additional strength needed to compensate for elevated moisture conditions in the subgrade material. White paper #4 (Appendix F) provides further discussion of permeable pavement design, including design to meet both structural and hydrologic design requirements.

4 Potential Mitigation Measures for Elevated Design Risk

Once a designer has identified the potential geotechnical impacts from an infiltration design, a range of possible mitigation measures can be considered to reduce the impact to acceptable levels. When considering potential mitigation measures, factors include:

- Technical feasibility – can the risk be adequately mitigated within the constraints of the site?
- Cost – Does the cost of mitigating potential impacts add considerably to the cost of the project? Is this cost increase justified by the increased level of runoff volume reduction that can be achieved?
- Public perception and affected parties - Is the area that would be used to mitigate a risk (i.e., for example, building soil buttress to mitigate slope stability risk) owned by another party? Does the project have access to this area? Is there perception of risk that cannot be mitigated?

Several types of approaches can be incorporated into a design to reduce the potential for geotechnical impacts from an infiltration design. These mitigations generally involve one of the following strategies: a) limiting the area of impact of the infiltration design, b) removing or reducing the geotechnical risk factor, or c) modification in the design of potentially impacted features.

4.1 Limit Area of Impact / Effective Site Design

By limiting the area of impact of an infiltration design, the increased moisture and/or effect on the groundwater table is limited and therefore, the geotechnical impacts are limited. For example, if a mitigation measure is incorporated to keep the infiltrated water away from a utility trench, the potential for flow in the trench is substantially removed. Mitigation options to reduce the impact include:

- Cut off walls/curtains;
- Subsurface drains;
- Setbacks from sensitive features; and
- Targeted infiltration locations.

A cut off wall or curtain is a relatively low permeable barrier that limits the amount of vertical or horizontal flow of groundwater. A cut off wall could take many forms depending on the application and the nearby sensitive feature. For example, a geomembrane (low permeability synthetic membrane barrier) may be placed in a trench upstream of a sensitive utility corridor. The geomembrane would likely be extended below the depth of the utility corridor and would encourage vertical infiltration below the depth of the utility trench and limit horizontal migration of water into the utility trench. An alternative may include excavation of portions of the utility trench backfill on regular intervals and backfilling with low permeable material (e.g., concrete, grout or clay) to limit flow along the trench. More costly options such as deeper slurry walls or sheet pile walls can be utilized. This type of barrier approach may also be suitable to limit lateral moisture movement toward collapsible or expansive soils or areas where frost heave may cause unacceptable soil movement.

Additional drainage features may provide another method to reduce the potential for infiltrated water to impact sensitive features. For example, a subsurface drain could be installed upgradient of a retaining wall or slope that would intercept subsurface moisture and direct it away from the sensitive feature.

Setbacks can be incorporated into site design and the selection of locations for infiltration systems such that the potential for impacts are reduced. For example, an option would be to limit infiltration to areas a minimum specified distance from the crest of a slope or from an underground structure/vault, foundation or retaining wall. This would reduce the amount of infiltration in the vicinity of the sensitive feature and therefore reduce the potential impacts. The setback distance will depend on the quantity and duration of infiltration and design considerations for the sensitive feature. When infiltration can be considered early in the process of laying out the project, it may be possible to identify key areas for infiltration that observe necessary setbacks so drainage can then be routed to these suitable areas. Site design approaches to mitigate infiltration risks are generally more applicable for new projects than for lane-addition projects and retrofits.

In cases where increases in moisture near the ground surface or at specific depths may result in an unsatisfactory impact, the designer may encourage infiltration at deeper or at specific depths by installing features such as french drains, wick drains or dry wells. These types of features provide preferential vertical drainage paths that allow water to reach deeper soil layers. This may be an option to reduce the potential for frost heave by limiting a source of water near the surface for ice lens growth or to reduce potential surficial stability issues. In addition, by providing surface grading that directs flows away from structures, infiltration zones can be targeted that may reduce impacts on nearby structures.

4.2 Remove or Reduce the Geotechnical Risk Factor

By removing or reducing the geotechnical risk factor, the potential for negative impacts from an infiltration type design are inherently reduced. In particular, this category of mitigation measure reduces the potential impacts by removing the geotechnical component that may cause the impact, such as:

- Prewetting or moisture conditioning and recompaction of collapsible soils;
- Removal and replacement of expansive soils;
- Lime treatment of expansive soils;
- Removal of frost susceptible soil to reduce risk of frost heave;
- Densification of potentially liquefiable soils;
- Overexcavation of soft of weak soil layers beneath foundations and slopes; and
- Utilization of bound materials (e.g., cement treated) in pavement section.

Many of these approaches may already be conducted as part of a conventional project to account for incidental infiltration that may occur. In considering the effectiveness of these practices for improving infiltration feasibility, consideration should also be given to negative impacts on infiltration rates that may result. For example, it does not make sense to recompact collapsible soils to allow for stormwater infiltration if the result would be a reduction in infiltration rate to the point where achievable infiltration does not meet project goals.

4.3 Design of Features to Incorporate Infiltration

By either retroactively or proactively designing structures or other features to accommodate increased infiltration, undesirable geotechnical impacts can be minimized. Retroactive approaches would apply to existing structures, slopes, utilities, retaining walls, and other features that were previously constructed without consideration of stormwater infiltration and would be potentially influenced by the addition of stormwater infiltration. Proactive approaches would apply to the design of features that are to be constructed.

Retroactively, mitigation designs may include features such as:

- Waterproofing of subsurface structures to limit seepage/infiltration;
- Addition of tie-back anchors, soil nails, or struts to provide additional support to counteract hydrostatic forces on retaining walls;
- Adding anchors or additional weight to counteract buoyant forces on subsurface utilities or vaults;
- Constructing drainage behind retaining walls to reduce hydrostatic forces on the wall;
- Increase capacity of drainage systems along roadway shoulders to reduce potential migration of water to pavement system;
- Modify slope inclination, add soil buttresses, rock anchors/tie-backs or drainage features to increase slope stability; and
- Increase deep rooted vegetation on surficial slopes to reduce potential of surficial slope failures.

Proactively, structures or features that may be designed concurrently or after the infiltration design may include the following in their design:

- Preloading areas of soft sediments to induce consolidation settlements in advance of settlement sensitive structure construction;
- Assuming increased moisture content or modified groundwater conditions in design of slopes, retaining walls and foundations;
- Setting structure foundations below the frost line;
- Providing a capillary break beneath frost susceptible soils;
- Providing significant vertical separation of drainage features from frost susceptible soils;
- Placement of frost susceptible soils at depth in fill embankment;
- Providing redundant drainage features;
- Accommodating potential for differential settlement resulting from fluctuating moisture conditions in the subsurface by stiffening foundations and walls;
- Designing pavement section to account for elevated moisture, and
- Utilization of bound materials (e.g., asphalt treated permeable base, cement treated materials) in pavement section.

The feasibility of these approaches is expected to vary greatly on a site-by-site basis, and should be evaluated using "what if" scenarios based on site-specific information. Not all mitigation measures may be physically or economically feasible.

5 Summary of Implications for Volume Reduction Design

Achieving volume reduction via infiltration of stormwater inherently introduces a greater quantity of water into the subsurface geology than would otherwise occur. This has ramifications at each phase of project design. Figure 6 below summarizes a general sequence for incorporating stormwater infiltration into project design and identifies key questions that may need to be answered at each project phase. Because additional information is obtained through this process, it may be necessary to iterate between steps for goal refinement (i.e., what can be safely and practicably achieved) and site design (i.e., where should infiltration be sited).

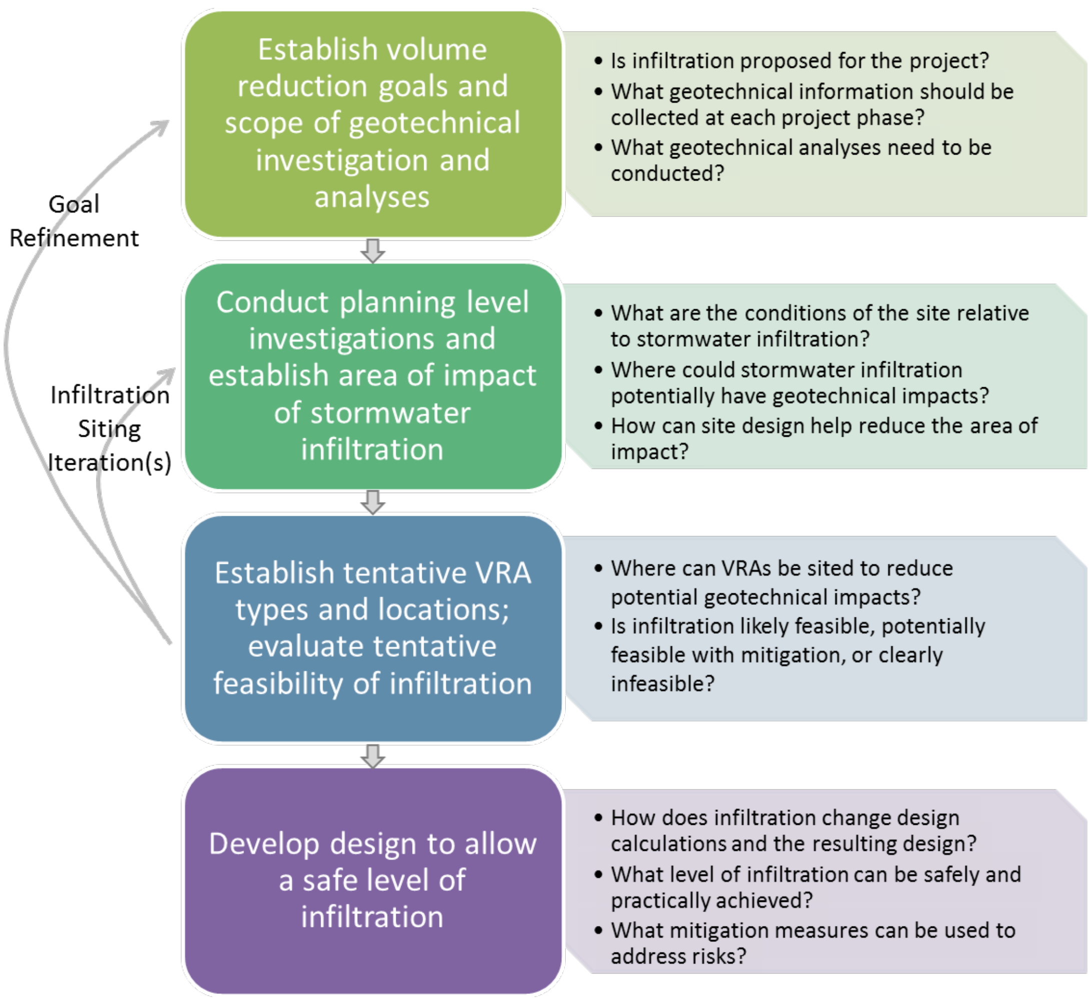

Figure 6. Example Approach for Including Stormwater Infiltration in the Geotechnical Design Process

Table 1 provides a summary of the indicators of elevated risk and potential design implications associated with each category of geotechnical hazard identified in Section 3. Table 2 provides summary of potential opportunities and constraints for specific categories of VRAs related to geotechnical considerations. The guidance in these tables is intended to provide a brief summary and synthesis of the information presented in Section 3 and 4 and is not intended to replace the need for sound engineering judgment based on project-specific data.

Table 1. Summary of Geotechnical Considerations

Geotechnical Hazard Category	Example Indicators of Elevated Risk[1]	Example Design Implications[1]
Utility Considerations	• Presence of utilities in ROW • Historic infrastructure • Underground vaults below groundwater table • Permeable backfill in trenches	• For existing and proposed utilities: Allow adequate setbacks or otherwise control area of impact and/or limit flow within utility corridors with cut off walls, • For proposed utilities: Design utilities to allow for infiltration, if needed
Slope Stability	• Presence of slopes greater than 20 percent (5V:1H), or otherwise potentially affected • Highway sections on embankment • Soil strength is sensitive to water content • Instability observed in adjacent areas	• Avoid infiltration near slopes • Provide features, such as cutoff walls or drainage systems, to control lateral migration near slopes, if needed • For proposed slopes, may be possible to allow for infiltration in design assumptions; incidental infiltration may already be assumed in standard calculations
Settlement and Volume Change		
Hydrocollapse and calcareous soils	• Younger alluvial, aeolian or colluvial soils • Soils with calcium carbonate cementation • History of hydrocollapse/calcareous soils in area	• Investigation and remediation typically part of standard highway design within the roadway footprint • Remedial options, including prewetting or compaction may reduce infiltration rates
Expansive soils	• Typically associated with certain types of clays	• Investigation and remediation typically part of standard highway design • Ability to remediate with removal and/or lime treatment may depend on depth and extent of expansive soils
Frost heave	Each of the following present: • Water content near surface, • Frost susceptible soils (soils with high capillarity and adequate permeability (typically silts and loams) • Freezing weather conditions	• Investigation and remediation typically part of standard highway design • May be limited by setting infiltration surface well below ground surface
Consolidation	• Saturated soft sediments, and • Potential for significant increase in weight of surface layer or elevation of groundwater	• Design to allow settlement • Distributed infiltration more evenly to result in lower increase in weight per unit area

Geotechnical Hazard Category	Example Indicators of Elevated Risk[1]	Example Design Implications[1]
Dispersive Soils and Piping	• Moderate to high subsurface gradients • Dispersive soils or fine-grained cohesionless soils	• Design filtration systems to reduce subsurface erosion • Reduce subsurface gradients • Lime treatment of dispersive soils
Liquefaction	• Saturated, loose to medium dense sandy and silty soils, and • Rapid cyclical loadings (earthquakes)	• Design to eliminate one or more of the three key risk factors • Design to balance consequence of failure versus probability of earthquake
Retaining Walls and Foundations	• Presence of retaining walls or foundations within influence area • Finer grained soils with bearing strength sensitive to moisture content • Potential for significant increase in weight of surface layer or elevation of groundwater	• Avoid infiltration near retaining walls and foundations • Provide features, such as cutoff walls or drains, to control lateral migration, if needed • For proposed features, may be possible to allow for infiltration in design assumptions; incidental infiltration may already be assumed in standard calculations
Pavement Impacts	• Moisture sensitive base, subbase, or subgrade materials • Insufficient drainage systems to limit water contact with pavement system • Poorly draining base or subbase	• Increased pavement section thickness to accommodate reduced subgrade modulus • Design of free draining base, subbase layers • Increased maintenance requirements • Inclusion of filter in pavement design to limit fines migration

1 – Examples provided to identify typical indicators of risk and possible design implications. Additional risk factors and design implications may be present based on site-specific conditions. More information regarding risk indicators, design implications and potential mitigation measures is provided in Section 3.

Table 2. Summary of Potential Opportunities and Constraints for Specific Categories of VRAs

Category of VRA	Characteristic Properties	Example Opportunities and Constraints related to Geotechnical Issues1	
		Opportunities	**Constraints**
Direct infiltration into roadway subgrade	• Broad footprint; may only receive direct rainfall or equivalent • Road subgrade has important structural considerations, particularly for flexible pavement design	• Broad footprint may allow infiltration in relatively dense soils • Standard roadway designs typically account for wetting of subgrade • Rigid pavement design (i.e., concrete) less sensitive to strength of subgrade	• Utilities in ROW • Settlement and volume change could damage roadway • Reduction in strength of subgrade material may render infeasible or require higher construction costs
Infiltration in shoulders	• Outside of main travel lanes; significantly less loading • Smaller footprint; more concentrated zone of infiltration	• Designed to accommodate less loading or no loading • Well-distributed inflow • Can have moderate to high tributary area ratio[2] • Linear configuration less susceptible to groundwater mounding than basin configurations • Underdrain with outlet can control amount of water infiltrated	• Typically shoulder must be compacted to same degree as mainline roadway • Potential for water to migrate laterally into mainline subgrade rock or nearby development • Settlement or volume change could lead to damage to roadway • Potential reduction in slope stability for embankment or depressed sections
Compost amended slopes/filter strips adjacent to roadway	• Allows incidental infiltration over relatively broad area; also provides ET • Typically coupled with vegetated conveyance at toe of filter strip	• Drainage over shoulder is a typical design feature • Compost amended results in relatively limited increase in infiltration compared to standard design • Higher proportion of losses to ET than other VRAs	• May lead to erosion issues if applied on slopes that are too steep • Slopes may need to be compacted to same degree as mainline roadway • In some cases, settling or volume change could damage roadway

Category of VRA	Characteristic Properties	Example Opportunities and Constraints related to Geotechnical Issues1	
		Opportunities	**Constraints**
Channels, trenches, and other linear depressions offset parallel to roadway	• Tends to be located 10 or more feet from travel lanes • Typically effective water storage depth is between 6 inches and 36 inches • Tributary area ratio may be low • May be fully or partially infiltrated	• Channels with positive grade are common drainage features; have relatively limited increase in risk • Due to horizontal separation, features have less potential to damage roadway • Some settlement may be tolerable	• Greater potential for impacts out of ROW due to proximity to ROW line. • Greater potential for mounding due to concentration of infiltrating footprint. • May reduce stability of slopes if located near top or toe.
Basins and localized depressions	• Typically located in more centralized locations • Typically have a relatively low tributary area ratio • Typically effective water storage depth is between 12 inches and 60 inches	• Centralized areas, such as wide spots in ROW or interchanges may allow ample setbacks from foundations, slopes, and structural fill • May be possible to preserve natural soil infiltration rates through construction • Impacts of potential settlement may be minor	• Broad footprints and deeper ponding depths may result in substantial groundwater mounding and lateral water migration in some cases which may impact settlement, slope stability and nearby foundations or retaining walls. • Due to more concentrated flows from large tributary area, greater setbacks may be needed than would be applied for more distributed systems.

1 – Examples provided to identify typical opportunities and constraints of the infiltration design feature. Additional opportunities and constraints may be present based on site-specific conditions. More information regarding risk indicators, design implications and potential mitigation measures is provided in Section 3.

2 – Tributary area ratio refers to is the ratio of the infiltrating surface to the tributary area. A high tributary area ratio indicates that the infiltrating footprint makes up a large portion of the total tributary area; and vice versa.

6 Recommended Contents of Geotechnical Infiltration Feasibility and Design Reports

A "Geotechnical Infiltration Feasibility and Design Report" or equivalent is recommended to summarize the geotechnical feasibility of stormwater infiltration and associated design parameters. The exact contents of this report may vary as a function of project type, site conditions and associated conditions of the concern, regulatory context, and agency preference. However, the key underlying questions are generally similar:

Feasibility screening:

- Where within the project site do conditions potentially allow infiltration to be used?
- To what degree is infiltration potentially feasible in these areas? Potential findings may be that is potentially feasible to infiltrate the full design volume or a portion of the design volume (or an incidental amount of volume); or that it is infeasible to infiltrate any volume. See discussion of Category 1, 2, and 3 VRAs condition in Section 5.2of the main body of the Guidance Manual.

Design analysis.

- For locations where infiltration is proposed, what design infiltration rates should be used?
- What design elements (i.e., modified design parameters, mitigation measures) are recommended to be included in designs to safely allow infiltration to occur in these locations?

The Geotechnical Infiltration Feasibility and Design Report should address each of these questions, as appropriate for site conditions and the phase of the project. Potential elements of the report that may be relevant to address these questions include, but are not limited to:

- Location and area of influence of stormwater infiltration systems
- Depth to the seasonally high water table
- Typical subsurface soil or rock conditions, including bore logs and/or results of other investigations
- Permeability/hydraulic conductivity of the subsurface materials, including results of testing, as appropriate (See Appendix C)
- Controlling factors in subsurface water flow (e.g., percolation, flow in joints, or along an limiting layer or aquitard)
- Potential impact on the local groundwater table from stormwater infiltration (i.e., groundwater mounding)
- Recommended design infiltration rates and factors of safety, considering the results of infiltration testing, assessment of controlling subsurface factors, assessment of groundwater mounding, and other factors presented in Appendix C
- Presence of expansive, collapsible, compressible or liquefiable soils
- Presence of slopes and structures and evaluation of stability under stormwater infiltration conditions, including mitigation measures (e.g., setbacks, isolation systems, drains), if appropriate

- Recommended pavement design parameters, such as the modulus of resilience of subgrade soils in presence of infiltration systems

Various other elements may be appropriate to include in the report, at the discretion of the responsible geotechnical engineer.

7 References

ARA, Inc., ERES Consultants Division. (2004). "Guide for Mechanistic-Empirical Design of New and Rehabilitated Pavement Structures", Final Report, NCHRP Project 1-37A, A.R.A. Inc, Chapaign, IL.

ASTM International (ASTM) (2012a). ASTM Volume 04.08 Soil and Rock (I): D420-D5876, updated annually.

ASTM (2012b). ASTM Volume 04.09 Soil and Rock (II): D5877 – latest, updated annually.

Bray, Jonathan, D. and Sancio, Rodolfo, B. (2006). "Assessment of the Liquefaction Susceptibility of Fine-Grained Soils," *Journal of Geotechnical and Geoenvironmental Engineering*, Vol. 132, No. 9, September 1, 2006.

Bowles, J.E. (1996). *Foundation Analysis and Design*, McGraw-Hill, 5th Edition.

Colorado Geological Survey, http://geosurvey.state.co.us/hazards/Swelling%20Soils/Pages/Definition.aspx

Day, Robert, W. (2010). *Foundation Engineering Handbook, Design and Construction with the 2009 International Building Code*, McGraw-Hill, 2nd edition.

Federal Highway Administration (FHWA) (1996). *Geotechnical Engineering Circular No. 2 – Earth Retaining Systems,* Report No. FHWA-SA-96-038.

FHWA (2002). *Geotechnical Engineering Circular No. 6, Shallow Foundations,* Report No. FHWA-SA-02-054.

FHWA (2006). *Geotechnical Aspects of Pavements Reference Manual*, Publication No. FHWA NHI-05-037.

Geo-Slope International (2012). SLOPE/W 2012 Slope Stability Analysis Software. www.geo-slope.com

Hall, K.T. and C.E. Correa (2003). *NCHRP Report 499: Effects of Subsurface Drainage on Performance of Asphalt and Concrete Pavements*, Transportation Research Board of the National Academies, Washington, D.C.

Hall, K.T. and J. A. Crovetti (2007). *NCHRP Report 583: Effects of Subsurface Drainage on Pavement Performance: Analysis of the SPS-1 and SPS-2 Field Sections*, Transportation Research Board of the National Academies, Washington, D.C., 92 pages and Appendices.

Holtz, R. D, W.D. Kovacs, and R.C. Sheahan (2010). *Introduction to Geotechnical Engineering, Second Edition*, Prentice Hall PTR, 864 pages.

Idriss, I.M. and R.W. Boulanger (2010). *Soil Liquefaction During Earthquakes*, Earthquake Engineering Research Institute Monograph MNO-12.

International Code Council (2012). International Building Code (IBC) – 2012.

Kramer, S.L. (2006). *Geotechnical Earthquake Engineering*, Prentice Hall, 653 pages.

Los Angeles County Department of Public Works (LACDPW) (2011). *Low Impact Development Best Management Practice, Guideline for Design, Investigation, and Reporting*, Administrative Manual, County of Los Angeles, Department of Public Works, Geotechnical and Materials Engineering Division dated June 1, 2011.

Michalowshi, R.L. (2002). “Stability Charts for Uniform Slopes,” *Journal of Geotechnical and Geoenvironmental Engineering*, Vol. 128, No. 4, April 1, 2002.

Nakata, J.K., C.E. Meyer, H.G. Wilshire, J.C. Tinsley, and W.S. Updegrove (1990). Photographs from the October 17th 1989 Loma Prieta, California Earthquake: U.S. Geological Survey Open-file Report 90-547, 103 slides.

National Oceanic and Atmospheric Administration (NOAA) (1978). “Geodetic Bench Marks” NOAA Manual NOS NGS 1, Figure 13. By Lt. Richard P. Floyd. http://www.ngs.noaa.gov/PUBS_LIB/GeodeticBMs/

Naval Facilities Engineering Command (NAVFAC) (1986). “Soil Mechanics Design Manual 7.01”, dated September 1986.

Power, M.S. and T.L. Holzer (1996). “Liquefaction Maps,” Applied Technology Council Technical Brief, ATC-35 Research Utilization Project.

Rocscience, Inc. (2012). Slide 6.0 slope stability analysis software. www.rocscience.com

Taber, S. (1929). "Frost Heaving". *Journal of Geology*, Vol. 37, No. 5, pages 428–461. Bibcode 1929JG 37.428T. doi:10.1086/623637

Taylor, D.W. (1937). “Stability of earth slopes,” *J. Boston Soc. Civil Eng.*, Vol. 24, Issue 3. Reprinted in: *Contributions to Soil Mechanics 1925 to 1940*, Boston Society of Civil Engineers, 337–386.

Terzaghi, K., R.B. Peck, and G. Mesri (1996), *Soil Mechanics in Engineering Practice*, John Wiley and Sons, Inc., New York, N.Y., 549 pages.

United States Department of Transportation, Federal Highway Administration (2012). http://www.fhwa.dot.gov/engineering/geotech/pubs/05037/03a.cfm

Washington State Department of Ecology (WADOE) (2012). *Stormwater Management Manual for Western Washington - Volume 3: Hydrologic Analysis and Flow Control BMPs*. Retrieved from https://fortress.wa.gov/ecy/publications/summarypages/1210030.html

Appendix F

Review of Applicability of Permeable Pavement in Urban Highway Environments (White Paper #4)

1 Introduction

Why are permeable pavements of specific interest in achieving volume reduction in the urban highway environment?

What is the purpose of this white paper?

Advancements in permeable pavement design represent a potential opportunity for achieving volume reduction of urban highway runoff. In particular, the reconstruction of existing roads and construction of new roads are potential opportunities for implementation of permeable surfaces with open-graded subbase. These pavement systems replace the surfaces that are the point of generation of runoff and therefore reduce the space requirements for standard end-of-pipe treatment applications. Applications are potentially feasible, from a space perspective, in highly constrained roadway cross sections, and permeable pavements are capable of volume reduction in a wide range of conditions. Additionally, because they take the place of conventional pavements, the incremental capital cost of permeable pavement systems can be relatively minor in new road construction or lane addition projects particularly when considering that treatment BMP sizing requirements may be less or eliminated.

However, there has been limited usage of permeable pavements by road engineers in the urban highway environment due to a wide range of serious concerns regarding their use. Concerns including surface durability, implications of water in the road subbase (and lack of guidance for how this can be safely managed), construction phasing challenges and cost, maintenance requirements, snow management (particularly use of sands) and groundwater quality issues have all be cited as limitations for use, and indeed may be serious concerns in some cases.

These issues do not necessarily rule out permeable pavement in all or part of the highway environment and associated right-of-way areas, but should be understood by project owners and project teams as part of making an informed decision about whether or not and where to use permeable pavement. Specifically, the following important limitations should be understood about permeable pavements:

- In serving dual purposes (as a reliable transportation surface and a stormwater control feature), the design process for permeable pavements is more complex and requires the coordination of pavement design disciplines with water quality, geotechnical, maintenance, etc. that are not traditionally all required to work as closely as they would need to.
- Permeable pavements designs rely on parameters related to infiltration rates (White Paper No. 1), water quality and/or water balance impacts (White Paper No. 2), and geotechnical design (White Paper No. 3), therefore may require a more complex design process.
- Limited information is available on the lifespan of full-depth permeable pavement in the highway environment, and based on available information from installations to date, it appears likely that permeable would have a shorter lifespan than traditional pavement (See Section 3.1.3). More frequent replacement of the top course may be required as compared to a traditional asphalt or concrete pavement. This has practical implications on project whole lifecycle costs.
- Permeable pavement requires more maintenance than traditional pavement and is essential to keeping the system working. In high traffic highway conditions, it may not be practical to provide the level of maintenance required, and permeable pavement may be infeasible.

- In general, there is still limited information available to help make decisions about use of permeable pavement in the urban highway environment.

The purpose of this white paper is to provide an up-to-date analysis of the potential applicability of permeable pavement technologies in the urban highway environment and to provide practical guidance and information to help owners, project managers, and designers navigate the decision process about whether permeable pavements should be considered for a specific project. This white paper is based on a synthesis of published literature, experiences of the research team, and selected stormwater guidance documents. This white paper is not intended to provide a comprehensive set of criteria for evaluating permeable pavements that are applicable in all cases, nor is it intended to replace permeable pavement design manuals that exist or are currently under development. Project planning and design professionals should exercise appropriate judgment in considering permeable pavements for infiltration of stormwater from highways, including important site- and agency-specific factors such as climate and maintenance capabilities.

The remainder of this white paper is organized as follows:

- **Section 2** introduces permeable pavements technologies.
- **Section 3** provides discussion of the applicability of permeable pavement and provides guidance for evaluating the applicability of permeable pavement for a given project based on a number of factors.
- **Section 4** introduces considerations for design of permeable pavement systems in the urban highway environment and introduces a conceptual design approach that can be followed to help coordinate efforts of different design disciplines.
- **Section 5** discusses needs for future research related to pavement durability, quality assurance measures, and maintenance aspects of permeable pavements.

Significant work is being conducted in parallel with this report. American Society of Civil Engineers (ASCE) (Permeable Pavements Technical Committee, Low Impact Development Standing Committee, Urban Water Resources Research Council, Environment and Water Resources Institute) in preparation of a Manual of Practice on Recommended Design Guidelines for Permeable Pavements. The ASCE Manual should serve as a valuable reference for more detailed design information when it is published. NCHRP Project 25-25/Task 82 posted a report online in October 2013 providing guidance on design, construction, and maintenance of permeable shoulders with stone reservoirs.

2 Permeable Pavement Technologies

This section discusses the history of permeable pavement technologies, current permeable pavement technologies and the differences between flexible permeable pavements (i.e., permeable asphalt and pervious friction course) and rigid permeable pavements (i.e., permeable concrete).

What permeable pavement technologies are available to the designer?

How have permeable pavement technologies evolved with time?

2.1 Types of Permeable Pavement Technologies

Permeable pavement is a universal term used to describe an assortment of materials that provide a hardened surface yet facilitate stormwater infiltration. Examples of these materials include flexible pavements (i.e., permeable asphalt and permeable friction course/open-graded friction course - PFC/OGFC), rigid pavements (i.e., permeable concrete), concrete pavers, polymer based grass pavers, plastic turf reinforcing grids and geocells.

What are the key differences between flexible and rigid permeable pavement designs?

How does this influence their applicability in the urban highway environment?

Full depth permeable pavements are capable of substantial storage and infiltration. Friction courses (PFC/OGFC) that are overlain on pavement have minimal storage and volume reduction capabilities, therefore is not considered a volume reduction approach. However, the mix designs for these pavement types are nearly identical and experiences with installation and durability of PFC provide a reference for the potential applicability of full-depth permeable pavement systems and help address questions about the design life of permeable wearing courses that would be used in full depth permeable pavements.

Full depth permeable pavements can be divided between flexible and rigid designs. These types of permeable pavement have similar applications but they vary in a number of ways. Some key differences between flexible and rigid permeable pavement are discussed in Table 1.

Table 1. Comparison of Flexible vs. Rigid Permeable Pavement

Attribute	Permeable Asphalt	Permeable Concrete
Deformation	Flexible	Rigid
Strength	Typically lower strength; rely on combination of admixtures and subbase construction for strength.	Typically high strength.
Color	Dark in color; absorbs heat. Facilitates faster melting in winter.	Lighter in color; reflects sunlight. Can be more issues with ice formation.
Mix Design	More commonly used in northern climates; more suppliers and contractors familiar with product.	More common in southern climates. Less common in general; mix is more susceptible to installation issues (water content must be exact).
Installation	No special tools/equipment required; good quality control in subbase preparation is critical to achieve sufficient compaction and infiltration.	Pavement must be covered immediately after installation to retain moisture and allow concrete to cure; generally a slower construction process than permeable asphalt. An extended curing period is required prior to application of chloride deicers.

2.1.1 Full Depth Permeable Asphalt

Permeable asphalt is an alternative to traditional warm mix asphalt (WMA) or hot mix asphalt (HMA) and is produced by eliminating the fine aggregate from the asphalt mix (Barrett and Stanard, 2008). Permeable asphalt typically consists of conventional WMA or HMA with significantly reduced fines resulting in an open-graded mixture that allows water to pass through. It is similar in appearance to conventional asphalt pavement, although generally coarser in texture. With the removal of aggregate fines there is a decline in pavement strength which is typically addressed by the use of higher grade binders and/or admixtures. Common admixtures include fibers and styrene butadiene rubber (SBR), a synthetic rubber. Permeable asphalt usage is greatest in the northern US where summer temperatures have less of a limiting impact for asphalt based flexible pavements.

The permeable asphalt surface void space typically ranges from 16 to 25 percent. In comparison, voids for standard asphalt are typically 2 to 3 percent and are not interconnected. Permeable asphalt pavement is placed directly on an open-graded aggregate subbase to allow for infiltration. This subbase includes a choker course or an asphalt-treated permeable base (ATPB), an optional filter course (typically included for added water quality treatment), and a reservoir course for volume control, infiltration, and to act as a frost protection capillary barrier in northern climates. This subbase is used in replace of a conventional dense-graded subbase. Preparation of the subbase requires care to ensure that compaction requirements are met to both provide sufficient structural support while still maintaining sufficient infiltration capacity. This is achieved through construction quality controls. The balance of compaction and infiltration is one of the keys to achieving successful permeable pavement installations.

Typical installations of permeable asphalt have included low to moderate traffic parking areas, sidewalks, pathways, recreational areas (e.g., basketball courts), driveways and low-speed/low-volume roadways. Higher volume roads and highways have been also constructed of permeable asphalt with success (Hossain et al 1992, MaineDOT 2010); however, installation of full-depth permeable asphalt in the urban highway environment is not a common practice. This is in large part due to the contradictory design elements of standard road construction with focus in part on keeping water out of the roadbase, in contrast with a well-drained permeable pavement subbase. Additional unique challenges exist for the construction phasing of a permeable subbase, in particular for redevelopment. This white paper will address each of these concerns in order to evaluate whether permeable asphalt is potentially a viable option for achieving volume reduction of highway runoff in the urban environment.

2.1.2 Full Depth Permeable Concrete

Permeable concrete or permeable rigid pavement can be used in place of traditional rigid pavement where site conditions are suitable. Permeable concrete contains little or no sand content therefore creating a substantial void ratio (ACI 2008). The usage of permeable concrete is greatest in the southern US where the hot summer temperatures can limit the usage of flexible (asphalt) pavements although recent efforts have evaluated the application of permeable concrete for cold weather climates and generally made advancements in this area (Schaefer et al 2006). In climates where freeze-thaw is not a concern, permeable concrete has the substantial benefit of requiring little or no subbase for structural demands. Subbase construction may still be required to achieve volume control and infiltration requirements/goals. Strength requirements for permeable concrete are based on subgrade strength, pavement rigidity, and thickness of the concrete layer. In northern climates, substantial concerns exist for the usage of permeable concrete and durability issues associated with the application of chloride deicers. Lengthy curing periods of up to 12 months have been reported prior to chloride application (UNHSC 2012)

The production and installation of permeable concrete involves substantial quality controls to ensure successful applications. The American Concrete Institute has developed national guidelines (ACI 2008) for the production, design, and construction of permeable concrete. Material testing standards have been developed from the American Society for Testing and Materials (ASTM) Subcommittee C09.49 on Permeable Concrete. The industry has developed a Permeable Concrete Contractor Training Course and Certification that is administered by the National Ready Mix Concrete Association. The state of the practice in production and installation has improved via these national level efforts.

2.1.3 Potential Alternative Configuration for Stormwater Storage and Infiltration below Traditional Asphalt

WSDOT is conducting research on an alternative configuration for permeable pavement that would provide storage and infiltration surface area below a traditional pavement, by providing a system of inlets and pipes to convey water into this storage/infiltration area rather than through a permeable pavement top course (Personal Communication, Mark Maurer). This configuration could help avoid potential issues with the strength, durability, constructability, and maintenance of the permeable top course, while still making use of storage capacity and infiltration surface area below the road or shoulder. In such a configuration, the designer would need to be conscious of the potential sediment load that would be transported directly into the subbase layers and include provisions for removing sediment in the flows prior to routing flows to the subbase reservoir to avoid premature clogging of the infiltrating surface (below the subbase reservoir). In contrast, a permeable top course provides both a conveyance and filtering function; hence the need for vacuum maintenance of the top course to remove captured sediment before it clogs the surface or migrates to the subsurface. The tradeoffs between these two approaches for delivery of stormwater to the subbase reservoir should be considered in evaluating this alternative configuration.

2.1.4 Other Permeable Pavement Technologies

Permeable friction course (PFC) overlays are the most commonly used method of permeable pavement in urban highway environments. PFC overlays, also known as OGFC have been installed on roadways in an effort to make existing roadways quieter and safer. The pervious course leads to shorter safe stopping distances for cars, quicker surface drying periods and less splash and spray during precipitation events as well as more muffled sounds in general. Permeable friction course overlays can be cost effective and appropriate in the urban highway environment (TxDOT 2012, Younger 1994, NAPA 2002a). They have been shown to provide water quality benefits in terms of concentration reductions. However, because PFC does not appear to provide significant volume reductions (some more minor evaporation losses), it is not considered a significant volume reduction approach (VRA).

Permeable concrete pavers are concrete units installed with open, permeable spaces typically filled with aggregate. Concrete pavers provide an architectural appearance while also providing structural load bearing capabilities. Concrete pavers are most commonly used for suitable for walkways, patios, driveways, and parking lots. The design of permeable paver systems is analogous to permeable asphalt (flexible design), however the cost of permeable paver systems tends to be substantially higher than permeable asphalt or permeable concrete due to the substantial handwork required and may be less applicable to the urban highway environment. Methods of placing pavers in blocks with specialized equipment may improve the efficiency of installation and associated costs to some extent.

Polymer based **grass pavers, plastic turf reinforcing grids and geocells** provide load bearing reinforcement for unpaved surfaces of gravel or turf. These systems are comprised of a series of flexible plastic rings in a grid filled with aggregate laid on top of a base course. The flexible plastic grid is engineered to withstand structural loads, reduce compaction of the aggregate, and provides stability, flexibility and continuity over large surface areas. This permeable pavement type has been used for parking aisles and bays, automobile or truck storage yards, loading dock areas, recreational trails, boat ramps, infiltration basins and residential sidewalks and driveways. It is not applicable for direct highway use in travel lanes. However, it could be considered in shoulder applications.

2.2 History of Permeable Pavements in the Highway Environment

2.2.1 Early Experiences

In the late 1960s, permeable pavement was produced to promote infiltration, reduce storm sewer loads, reduce floods, raise water tables, and replenish aquifers (NAPA, 2002b). Throughout the 1970s, the concept was discussed and refined to a point where the Environmental Protection Agency (EPA) sponsored research to determine the capabilities of several types of permeable pavements for urban runoff control (Thelen, 1978). Many permeable pavement sites have been constructed since the late 1970s, with both failures and successful applications. Failures tended to be associated with clogging of the surface from silt, raveling (i.e., individual aggregate particles dislodge from the pavement surface) and degrading of underlying layers (NAPA, 2002b). As failures occurred, lessons were learned that have been applied to manufacture, design, and installation of permeable pavements. A substantial body of research has emerged as a result in part for the development of PFC/OGFC and for full depth permeable pavements.

2.2.2 Introduction of Permeable Friction Course

In 1974, the Federal Highway Administration (FHWA) developed a mix design procedure for permeable friction course (PFC) which is also known as open-graded friction course (OGFC), plant mix seal, popcorn mix, or asphalt concrete friction course (Mallick, et.al., 2000; NAPA, 2002a). The drainage consists of rainwater infiltrating vertically through the PFC to an impermeable underlying layer of standard hot mix asphalt and then runoff moving laterally from the crowned point to the day-lighted edge of the PFC (NAPA, 2002a). PFC was found to reduce hydroplaning, provide higher frictional resistance in wet conditions, reduce spray, enhance visibility, reduce nighttime glare, and reduce pavement noise (NAPA, 2002a).

Early issues arose concerning the durability of these open-graded mixtures resulting in delamination and raveling of surface overlays (Younger et al, 1994). These issues were particularly severe in northern climates. These problems have been resolved, to some extent, through the use of modified asphalt binders with polymers and fibers, open aggregate gradations, and quality control assurances (Younger et al. 1994, Kandhal and Mallick 1999). In the United States, Oregon, Washington, California, Nevada, Arizona, Florida, Vermont, and Georgia have used OGFC extensively. Some other DOTs have discontinued the use of PFC due to concerns about durability and service life. In Europe, it has been widely implemented since the 1980s in Germany, Netherlands, France, Italy, United Kingdom, Belgium, Spain, Switzerland, and Austria.

The Oregon Department of Transportation (DOT) has been using PFC on its highway systems since the late 1970s (Younger et al. 1994, NAPA, 2002a), both in the more temperate western portions of the state and the higher elevation eastern portions of the state. PFC became Oregon DOT’s preferred choice for surface course and has applied it to more than 2000 lane miles of Oregon highways (NAPA, 2002a). However, in light of recent ODOT experiences with premature failures corresponding to increased snow conditions in 2008 and 2009. ODOT has updated its *Pavement Design Guide* to recommend that PFC not be installed where there is frequent snowplow activity, significant landslide activity, or underlying hot mix asphalt is experiencing damage from moisture. Meunch et al. (2011) additionally recommended that ODOT consider the limiting use of PFC to lower traffic volumes and considering the use of studded tires, particularly on busses and trucks, in evaluating PFC applicability. Consistent with the ODOT experience, PFC has had less success in more northern climates in part due to problems resulting from use of studded tires, freeze-thaw cycles, and subsequent pavement delamination. There is potential for water to become trapped in void spaces at the boundary between the PFC and the underlying impermeable roadway and this water can then freeze and become dangerous as black ice and exacerbates deicing challenges.

2.2.3 Full Depth Permeable Pavement Systems

One of the earliest installations of full depth permeable asphalt was a 1975 demonstration project at the Walden Pond State Reservation in Concord, Massachusetts (Wei 1986, Ferguson 2005). The parking lot is nearly 40 years old. A 2007 inspection indicated the pavement had modest infiltration capacity however the pavement surface was observed to be in good condition, with no visible cracks or evidence of heaving

(Briggs et al 2008). There have been tremendous advances in the technology of permeable asphalt mix design since this time. This heavily visited park has no winter sanding operations, good underlying soils, street sweeping included in the maintenance program, and no fine soils or silts being tracked on or deposited on the pavement. The principal cause of breakdown of asphalt parking lots in northern climate is cracking due to freeze-thaw. However, a deep and well-drained subbase typical of full depth permeable pavements appears to be less susceptible to free-thaw.

Full depth permeable pavement systems represent a distinctly different concept in stormwater control with emerging applicability in the urban highway environment. Unlike PFC, full depth permeable pavement consists of a permeable wearing coarse, underlain by an open-graded permeable subbase, such that water is conveyed directly into the subbase reservoir where it then infiltrates into the subgrade and/or is discharged via an underdrain system. The wearing course of full depth permeable pavements is nearly identical with respect to the mix design and production of PFC, full depth systems however deviate from traditional pavements in terms of the design and construction of the subbase. The 2003 publication by NAPA presents the essential design elements for both permeable asphalt mix design and subbase construction (2003, 2008 NAPA). Guidelines for permeable concrete design have been developed by the American Concrete Institute (ACI 2008).

Full depth permeable pavement systems have been widely applied in parking lots and walkways, and are seeing more widespread use in low volume roadways. For example, in Pelham, New Hampshire, a 1,300 foot permeable asphalt road was installed for a new residential adult community to provide increased groundwater recharge (UNHSC, 2009). This project sits atop a large sand deposit, ideal for the infiltration of stormwater runoff. The permeable asphalt mix design included a combination of asphalt admixtures to boost strength and durability. Although the cost of materials for a permeable asphalt road was 25 percent higher than that of a traditional asphalt roadway, the developers saw overall project cost savings of 6 percent (compared to the total project costs with a traditional road) by avoiding substantial stormwater management infrastructure like curbs, catch basins, and retention ponds (UNHSC, 2009).

2.2.4 Moving Towards Urban Highway Applications of Full Depth Permeable Pavements

Research and development in PFC for highway applications has led to additional durability improvements to the permeable pavement wearing surface through the use of additives and higher performance-grade binders. These additives have the potential to broaden application of permeable surfaces to highways with increased ESAL load/traffic demands. Additionally, recent project experiences and research have established empirical evidence for the potential suitability of permeable surfaces within high volume roadways with heavy loadings. Finally, some of the issues identified with durability of PFC overlays in northern climates are addressed in part by the use of a full depth permeable system that does not trap water in surface pores where freeze-thaw cycles can be damaging.

St. John and Horner (1997) were sponsored by WSDOT to evaluate permeable asphalt highway shoulders, gravel shoulders, and conventional asphalt shoulders on a highway near Redmond, Washington. They found that permeable asphalt shoulders reduced runoff by 85 percent and solids and pollutants by more than 90 percent as compared to a conventional asphalt shoulder; permeable asphalt shoulders also significantly out-performed gravel shoulders. After one year, the permeable asphalt showed no signs of clogging and had an infiltration rate of 1750 in/hr. Recent information regarding the long-term performance of this system could not be located, however an unpublished research proposal on the WSDOT website (WSDOT 2013) indicates that the agency is proceeding with plans to study permeable shoulders, including potentially conducting a retrospective analysis of the retrofit originally studied by St. John and Horner (1997). WSDOT is also considering use of permeable shoulders on SR-18 in Federal Way, WA (Personal Communication, Mark Maurer). WSDOT has policy limitations related to the use of permeable pavements in travel lanes, however this does not prohibit permeable shoulder installations (WSDOT 2011).

In 2009, the Maine Department of Transportation constructed the first state permeable asphalt roadway in the northeast. The project includes a full depth permeable pavement for 1,500 feet of a 4 lane highway reconstruction for the Maine Mall Road in South Portland. In 2008 the average annual daily traffic for 2008 was 16,750 vehicles, including significant heavy truck traffic volumes. Four years after construction, the durability has been noted as exceptional. MaineDOT is conducting ongoing monitoring and information about long-term performance for volume reduction and durability (MaineDOT 2010). A cost assessment was conducted by MaineDOT and the full depth construction was found to be about 2 percent more than standard new road construction. In this case, however, the road was reconstructed, so costs were greater than would have been seen in a traditional reconstruction (for reasons discussed later in this white paper).

The University of California Pavement Research Center (UCPRC), in cooperation with Caltrans, has recently completed laboratory and modeling investigations to evaluate the structural and hydraulic performance of permeable pavement as highway shoulders that can be subject to heavy loads (Wang et. al. 2010, Jones et al., 2010; Chai et al., 2012). This study included in-depth analysis of subgrade properties and influence of compaction on permeability as well as strength. The studies conducted by UCPRC have determined that the retrofit of roadways is technically feasible and is economically advantageous in freeways when compared to conventional stormwater structure installation (Chai et al. 2010, summarized by Kayhanian, 2012). Further, this research suggested that permeable shoulders could be effective for subgrade permeability as low as 10^{-5} cm/s (0.014 in/hr). However, field scale evaluation was not conducted, and as noted by Kayhanian, "any simulated design must be constructed and tested before [wide scale] implementation in highway or road environments."

Permeable concrete applications are also being advanced through research. FHWA-sponsored research in progress is evaluating a full depth permeable concrete shoulder with photo-catalytic cement binder for water quality as well as air quality benefits in a 46,000 AADT urban freeway in St. Louis (MO) (Cackler et al., 2012). This pilot included development of full depth permeable concrete shoulder designs capable of withstanding highway loadings. Monitoring, which includes hydrology and water quality as well as pavement integrity, is expected to conclude in 2013 or 2014, with a follow up report prepared after monitoring is completed. Other FHWA-sponsored research in progress is evaluating permeable concrete overlays for a low volume highway in Minnesota (Schaefer et al., 2011). While issues have been encountered with durability at joints, the researchers suggest that these issues could be addressed with modifications to design and construction, and overall found significant potential for widespread application of permeable concrete wearing course even in these more northern climates.

2.3 Benefits Full Depth Permeable Pavements – Basis for Consideration

Why should full depth permeable pavements be considered in the urban highway environment?

When compared to other highway surfaces, permeable pavement surfaces (including full depth permeable pavement and PFC) have demonstrated the following advantages (FHWA, 1990):

- Provide and maintain excellent high speed, frictional qualities (the frictional characteristics are relatively constant over the normal operating speeds);

- Reduce the potential for hydroplaning;
- Reduce the amount of splash and spray;
- Are generally quieter, often providing a 3 to 5 decibel reduction in tire noise when new;
- Improve the wet weather, night visibility of painted pavement markings;
- Filtering and capture of pollutants.

Benefits that may be associated with full depth permeable pavements, that are not associated with PFC, include:

- High levels of runoff volume reductions can be achieved annually, in many some instances greater than 90 percent of the potential runoff is infiltrated;
- Reduction in magnitudes and durations of peak flow rates;
- Reduction in temperature of runoff during summer months;
- Reduction in the usage of costly drainage infrastructure including catch basins, pipe, ponds, hydraulic control structures, and/or stormwater treatment controls;
- Reduction in the use of land area required for stormwater management including limited right of way and adjacent areas;
- Improved safety in cold weather climates (reduction in black ice), when subgrade drainage is adequate;
- The use of quality control measures are well established in the transportation environment for materials production and construction that can be adapted for permeable pavements;
- Relatively minor cost difference compared to traditional pavements for new road construction.

For these reasons, full depth permeable pavements warrant consideration in the urban highway environment, and indeed is being considered by DOTs in both warmer and colder climates.

3 Evaluating the Applicability of Permeable Pavement in the Urban Highway Environment

While permeable pavements have been used successfully in many applications, including some experience in the urban highway environment, a number of questions remain regarding whether permeable pavements are applicable in a high traffic, high loading, and/or maintenance-constrained environment. This section provides guidance for evaluating these questions for urban highway projects. These questions are critical for project owners to evaluate on a case-by-case basis to determine if permeable pavement is appropriate for a given project.

Section 3.1 begins with a summary of the state of the practice related to common questions that are raised about permeable pavement in the context of the urban highway environment. Based on this review of issues and summary of the state of the practice, Section 3.2 then provides a summary of factors that influence permeable pavement applicability and the conditions that are more suitable and less suitable for permeable pavement.

3.1 Common Considerations Associated with Full Depth Permeable Pavements in Urban Highway Applications

There are a range of perceptions about the applicability of permeable pavements in the urban highway environment. While some of these are consistent with current state of knowledge or lack thereof, some may be based on historic pavement technologies, improper siting, or issues with quality control, which may be possible to address in modern pavement designs and proper construction with good guidance and implementation. This section identifies key perceptions about permeable pavements in the urban highway environment and discusses implications for incorporating these technologies into the urban highway environment. It is important to note that some concerns may not be possible to cost-effectively address and may render permeable pavement infeasible for a given project or region.

What are key issues that are commonly identified as limiting the applicability of permeable pavement in the urban highway environment?

Where is the current state of the practice relative to these issues?

What are the implications for use of permeable pavement in the urban highway environment?

3.1.1 Design Complexity

Issue: Permeable pavements can be more complex to design and specify than traditional pavements.

Permeable pavements are required to serve dual purposes (as a transportation surface and a stormwater control feature), therefore the design process for permeable pavements is inherently more complex and crosses over multiple design disciplines. However, detailed design guidelines and specifications, for permeable pavements in general, can be found from research and engineering institutions (e.g., ASCE 2013, UNHSC 2009)), industry organizations (NAPA 2008, ACI 2003), and increasingly in many state stormwater manuals, and department of transportation specifications. Specifications have become increasingly available in recent years; many state asphalt associations provide additional guidance and specifications, so it is important to check local requirements. All specification guidelines and information should be evaluated on a case-by-case basis and will vary due to local materials/practices and the specifics of the project (size, traffic, climate, etc.). There is an increasing familiarity with these mix designs by the major suppliers whom are accustomed to the production of PFC. Standard mix designs are becoming more commonplace in many areas and thus easier to specify and produce.

Nonetheless, standardized design guidance specifically tailored for urban highway applications remains relatively limited. Table 2summarizes some of the major design considerations applicable to installation of permeable pavements in the urban highway environment and discusses how this guidance can be adapted to urban highway applications.

Table 2. Summary of Design Considerations in the Urban Highway Environment

Design Element	Typical Applications in Non-Highway Environment	Considerations Specific to the Urban Highway Environment
Permeable Asphalt Wearing Course	• Designed as flexible pavement per AASHTO (1993) or applicable local guidance. • Typically 10 to 15 cm (4 to 6 inches) thick. • Common admixtures include cellulose or mineral fibers for added strength and Rubber Solids (SBR)	• Two-layer wearing course typically required for higher loadings, including asphalt-treated permeable base (ATPB) directly beneath permeable asphalt • Admixtures typically needed for strength
Permeable Concrete Wearing Course	• Designed as "rigid pavement" per ACI (2008). • Typically 15 to 20 cm (6 to 8 inches) thick. • Design of wearing course should account for lower modulus of elasticity, flexural strength, and subgrade strength compared to traditional concrete pavement.	• Thickness of wearing course may need to be considerably greater to accommodate highway loadings and provide reliability consistent with highway applications.
Asphalt-Treated Permeable Base (ATPB)	• Course permeable layer underlying surface course • Typically 7.5 to 15 cm (3 to 6 in.) in depth • Provides a more stable paving platform for permeable asphalt wearing course	• ATPB adds structural strength to the pavement needed for the highway environment
Choker Course (Optional)	• Clean-washed, uniformly graded aggregate course (typically AASHTO No. 57) • Typically 2.5 to 5 cm (1 to 2 in.) in depth • Provides an even surface for paving and prevents excessive pavement loss into the reservoir coarse	• When an ATPB is used, a choker layer may not be necessary.
Filter Course (Optional)	• Poorly graded (i.e., open graded) sand filter course between choker and reservoir course • Typically 20 to 30 cm (8 to 12 in.) in depth • Must be underlain by intermediate setting bed of 8 cm (3 in.) of pea gravel.	• Provides additional water quality benefits, but may be vulnerable to clogging where sediment loads are higher.
Reservoir Course (i.e., Subbase)	• Clean-washed, uniformly graded aggregate (often AASHTO No. 2 or 3) with 40 percent voids • Typically 0.2 to 0.9 m (8 to 36 in.) in depth • Depth determined from analysis of requirements for volume/peak control, frost depth, structural, depth to bedrock, seasonal high water table and site grading	• In permeable asphalt, subbase thickness is a key design variable for structural loading requirements as well as stormwater requirements. • In permeable concrete applications, subbase is not necessarily required; typically controlled by stormwater requirements.
Geotextile Liners	• Recommended on vertical sides of excavation to prevent inward migration of fines and soil piping and	• No specific additional considerations.

Design Element	Typical Applications in Non-Highway Environment	Considerations Specific to the Urban Highway Environment
	short-circuiting • Required bottom of excavation for soils with poor load bearing capacity or high fines content	
Subgrade Compaction and Preparation	• Subgrade compaction by heavy equipment traffic should be minimized where feasible by construction phasing • Where operation of heavy equipment on subgrade is required, bed bottom should be prepared by scarifying or tilling the subgrade to no less than 20 cm (8 in.)	• Avoiding subgrade compaction may be more challenging in roadway construction and/or may be required for strength requirements; a factor of safety in infiltration rates may be needed to account for unavoidable/necessary compaction.
Pavement and Subgrade Slope	• Pavement surface should be as level as feasible to promote uniform infiltration. • Bed bottom should be as level as feasible to promote uniform infiltration.	• Level surface is not common in highway installations for pavement or bed bottom. • Segments with significant longitudinal grade may significantly complicate design and construction of permeable pavements. • For sloping systems, earthen berms or fabric barriers should be constructed prevent bed bottom erosion and provide runoff storage capacity.
Underdrain and Outlet Control	• Underdrains should be used for soils with lower infiltration capacity or uncertain long-term infiltration capacity (i.e., lower than 0.05 in/hr). • Elevated underdrains can be used to provide infiltration of the water quality volume at minimum prior to discharge. • Manhole inlets may be designed with outlet control features similar to typical detention systems to manage factors such as peak discharge rate and volume.	• Positive overflow from the underdrain/overflow system to a location that will not create negative impacts to slope stability or highway flooding is imperative.
System Fail Safe	• Typically a backup method for water to enter the reservoir course is incorporated into design • This ensures system function in the event that the pavement becomes clogged or is sealed or repaved with standard impermeable asphalt.	• Overflow pathways to a supplemental drainage system are critical in the highway environment to help ensure that lanes are not flooded in the event of pavement failure or highly intense, sustained precipitation.
Tributary Area to Permeable Area Ratio	• Contributing drainage area to the permeable asphalt surface typically limited to two to four times the area of permeable pavement • High sheet flow loading (run-on) to permeable asphalt may lead to premature clogging or could require excessive maintenance • Additional hydraulic loading (i.e., adjacent rooftops) is best routed to pavement subbase through drywells or infiltration trenches with access for maintenance	• In cases where permeable pavement is only installed on road shoulders, ratios greater than 4:1 may be unavoidable. • Increased ratios associated with permeable shoulders will tend to increased maintenance demands.

Design Element	Typical Applications in Non-Highway Environment	Considerations Specific to the Urban Highway Environment
Quality Control	• Quality control needs to be documented and implemented during mix production, subbase construction, and materials placement	• Application of QC controls in highway environment can leverage the usage of standard QC elements adapted to evaluate infiltration and compaction requirements.

Sources: ASCE (2013 in draft), NAPA (2008), ACI (2008), unpublished professional experience.

3.1.2 Strength Limitations

Issue: Is it possible to design a permeable pavement with enough strength for urban freeways?

Strength needed for urban highways can be achieved in most conditions through top course mix design and subbase design, as discussed in Section 3.1.1, above.

Asphalt. Contemporary guidance for permeable asphalt mix designs specifies the use of an asphalt binder two grades stiffer than the local standard and/or admixtures to boost strength and durability. Most examples of poor durability pavements have not included stiffer binders and/or admixtures. Admixtures typically include fibers and/or synthetic rubbers such as styrene butadiene rubber (SBR) or styrene butadiene styrene (SBS), which allow the pavement to stand up to the shear stress of high speed traffic on urban freeways (NAPA 2003). Strength and durability of the wearing course can be evaluated through a number of standardized QC tests including: Air Void Content (ASTM D6752), Binder Draindown (ASTM D6390), Retained Tensile Strength (AASHTO 283), Cantabro Abrasion Test on un-aged and 7 day aged samples (ASTM D7064-04). The overall strength of a flexible pavement is also a function of the subgrade modulus of resilience (Mr) and the subbase design. The strength of permeable pavement systems can be augmented through the use of an ATPB layer, a thicker subbase, and/or higher compaction of the subgrade material. Chai et al., (2012) found that permeable pavement designs were feasible for highway shoulders, considering a detailed evaluation of loading, pavement design, and subbase/subgrade properties. In some soils, saturated subgrade strength is very limited, however the researchers found that this lack of strength could be compensated using a thicker subbase layer.

Concrete. For permeable concrete designs, the thickness of permeable concrete can be increased to meet additional structural requirements and compensate for lower modulus of elasticity, flexural strength, and subgrade strength (Delatte 2007) associated with permeable concrete applications. ASTM, Standard C-1754/1754M provides a standard test method for density and void content of hardened permeable concrete. Improvements in cement binders and admixtures Kevern and Farney (2012) show promise for improving surface durability and curing times.

3.1.3 Lifespan and Longevity

Issue: Are permeable pavements subject to premature failure and rapid raveling? Is information available about the life span of permeable pavements in the urban highway environment?

While notable experiences with pavement failure are relevant and important to consider, technologies and quality control have improved considerably. Earlier applications of open-graded friction course were considered dry (relatively low liquid asphalt content) and subject to premature failure and rapid raveling (McGee et al., 2009). Mixes with higher asphalt content were shown to perform very well; however, would ravel more quickly once the mix oxidized (McGee et al., 2009). Additions of polymers to mixes have reduced the potential for materials to oxidize, thereby theoretically making the pavement more flexible and durable for a longer period of time. Relatively extensive experience with PFC overlays in the urban highway environment provides empirical evidence of lifespan of permeable asphalt top course and has supported enhancements in permeable asphalt durability. According to a 1998 National Center for Asphalt Technology survey, which included responses from 43 state highway agencies, more than 73 percent of respondents reported greater than 8 years average service life (Table 3).

Table 3. Average Service Life of PFC Overlays

Average Service Life	Percentage of 43 States
Less than 6 years	17%
6 to 8 years	10%
8 to 10 years	30%
10 to 12 years	33%
12 years or more	10%

Source: Kandhal and Mallick, 1998

However, PFC wearing surfaces do tend to have a shorter life span than the standard dense-graded hot mix asphalts due to the occurrence of a frost lens at the interface of the PFC and the impermeable binder course. Liu, et al., (2010) reported reduced performance associated with reduced durability and functionality (i.e., permeability and noise reduction effectiveness) and raveling and clogging. Full depth permeable asphalts are expected to avoid these issues to some extent as water is not trapped near the surface and susceptible to freeze-thaw cycles, however more research will be needed to assess the structural durability in northern climates were frost heave damage is commonplace for roadways. In addition, in some states allowed seasonal studded tire use, which has also been noted to be a significant factor in pavement design life, for both PFC and dense mix asphalt (Meunch et al. 2011). A review of the effect of traffic volume on pavement lifespan (Meunch et al. 2011) suggests that permeable shoulders, rather than travel lanes, may help improve lifespan of permeable pavement relative to vehicular wear due to their lower traffic volume.

Wang et al., (2010) assumed a replacement time of 10 years in their lifecycle cost assessment of permeable shoulders, and found them to be economically advantageous compared to other stormwater controls, indicating that a long lifespan is not necessarily required to make permeable pavement economically viable.

A key consideration related to longevity of permeable concrete has been the freeze-thaw durability of permeable concrete and durability around joints. Schaefer (2011) found that some issues remain with durability around permeable concrete joints, however this FHWA-sponsored research concluded that "well-designed permeable concrete mixes can achieve strength, permeability, and freeze-thaw resistance to allow use in cold weather climates." Kevern and Farney (2012) have noted significant improvements in binders and admixtures.

3.1.4 Infiltration Limitations

Issue: Can permeable pavements be constructed on low permeability soils? Does the necessary compaction of roadway subgrade for structural reasons eliminate the potential for infiltration?

It is rare that a site would not be suitable for use of permeable pavements solely due to low permeability soils. Permeable pavements typically have a much larger footprint to drainage area ratio than other types of volume reduction approaches, which means that a given storm depth results in a smaller depth of ponding on an infiltrating surface and therefore lower infiltration rates can still infiltrate this runoff in a reasonable time period. Similarly, as a traditional road base design for poor load bearing soils requires construction of a sufficiently deep structural base, a sufficiently deep reservoir course would be needed in combination with an appropriate underdrain design. Chai et al. (2012) found that permeable shoulders were feasible in typical California soils (silts and clays) with limited saturated bearing strengths to a lower effective permeability of approximately 10^{-5} cm/s (0.014 in/hr).

With any design project is it important to first determine that site conditions are suitable for the intended design. For permeable pavement projects one of the first steps is to determine if full or partial infiltration is a viable option at the site. Obstacles preventing or limiting infiltration could include very low subsurface soil infiltration rate, utility conflicts, high groundwater table, contaminated subsurface soils and/or groundwater plumes down gradient, proximity to water supply and bedrock, swelling soils, and other factors. If infiltration is limited by one or more of these obstacles, partial infiltration can be achieved by providing an underdrain (or raised underdrain) in a reservoir course beneath the pavement surface. If infiltration is not allowable or desired then the systems can be lined with a compacted soil liner. While there is negligible potential for volume reduction in a lined system, these designs can provide stormwater management in the form of filtration, detention and slow release of collected stormwater off-site.

In addition to subgrade, the subbase layers also need to be tested for infiltration capacity during installation. Once the reservoir course aggregate is installed, the filter course is installed typically to both a minimum 95 percent standard Proctor density (ASTM D698 or AASHTO T99) and a minimum infiltration rate typically no less than 10 ft/day determined by ASTM D3385 is recommended.

3.1.5 Pavement and Subbase Slope Limitations

Issue: Should installations be limited to flat or low slope surfaces? What challenges are posed by sloping installations?

Ideally infiltration systems are constructed with low or no slope bed bottoms as this allows a level pool to form within the reservoir. However in a highway environment, level longitudinal grades are not common and it is infeasible to adjust road grades simply to accommodate permeable pavement. If slope is not considered in the design of permeable pavements, the effectiveness of permeable pavements can be reduced (i.e., less effective storage volume) and erosion of subbase/subgrade material can occur as water flows longitudinally below the surface along the soil interface. To allow for level pools and to control longitudinal flow, subbases can be constructed in a tiered stair-step manner to maintain level bottoms, however this approach is challenging to construct. A more common approach for use in roadways involves the use of low transmissivity geotextiles along isoelevation contours at elevation intervals of 6 to 12 inches to create internal barriers within the reservoir course. This helps prevent erosion within the reservoir layer on slope and provide runoff storage capacity. This method is relatively inexpensive and simple to construct, and has been used successfully on slopes up to 9 percent in low volume roadways (UNHSC, 2009). However it does have the potential to slow construction and increase complexity, especially where slopes exceed 2 to 5 percent (at 12 inch elevation spacing, this would correspond to a fabric wall every 20 to 50 feet). Therefore, permeable pavements are more ideally used on slopes as flat as possible, and not greater than 2 to 5 percent.

3.1.6 Geotechnical and Pavement Design Concerns

Issue: Does permeable pavement introduce geotechnical and pavement design issues associated with saturated subbase/subgrade that cannot be mitigated?

Geotechnical and associated pavement design issues are a key site-specific factor in assessing the feasibility of permeable pavements. In general, this question can be divided into two elements:

- Would infiltration cause geotechnical issues associated with subgrade strength, slope stability, soil volume change, utility damage, or other factors, either immediate below the roadway or in adjacent areas? This is a site-specific question. Guidance for evaluating this issue based on site-specific information is provided in White Paper #3. In summary, conditions exist where geotechnical issues can be mitigated, however some complex or challenging geotechnical conditions provide a valid technical basis for limiting or preventing infiltration from the road base.
- Does infiltration into subgrade (and resulting saturation) result in geotechnical properties of subgrade materials that cannot be accommodated in pavement structural design? This is also a site-specific question that requires knowledge of the response of subgrade materials to saturation. However, generally low strength subgrade can be accommodated by adding greater strength (i.e., depth) to the subbase and wearing course profile (Chai et al., 2012). In fact, design guidelines for standard pavements generally assume some degree of saturation and provide guidance for designing pavement structure when subgrade materials have limited strength. Subgrade materials with the potential for swelling upon saturation are generally problematic in the highway environment, regardless of whether intentional infiltration is proposed.

In summary, these factors do not necessarily preclude the use of permeable pavements, and must be addressed in traditional roadway design as well as permeable pavement. However these factors must be considered in planning and design, and, in some cases, the introduction of additional water into the subbase can introduce issues that cannot be reasonable mitigated. See White Paper #3 for more information.

3.1.7 Asphalt Draindown Concerns

Issue: Is there potential for hot asphalt mix to drain down through the permeable pavement surface layer, reducing permeability and causing a clogging layer within the pavement?

Typical permeability rates for new, properly constructed permeable asphalt layer have been measured from 1000 to more than 3,800 cm/hr (400 to more than 1500 in/hr) (summarized from multiple sources in Ferguson 2005), although the permeability of the pavement surface is expected to decrease over time. Clogging of several permeable asphalt systems has been anecdotally attributed to long- term draindown of asphalt binder in the pavement layer, however this would be expected largely for older mix designs without the admixtures (Ferguson 2005). Draindown may occur when the asphalt binder slowly liquefies during summer heat and drains down off the aggregate during hot periods, mixing with sediment, dust and other materials to form a clogging layer deeper in the pavement. According to the National Asphalt Pavement Association (NAPA) (2002b), this can also be observed during construction when the asphalt mix is at its highest temperature.

As noted by Kandhal and Mallick (1999), this issue can be addressed directly by the usage of admixtures. Contemporary mix designs are easily achieving this draindown requirement with the usage of fibers which increase the asphalt coating thickness and with liquid admixtures to increase stiffness.

Careful mix design (especially the selection of the asphalt binder grade, mix production temperatures and the use of additives) is recommended to reduce of the risk of draindown. The addition of fibers to the permeable asphalt mix has been shown to effectively manage draindown while adding material strength. Potential for draindown can also be detected as part of QC during installation. Excessive draindown evidenced by pooling of asphalt in the trucks is a key visual measure of QC concerns.

3.1.8 Groundwater Contamination Potential

Issue: Do highway applications of permeable pavement have a higher risk of groundwater contamination than other applications of permeable pavement?

Infiltration systems including, permeable pavements, pose new potential risks to groundwater quality compared to traditional pavements (Pitt et al. 1999). Careful siting of infiltration systems is an important element of design in general and must be observed in the installation of permeable pavements in the urban highway. White Paper #2 addresses the potential for groundwater contamination as a result of infiltration of highway runoff, and provides guidelines for evaluating the relative risk posed by site. Key factors include the depth of groundwater (higher risk associated with shallower depths), the nature of subsurface soils (permeability, organic versus inert, etc.), and the design of VRAs. In general, White Paper #2 concludes that infiltration of stormwater runoff can be safely done where conditions are suitable. Key factors that differentiate permeable pavements from other infiltration systems include: (1) net influence of permeable pavements on chloride loading, and (2) remediation issues in the event of contamination spills.

Perhaps the most significant groundwater contamination issue faced in high use transportation corridors is salts associated with winter maintenance activities on impermeable roadways. As a result, numerous chloride TMDLs are in process or in place in the United States. This presents a unique management challenge to both balance public safety and environmental protection because chloride concentrations are not reduced with standard best management practices, leaving source control as the primary strategy. Permeable asphalt has been shown to require substantially less chloride for winter maintenance top achieve and equivalent level of service (Roseen et al 2013). As such permeable pavements represent a management opportunity with reduced risk for both public safety and environmental protection. In this respect, the use of permeable pavements may have a net benefit to surface water quality and potentially groundwater quality in cold climates. However, careful consideration of salt management strategies is recommended in determining whether permeable pavements would pose an incremental risk to groundwater quality as a result of providing a more direct pathway for salts to migrate to groundwater.

Where routine handling of hazardous materials and risk of spill exists, minimizing the use of infiltration systems may be warranted (Pitt et al. 1999). Typically, full-depth permeable asphalt installations have been sited in locations where the risk to groundwater contaminations is low (i.e., parking lots and sidewalks). The urban highway environment present risks associated with potential for spills from large commercial vehicles. If a spill were to occur on a highway paved with permeable pavement the roadway section would need to be closed for longer remedial activities to occur on the pavement surface and in the subsurface aggregate storage bed and soils. The need for longer road closures to remediate spilled contaminants is unique to the use of permeable pavements. However, given the relative infrequence of significant spills, it may not be appropriate to exclude the use of permeable pavements on this basis alone. Such systems would help to contain spills prior to surface discharges which in some cases could be much more complex to clean up.

3.1.9 Construction Considerations

Issue: Are permeable pavements slower to construct than traditional pavement? Do they pose major issues in the construction process?

Qualified engineering and construction oversight are essential elements of permeable pavement construction because installation differs significantly from conventional pavements. Quality control measures, in general, are routine and well established in roadway construction and material production. In fact, performance payments based on QC targets are standard for large projects. These QC measures are easily being adapted for the use of permeable pavements. General construction sequencing and QA/QC considerations for full depth permeable pavements are included in Table 4.

Table 4. Summary of Construction, Installation and QA/QC Considerations

Element	Typical Applications in Non-Highway Environment	Considerations Specific to the Urban Highway Environment
Mobilization and site preparation	• Installation and maintenance of erosion and sediment control measures. • Installation of permeable pavement systems should occur after site stabilization for erosion control is in place.	• Site grading in the vicinity of the roadway should be conducted and areas stabilized prior to initiation of pavement installation.
Preparation of bed bottoms (i.e., subgrade)	• Ensure existing subgrade is not over-compacted; where subgrade compaction is unavoidable scarify subgrade to restore infiltration capacity. • Inspection of subgrade infiltration is a standard QC element prior to placement of constructed materials. • Ensure all bed bottoms are level or for sloping systems, ensure earthen berm or fabric barriers are constructed. • Obtain owner/engineer approval of the prepared subgrade.	• In general, designs in the highway environment should design for a moderate to high degree of unavoidable/necessary compaction of the subgrade below travel lanes and shoulders.
Installation of geotextile	• Install geotextile (if used) in accordance with the manufacturer's standards and recommendations. • Adjacent strips of geotextile should overlap a minimum of 40 cm (16 in.). • Secure geotextile at least 1.2 m (4 ft.) outside the bed to prevent runoff or sediment from entering reservoir course (remove excess once site is fully stabilized).	• Geotextiles may be an important element to control erosion in the subgrade where longitudinal slopes exist.
Aggregate placement and compaction and infiltration	• Aggregate shall be placed in 20 to 30 cm (8 to 12 in.) lifts and compacted until there is no visible movement. • Construction equipment shall be kept off the bed bottom and traffic on the aggregate should be minimized. • Once the reservoir course aggregate is installed to the desired grade, either the filter course (if included in design) and/or choker course (if included in design) is installed.	• If used, the filter course should be tested at compaction to maintain a minimum infiltration rate typically no less than 10 ft/day determined by ASTM D3385.
Permeable asphalt top course application	• The permeable pavement is installed directly over the aggregate base layers. • Placement temperatures and compactive effort should be per engineer's specifications. • Rollers should move slowly and uniformly to prevent	• When admixtures are added for strength, compaction may need to occur at higher temperatures.

Element	Typical Applications in Non-Highway Environment	Considerations Specific to the Urban Highway Environment
	displacement of the mix, and rollers should not be stopped or parked on the freshly placed mat. • Permeable asphalt should not be installed on wet aggregate or wet treated bases or when the ambient air temperature is below 13°C (55°F).	
Two lift application	• Permeable pavement surface shall be conducted in two lifts • Permeable asphalt surface course shall be installed over the ATPB where required for high traffic loads and over choker course where lighter traffic loads allow.	• ATPB is typically needed for strength in higher load applications and may also be used to temporarily accommodate construction traffic prior to surface paving (Hansen 2008) • ATPB must be covered with geotextile and protected from sediment and must be vacuumed and flushed with water prior to the final lift of permeable asphalt paving
Curing of permeable asphalt	• No vehicular traffic is permitted on the pavement until cooling and hardening (curing) • Typical curing time: 48 hours minimum	• To ensure adequate curing time lane opening must be sequenced to ensure flow of traffic
Concrete top course installation and curing	• Typically formed and leveled by hand in traditional parking lot and sidewalk applications • Permeable concrete must not be “floated”, as this can seal the surface • Curing typically involves covering with sheet plastic for a curing period of at least 7 days.	• Faster installation methods may help improve viability in the highway environment. • Schaefer et al. (2011) successfully tested a self-consolidating and slip-formable mix in a MNDOT right of way. • Kevern and Farney (2012) have studied admixtures to improve curing time.

3.1.10 Capital Costs

Issue: Is permeable pavement is cost prohibitive?

Permeable pavements are commonly thought to add substantial cost to a project and are therefore thought to be cost prohibitive. In fact, for new development, total project costs for permeable pavements can result in significant cost reductions when the result is avoidance or downsizing of stormwater infrastructure (e.g., BMPs, ponds, piping, control structures) (Roseen 2013, Gunderson 2010, MEDOT, 2010, Wang et al. 2010).

Permeable asphalt surfaces are generally 20 to 50 percent higher in costs than conventional asphalt on a unit area basis. This is due to the use of additives (e.g., fibers, polymers) to control draindown and increase stability. Costs for permeable asphalt (not including the reservoir course) range from about \$21 to \$38 per m^2 (\$2 to \$3.50 per sf). As a complete system (inclusive of subsurface layers), the estimated cost is \$65 to \$130 per m^2 (\$6 to \$12 per sf). In 2009 costs for the Maine Mall Road were \$180/ton for permeable asphalt and $105/ton for the ATPB (Hodgman 2012). In 2013, costs were reported to be approximately an additional $14/ton for an 800 ton placement in Provincetown, MA (Roseen 2013). Additional costs were associated for one-time costs for usage of temporary equipment to handle specialty binder.

Much of the increased unit cost of permeable asphalt pavement systems comes from the stone storage reservoir, which in some instances is thicker than the aggregate base for conventional pavement. However for high use roadways, structural requirements for subbase thickness will often exceed needs for runoff storage and frost design. Additionally, costs can be offset by the significant reduction in the required conventional stormwater management elements (inlets, pipe, basins, right of way, curb and gutter, etc.). Furthermore, permeable pavements with reservoir layers often avoid the need for end-of-pipe detention basins or other, similar stormwater management systems. When these factors are considered, permeable pavement with infiltration can be approximately the same cost or often even less expensive than conventional pavement with its associated stormwater management facilities. For example, a 6 percent project savings was realized for a permeable asphalt residential road application in New Hampshire by avoiding the use of curbing, pipe, catch basins, detention basins and outlet control structures (Gunderson 2010). MaineDOT conducted a costing analysis and found the permeable asphalt roadway to be 2 percent more than a conventional roadway for new construction. Similarly, Wang et al. (2010) found that for California highways, permeable shoulders would have an economic advantage over traditional pavement with associated stormwater controls, even when a top course lifespan of only 10 years was assumed.

It is important to note that findings of potential cost savings applicable only to new construction or lane additions. Costs for including permeable pavement in refinishing and retrofit projects can be prohibitive because of the need for reconstruction of the subbase (i.e., the need to remove the traditional subbase layer and replace it with an open graded subbase), and the associated import and export hauling and disposal required. Refinishing or retrofit projects would also not recognize savings from reduced drainage and treatment infrastructure since these types of infrastructure are already in place; therefore, costs tend to be higher for redevelopment and resurfacing projects done with permeable pavement rather than conventional pavement. Additionally, where a supplemental drainage pathway is needed in critical highway segments (such as “sag” and “depressed” sections) to provide a failsafe against flooding, the economics of permeable pavement can be less favorable. Project-specific analysis of costs, considering costs that are incurred as well as avoided, is recommended.

3.1.11 Maintenance Costs

Issue: Are maintenance activities are cost prohibitive and/or impracticable for highway applications?

The success of permeable pavement systems relies not only on proper design and construction but also on effective maintenance. The primary goal of permeable asphalt system maintenance is to prevent the pavement surface and underlying storage/infiltration bed (if applicable) from becoming clogged by sediment and debris. Maintenance is often viewed as a barrier to implementation of low impact development (LID) technologies due to the minimal documentation of frequency, intensity and cost of maintenance for these practices. However, proponents of permeable pavements regard these practices as lower in maintenance when compared to other conventional stormwater controls (MacMullen, 2007; Powell et. al., 2005; EPA, 2000).

In 2012, the University of New Hampshire (Houle, et. al., 2012) compiled the results of a 6-year study (2004-2010) on maintenance. The primary maintenance activity indicated for the permeable asphalt parking lot in this study was pavement vacuuming. In this study the permeable asphalt system was found to have the lowest maintenance burden in terms of personnel hours and second lowest annual maintenance costs when compared to the seven other stormwater control measures included in the study. The lowest annualized maintenance costs expressed as a percentage of capital costs were permeable asphalt (4%) followed by the vegetated swale (6%), the subsurface gravel wetland (8%), and the bioretention systems (8%). At this rate, annual LID system maintenance expenditures equal total upfront capital costs after 24.6 years for the permeable asphalt system.

Additionally, researchers have found that vacuum sweeping of PFCs does not appear to be necessary as a result of the pressure/suction effects of high speed vehicle traffic (Stanard 2007). However, it has not been established whether similar effects would be observed in full depth permeable pavements since drainage conditions below the permeable top course are different.

Stormwater managers in the Pacific Northwest have (Personal Communication, Mark Maurer) have noted that they have observed problems with moss growth in permeable pavements at multiple sites that significantly reduced the infiltration rate and caused slip hazards. They have not found any method other than moss poisons that will remove the moss. When moss is pressure washed to remove the surface, the roots stay in the pores and regrow quickly. They are having to pressure wash to clean some sections 3 to 4 times a year, which is a significant maintenance burden.

While this suggests that maintenance should not pose a barrier for permeable pavements in general, there are specific maintenance issues exist with respect to the urban highway environment that may represent significant barriers in some cases. Table 5 summarizes the typical maintenance requirements for permeable pavement installations and considerations of implementation in the urban highway environment.

Table 5. Summary of Maintenance for Permeable Pavement

Maintenance Activity	Summary of Available Guidance	Considerations Specific to the Urban Highway Environment
Pavement Vacuuming	• Typical practice is to specify vacuuming the pavement surface twice per year in the late fall and early spring	• Vacuuming typically is performed at low speeds and therefore may require special provisions (e.g., traffic control) in a highway setting • Twice per year is recommended for typical applications—in areas that receive unusually high amounts of sediment, vacuuming should be done more frequently • Self-cleaning aspects of PFC notes by researchers under high speed traffic may apply to some degree for full depth pavements located in travel lanes
Pressure Washing	• Studies using PFC have shown good results from vehicles that contain both pressure washing and vacuum equipment to remove accumulated particles in the pavement surface	• Pressure washing is not likely practical in the urban highway environment due to the area required to be treated and the time required to implement treatment
Moss Removal	• In wet climates, moss can develop on permeable pavement and severely limit its permeability. Moss removal would require frequent pressure washing and vacuuming, which may still not be effective.	• Moss removal activities may be cost prohibitive and in the urban highway environment. • While this phenomenon may occur in only limited geographic regions and locations within a project (e.g., shaded spot, with poor exposure), this could be a fatal flaw for the use

Maintenance Activity	Summary of Available Guidance	Considerations Specific to the Urban Highway Environment
		and upkeep of permeable pavements.
Maintenance Costs	• In 2012, the University of New Hampshire (Houle, et. al., 2012) compiled the results of a 6-year study (2004-2010) for seven different types of stormwater control measures, including a permeable asphalt pavement system, logging all inspection hours and maintenance activities. They calculated a cost of $2700/ha/yr for permeable pavements, the lowest of the annual maintenance costs for LID systems and lower than wet and dry ponds by more than 50 percent • Others (e.g., Narayanan and Pitt, 2006) have found relatively minor incremental cost of permeable pavement maintenance compared to traditional pavement.	• Maintenance costs in the urban highway environment would be dependent on: o Need to close lanes for sweeping o Specialized equipment and operators compared to standard DOT equipment o Need for lane closure for remedial actions in the event of a contaminant spill • Given that maintenance is critical for long-term operation, in some conditions, maintenance costs and practicality could be a significant limitation for use of permeable pavements in urban highway environments.
Asset Tracking and Management	• Like components of other stormwater management systems, permeable pavements need to be tracked and provisions need to be in place to ensure that activities on top of or adjacent to these systems do not compromise their function (e.g., slurry seal over permeable pavement)	• DOTs typically have well established asset management systems, however permeable pavement have special considerations that may not fit within these systems. Systems should be adapted to permeable pavement before it is used on a widespread basis

3.1.12 Cold Climate Performance and Winter Maintenance

Issue: Will freeze/thaw crack and damage permeable pavements? Does permeable pavement function in colder climates? Does winter sanding and deicing clog the pavement surface? Do winter snow removal activities (plowing) destroy the permeable pavement surface?

Winter performance and maintenance is a common concern raised in in the context of permeable pavements and has been relatively well studied. Permeable pavements have been found to be more resistant to freezing than standard pavements due largely to the rapid disconnection from subsurface moisture and from rapid thawing due to rapid infiltration of meltwater (Backstrom, 2000). Roseen and Ballestero (2008) state that if designed properly, permeable pavements have demonstrated cold-climate performance, including drainage under freezing conditions, exceeding conventional practices by measures of both water quality and hydraulics.

Abrasives such as sand or cinders for winter traction must not be applied on or near the permeable pavement. While sanding is not recommended on PFC or full depth pavements, sanding is believed to occur on PFC applications in some states. However, the potential impacts of sanding PFC are different than full depth permeable pavements. First, in the event of clogging of PFC with sand, surficial drainage still occurs and many of the intended benefits of PFC are maintained. However, in full depth permeable pavement applications, sand has the potential to clog the surface course and/or migrated into the subbase and cause clogging there, either of which would significantly alter the intended hydrologic performance. As such, it is recommended that abrasives not be used at all on sites with permeable pavement to minimize sand being tracked onto the pavement by vehicles and to prevent accidental applications to the permeable asphalt surfaces. As with any pavement, frequent snow plowing is an essential component of maintenance in cold climates but should be done carefully to avoid damaging the surface with heavy equipment (Roseen, et.al., 2013). Additionally, in areas where studded tires are allowed and heavily used, frequency of maintenance of both traditional and pervious pavements is expected to increase.

Permeable pavement is less likely to form black ice and often require less plowing and fewer deicing chemicals. University of New Hampshire Stormwater Center found that a permeable asphalt parking lot reduced the salt needed for winter maintenance by approximately 50 to 75 percent that of a standard impermeable asphalt parking lot (Roseen et al 2013; UNHSC, 2010). Permeable asphalt pavements can be more challenging to de-ice if compacted snow and ice develop. The reduction in the amount of deicing materials is primarily observed with respect to the reduced development of black ice. Success will vary by application and sun exposure. Ultimately, if reduced winter deicing practices are acknowledged (UNHSC 2009b), permeable asphalt could have a winter maintenance cost benefit.

Permeable concrete has not been commonly used in cold climates because of concerns related to the effects of chloride deicers on curing and strength. Recent research (Schaefer et al. 2011) concluded that these concerns can be addressed in part. However, this topic requires further research and should be considered in selecting between permeable asphalt and concrete.

3.2 Summary of Potential Urban Highway Opportunities and Limitations

What type(s) of project conditions present high opportunity for permeable pavement applications?

What site information is needed to help determine whether permeable pavement is applicable?

As introduced in Section 3.1, the applicability of full depth permeable pavement for urban highways depends on a wide range of factors. While permeable pavement technologies and design/QC approaches have evolved and will continue to evolve, there are certain factors that apply in some cases that cannot be overcome. As a result, permeable pavements may be prohibitive for some project conditions and applications, but favorable for others. The purpose of this section is to synthesize the topics discussed in Section 3.1 to provide concise guidance for evaluating the potential suitability of permeable pavement for site-specific conditions.

Table 6 summarizes the factors that influence permeable pavement applicability and identifies data that can be relevant to help decide whether permeable pavements are a suitable and favorable VRA. Table 6 also summarizes the information that needs to be collected to support this evaluation. Table 7 provides a checklist for documenting the results of site-specific screening of permeable pavements.

Table 6. Summary of Opportunities and Limitations for Full Depth Permeable Pavement

Factors Influencing Applicability	Site Assessment and Project Characterization Needs	Favorable Conditions	Unfavorable/Challenging Conditions

Factors Influencing Applicability	Site Assessment and Project Characterization Needs	Favorable Conditions	Unfavorable/Challenging Conditions
Soil and Geotechnical Factors (strength, permeability, and hazards)	• Soil texture • Infiltration rates at various degrees of compaction • Assessment of strength under wetted conditions	• Granular soils tend to be more favorable • Soils that display limited loss in strength when wet • Soils that maintain some permeability when compacted	• Fine grained and plastic soils are more challenging • Soils that exhibit shrink/swell properties • Soils that exhibit substantial loss of strength when wet may require major • Soils that exhibit substantial loss of permeability when compacted (less than approximately 0.01 inches/hour
Project Strength and Durability Requirements	• What are the pavement system strength and durability requirements for the intended application? • What are is the strength of the underlying soil when saturated?	• Lower strength requirements, such that thickness of subbase required for hydrologic goals would meet strength goals • Shoulder applications	• Higher strength requirements that would require expensive additives and design features (subbase thickness) or cannot be achieved with currently available technologies • Travel lane applications
Groundwater Conditions/ Contamination Potential	• Depth to seasonably high groundwater table • Existing soil contamination • Soil texture, organic content, permeability • Use of deicing chemicals	• Greater than 2-10 feet between reservoir bottom and seasonally high groundwater table (varies by jurisdiction) • Steeper local groundwater gradient helps dissipate mounding • Soils provide natural pollutant attenuation via organic content and cation exchange	• Aquifer recharge areas where infiltration could impact groundwater quality • Natural or anthropogenic soil contamination or plumes • Shallow seasonally high groundwater table and coarse inert soils • Flatter groundwater gradient; higher mounding potential
Hazardous Material Spill Liability	• Potential for hazardous spills based on surrounding land uses that could require removal of pavement and subbase	• Typical urban highway projects without elevated risk factors	• High use sites with elevated volume of truck traffic and/or industrial activity • Where the risk of concentrated pollutant spills is more likely such as gas stations, truck stops, and industrial chemical storage sites
Pavement Surface Drainage Requirements	• What are the applicable drainage design requirements? • Can supplemental drainage pathways be provided?	• Design drainage flowrates (for peak events) are much less than capacity of permeable pavement (allowing margin of safety for clogging) • Supplemental overflow pathways are possible	• Any condition where clogging of permeable pavement could create a condition where travel lanes do not drain to meet applicable drainage design criteria

Factors Influencing Applicability	Site Assessment and Project Characterization Needs	Favorable Conditions	Unfavorable/Challenging Conditions
Cold Climate	• Freeze-thaw potential • Winter maintenance practices • Agency policies on level of service during cold weather	• Temperate climates • If cold climate, then sanding should not be used • If cold climate, Agency would consider permeable pavement as a traction improvement to reduce salt application	• Cold climates with sand usage and/or institutional barriers to alternative maintenance practices • Extensive use of studded traction tires
Moss Growth Potential	• Observation of moss growth on roadways in the project vicinity/region • Exposure of site	• Any region where moss growth is not a concern • Sunny sites, with good wintertime exposure and limited potential for tree canopy growth to change exposure conditions	• Regions that experience problems with moss growth on shoulders • Sites with poor exposure, particularly with potential for additional shading as trees grow • Roadway segments below grade that maximizes winter shading
Type of Highway Segment	• Width of shoulder/median • Adjacent slopes • Is back-up drainage critical?	• Wider shoulder/median helps provide greater area for permeable shoulders and low-speed sweeping practices • Relatively mild cross slope helps reduce geotechnical issues • Areas where back-up drainage is not needed	• Segments on berms, or in highly constrained sections • Segments on steeper cross slopes • Sag segments or constrained segments where supplemental drainage must be provided as a failsafe for peak flow conveyance
Type of Construction	• New construction or refinishing / retrofit • Export and import needed to rebuild subbase • Existing drainage/treatment infrastructure already in place	• New construction or lane additions where use of permeable pavement can avoid traditional asphalt and conveyance/treatment costs or retrofit construction where subbase is already being re-built	• Rebuilding subbase would be a major additional cost • Drainage infrastructure is already in place and would not otherwise need to be upgraded • Treatment already provided and would not otherwise need to be upgraded
Longitudinal Grade	• Review site plans	• Flatter segments (less than 1 to 2 percent) where internal berms/fabric cutoff walls could have a greater spacing	• Steeper segments greater than 2 percent pose more challenges for construction, but may be feasible. • In some cases, the compounding influence of decreased infiltration potential and increased frequency of berms could render permeable pavement cost prohibitive as a result of longitudinal grade

Factors Influencing Applicability	Site Assessment and Project Characterization Needs	Favorable Conditions	Unfavorable/Challenging Conditions
Adjacent Ground Cover	• Site plans • Characterization of area draining to permeable pavement	• Permeable pavement accepts runoff from only impervious areas • Vehicle tack-on from adjacent areas is limited	• Landscaped/pervious areas drain toward permeable pavement (e.g., depressed segment) • Areas with disturbed soils drain toward permeable pavement • Road experiences long-term track-on from adjacent streets/land uses
Department Acceptance/ Experience	• Evaluate local case studies and research • Review local specifications and design guidance	• Local experience and familiarity with permeable pavement design and maintenance • Prior successful installations • Interest in permeable pavement as a pilot project	• Negative perceptions about permeable pavement technologies • Lack of experience and/or prior installations
Local Contractor Experience and Owner QA/QC Experience	• Evaluate past experience/performance with permeable pavement • Visit previous installations and evaluate current performance • Evaluate owner construction QA/QC protocols	• Contractor has sufficient past experience (1+ successful installations) • Contractor's past sites appear to be functioning properly • Owner has experience with permeable pavement testing and acceptance (or can obtain it)	• Contractor has no experience installing permeable pavement • Past installations of permeable pavement conducted by Contractor have failed or are exhibiting attributes of failure and contractor has not changed its methods to correct the issues • Owner does not have processes in place to control quality of construction
Local Supplier Experience	• Evaluate potential local sources of specialized permeable asphalt and concrete mixes	• Suppliers are familiar with mixes and willing to develop custom batches • Existing installations have been successful	• Suppliers are not familiar with specialized mixes and/or are resistant to developing specialized mixes • No history of successful implementations

Factors Influencing Applicability	Site Assessment and Project Characterization Needs	Favorable Conditions	Unfavorable/Challenging Conditions
Owner Maintenance Equipment, Capabilities, and Asset Management	• Evaluate personnel resources available for pavement maintenance • Evaluate financial resources available for pavement maintenance • Evaluate current asset management and tracking framework	• Budget available to purchase a vacuum street sweeper for department use and staff available to operate equipment according to maintenance schedule OR budget available to hire an outside company to conduct regularly scheduled maintenance • Systems in place to track installations and maintenance and ensure that they are not compromised by other road treatments	• Budget not available to purchase vacuum street sweeper or to hire an outside company to conduct required maintenance • Systems not in place to track installations and maintenance and/or significant justified concerns that installations would be compromised by other road maintenance
DOT and/or Local Contractor Experience with Maintenance	• Are maintenance contractors available in the location with experience maintaining permeable pavement? • Can the DOT self preform the maintenance?	• Contractors and/or DOT staff with experience	• No contractors or DOT staff with experience

Table 7. Checklist of Permeable Pavement Selection and Design Considerations

Permeable Pavement Selection and Design Considerations	Results of Project-Specific Screening (check the box that applies)		
	Evaluated - not an issue	Potential issues must be addressed in design, spec, and construction	Prohibitive
Soil and Geotechnical Factors			
Project Strength and Durability Requirements			
Groundwater Conditions/ Contamination Potential			
Hazardous Material Spill Liability			
Pavement Surface Drainage Requirements			
Cold Climate			
Moss Growth Potential			
Type of Highway Segment			
Type of Construction			
Longitudinal Grade			
Adjacent Ground Cover			
Department Acceptance/ Experience			
Local Contractor Experience			

Permeable Pavement Selection and Design Considerations	Results of Project-Specific Screening (check the box that applies)		
	Evaluated - not an issue	Potential issues must be addressed in design, spec, and construction	Prohibitive
Owner QA/QC Experience/Protocols			
Local Supplier Experience			
Owner Maintenance Equipment, Capabilities, and Asset Management/Tracking			
DOT and/or Local Contractor Experience with Maintenance			

3.3 Permeable Shoulders versus Travel Lane Opportunities

Generally, DOT-related practice and research tends to be emphasizing opportunities for permeable shoulders rather than full width permeable pavement across travel lanes. Shoulders tend to have lower traffic load and are more accessible for maintenance, which may be compelling reasons to emphasize shoulders. Shoulders may provide adequate area for stormwater management. The compendium of factors introduced above suggests that shoulders likely provide a better balance between benefits and constraints. However, some advantages of permeable pavement in travel lanes should also be considered in deciding where to implement permeable pavements. Table 8 provides a summary of advantages and disadvantages associated with permeable shoulders and permeable travel lanes.

Table 8. Comparison of Opportunities for Permeable Shoulders and Permeable Travel Lanes

	Advantages	Disadvantages
Permeable Shoulders	• More widely accepted and studied • Tend to have lower traffic load • May support lower levels of compaction in some cases • Tends to provide better access for maintenance • Typically adequate area to capture and manage a water quality design volume • Failure less likely to have impact to travel lanes	• Permeable shoulders may require higher infiltration rates to achieve the same level of volume reduction due to smaller footprint and run-on from adjacent areas • Can be prone to clogging, particularly if maintenance is limited or high sediment production from adjacent travel lanes or landscaping
Permeable Travel Lanes	• Can achieve higher levels of volume reduction in lower infiltration rate environments • Experience suggests that water pressure from high speed traffic may help keep pores open • Can realize other benefits such as reduction in splash, reduction in noise, safer cold weather conditions, and better ride quality	• Less widely accepted/studied • Must design for higher loads and higher traffic volumes • Design lives expected to be shorter, particularly in areas with significant studded tire use • Maintenance with vacuum sweeper would require lane shut down • Remediation of spilled contaminants would require lane(s) to be shut down

4 General Approach for Permeable Pavement Design Integration

What is a recommended work flow to help reduce complexity and foster common understanding?

Based on the considerations described in Section 3, permeable pavements may be applicable in some cases in the urban highway environment. In cases where project conditions appear favorable for permeable pavement and the design team has selected permeable pavement as a project VRA, how should the design team approach the challenges associated with designing of these systems? How can barriers associated with design complexity be overcome? This section is intended to introduce a general stepwise process for helping design teams incorporate permeable pavements into urban highway designs. This process relies heavily on the project team's familiarity with existing design guidance, such as AASHTO publications, and primarily discussed how this guidance can be adapted to accommodate permeable designs.

4.1 Conceptual Steps for Design Integration

As introduced above, the design permeable pavement is fundamentally different from other VRAs, as permeable pavement serves a dual role – as a structural surface for vehicular traffic and as a hydrologic/hydraulic control. This can complicate the design process. Once the design team has decided to utilize a permeable pavement design or at least developed a detailed design for evaluation, the following steps have been developed to help provide guidance for approaching this process as a project design team.

Step 1 – Site Characterization and Screening of Permeable Pavement Suitability. Successful permeable pavement design and installation begins with good site characterization and screening as described in Section 3. Other alternatives should be considered as part of the overall screening and prioritization process described in the Guidance Manual. The project team should arrive at permeable pavement as a potential option after conducting an initial site suitability and feasibility screening process. Implementing permeable pavement also requires a commitment to coordination between multiple design disciplines as well as a commitment to maintenance of the finished product. When permeable pavement is being considered, it is recommended that a design workshop be conducted at the outset of the design process to obtain multiple discipline and department buy-in for the coordination needed to implement permeable pavement. While each agency may approach this step differently, this step should not be overlooked.

Step 2 – Develop Initial Hydrologic and Structural Designs. Hydrologic and structural designs are based on fundamentally different considerations; and inherently require iterations to balance these considerations. As an initial step, these disciplines may work independently to develop designs that meet the respective functional purpose. As a design team gains familiarity with respective criteria that apply to each discipline, the need for iterations may decrease. Sections 4.2 and 4.3, below, provide guidance for conducting each of these analyses.

Step 3 – Compare Structural and Hydrologic Designs and Develop Design Modifications to Balance Structural and Hydrologic Considerations. Differences between the design requirements for the respective functional purposes must be compared and reconciled to yield a design that effectively and efficiently serves both design functions. A number of adjustments, such as increasing subgrade compaction, including different structural elements, and other factors may help balance these considerations. Section 4.4, below, provides guidance for adapting design features to balance these considerations.

Figure 1 illustrates this conceptual design process.

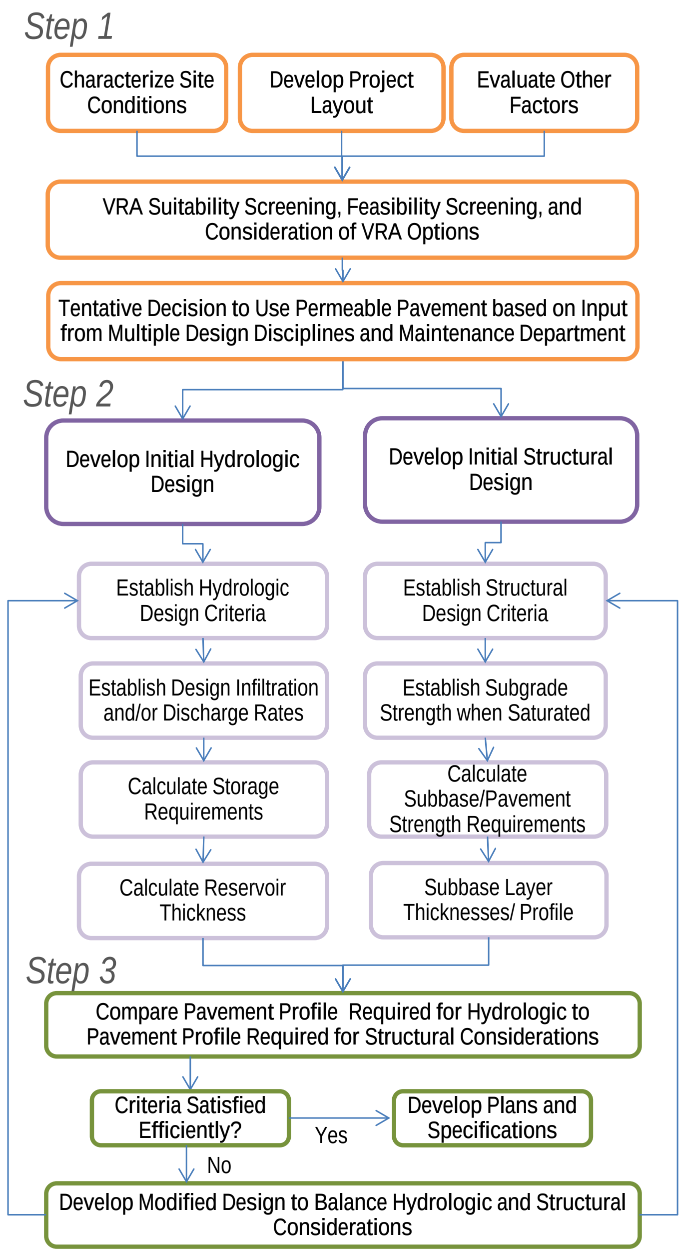

Figure 1. Conceptual Process for Permeable Pavement Design

4.2 Overview of Structural Design Approach

Structural design approaches differ between flexible pavements (i.e., asphalt and paver blocks) and rigid pavement (i.e., concrete). Fortunately, well proven methods used for the design of traditional pavements can be adapted to the design of their permeable counterparts through the use of different input parameters. In general, permeable materials are not as strong as their traditional counterparts, and a higher degree of saturation in the subgrade material must be considered, which can reduce bearing strength in some types of soil. Finally, because permeable materials have not been as well tested as their counter parts, designs typically include factors of safety, and/or are designed for higher reliability.

Design guidance and tools for flexible and rigid permeable pavement design can be found from the respective trade organizations:

www.asphaltpavement.org/index.php?option=com_content&view=article&id=359&Itemid=863

http://acpa.org/PerviousPave/

These design approaches, and adaptations for permeable pavement design, are introduced below.

Flexible Pavement

Many local transportation agencies use the empirically-based AASHTO 1993 *Guide for Design of Pavement Structures* whose underlying concepts emerged from test pavements (many with dense-graded bases) in the 1950s repeatedly trafficked by trucks that established relationships among materials types, loads and serviceability. The AASHTO equation in the 1993 Guide calculates a required Structural Number or SN given traffic loads, soil type, degree of compaction, climatic and soil moisture conditions, and associated assumptions about subbase strength (ASCE, 2013 DRAFT). The designer then finds the appropriate combination of pavement surfacing and base materials whose strengths are characterized with layer coefficients. When these layer coefficients are added together, the sum product of the coefficients (a_i) multiplied by the pavement material thicknesses (d_i) should meet or exceed the SN required for a design (where $SN = \Sigma a_i \times d_i$).

This empirical design approach is applicable to permeable pavement with consideration given to layer coefficients that represent permeable pavement system components (ASCE, 2013 DRAFT). Table 9 provides recommended AASHTO layer coefficients for structural evaluation of permeable asphalt. Based upon the recommended SNs the recommended minimum thicknesses for the permeable asphalt surface for roadway traffic loadings ranges from a minimum of 4.0 inches for a residential street (some truck traffic) to 6.0 inches for heavy truck traffic.

Table 9. AASHTO Layer Coefficients for Permeable Asphalt Systems

Material	Layer Coefficients in Traditional Design	Layer Coefficients in Permeable Design
Top Course Asphalt	0.40 – 0.44 (Dense mix)	0.2 - 0.3 (permeable mix, possibly higher with admixtures)[1]
Asphalt Treated Permeable Base (ATPB)	0.30 – 0.35	0.30 – 0.35
Open-Graded Aggregate Base Layers	0.12 – 0.14 (Dense Graded)	0.06 – 0.09 (Open Graded)

Source: Hansen, 2008; Hein, 2006; Hein, et al. 2013.

[1]The Oregon Department of Transportation uses the same structural coefficient for PFC as dense-graded HMA (NAPA, 2002).

Rigid Pavement

AASHTO, ACI, or PCA design procedures for rigid pavements are appropriate for permeable concrete design, however strength and durability relationships have not been as well established or field verified, so designs should provide a higher design conservatism (Delatte, 2007). Pavement designers should assume lower flexural strength, lower modulus of elasticity, and should characterize subgrade design strength assuming saturated conditions. Cackler et al. (2012) summarized that results reported in the literature show a linear decrease in modulus of elasticity values for permeable concrete with increased void content, with a similar to relationships for strength. Reported modulus of elasticity for permeable concretes with 25% voids was around 3.6 million psi (24,800 MPa) and decreased to 1.5 million psi (10,300 MPa) at 40% voids, for mixtures using limestone aggregate (Crouch et al., 2007). Consistent with Cackler's summary, Hein et al. (2013) states that the flexural strength of permeable concrete typically ranges from about 2 to 3 MPa (290 to 435 psi) in comparison to about 4.5 to 6.5 MPa (650 to 945 psi) for traditional concrete.

4.3 Hydrologic Considerations

In addition to the structural considerations, the pavement system must be sized to meet the hydrologic/stormwater management design requirements required by site-specific regulations. To determine the storage required for stormwater management, all system components including infiltration rate of the underlying soils and underdrain system outflow must be taken into consideration. Similar to the other VRAs described in these guidelines, permeable pavement can be considered to consist of a storage volume and an associated time and pattern by with that storage volume is discharged. Using the "Volume Performance Tool" developed as part of these Guidelines, hydrologic analysis can be conducted to estimate long-term volume reduction as a function of local climate and various design parameters. Other hydrologic design approaches, such as continuous simulation or event-based simulation, using locally-approved methods can be used to evaluate the storage and discharge characteristics needed to meet hydrologic design goals/requirements. This will be expressed in terms of a stone reservoir layer and outlet elevations and sizes. However, for evaluation of long-term performance of surface runoff volume reduction, a method that utilizes long-term simulations of precipitation and runoff/infiltration is required.

For design purposes, an appropriate factor of safety should be applied to the measured site soil infiltration rate to account for compaction of subgrade and long-term clogging effects (see White Paper #1). The design infiltration rate may be influenced by the type of soils encountered, the degree of compaction required, and the amount of anticipated sediment loading from run-on from traditional pavement. In some hydrologic design processes (such as a continuous simulation performance-based design process, the infiltration rate may be a factor in determining the required storage volume.

4.4 Balancing Structural and Hydrologic Considerations in Permeable Pavement System Design

The result of the structural design process will be layer thicknesses, material types, and associated assumptions about the degree of compaction of subbase materials. Similarly, hydrologic design requirements will be expressed in terms of a thickness of subbase and assumed compaction/permeability of subgrade materials. The hydrologic design process also includes assumptions about the slope of the bed, outlet controls/underdrains, permeability of top course and filter materials, if used, and the porosity of subbase reservoir. As such, the overlapping design parameters primarily include: subbase thickness, subbase strength (which are a function of porosity), and subgrade strength (which may be a function of degree of compaction and saturation).

A "balanced" design is found when both structural and hydrologic criteria are met, and the overlapping parameters converge such that the design achieves efficiency. An imbalanced design would occur when one design function results in a depth much greater than needed to serve the other design function. If the base/subbase thickness required for hydrological design is significantly thicker than required for structural capacity, the designer may modify some of the design parameters to make the design more cost effective. This may include:

- Modify the base/subbase materials to increase porosity to provide greater storage (but sacrificing strength).
- Modify the base/subbase materials to increase permeability (e.g., less compaction) to permit more rapid drainage of water from the pavement section (but sacrificing strength).
- Increase the frequency, diameter or slope of outlet pipes to increase supplemental water outflow.
- Increase the area of permeable pavement to non-permeable pavement if a shoulder application or otherwise includes significant run-on from up-slope non-permeable pavements.

If the base/subbase thickness required for structural design is significantly thicker than required for hydrological design, this may result in excess capacity for hydrologic functions. The designer can choose to accept this surplus as a factor of safety against long-term clogging, or could pursue one of the following options to increase strength and reduce hydrologic function as part of balancing the design:

- Increase the thickness of the surface layer. These layers have a higher structural capacity than the base/subbase layers. Minor increases in the thickness of these layers may allow a reduction in the thickness of the subbase and associated storage volume.
- Add an ATBP layer, if designing permeable asphalt, to help reduce the required thickness of the based and associated storage volume.
- Improve the quality of the base/subbase layers to provide a higher layer coefficient, while potentially reducing the porosity (thus sacrificing some volume).
- Providing additional compaction of the subgrade, thus potentially increasing strength by potentially reducing the infiltration rate.

A number of other factors, such as interface with adjacent pavement structures, may also control design. Ultimately, a balanced design is not essential, as long as both structural and hydrologic design criteria are met at a cost that is determined to be feasible and affordable.

5 Options for Further Research

What are high priority needs and opportunities for further research?

Areas of future research can generally be categorized as:

- Water quality and quantity performance
- System design
- Construction methodologies
- Quality control testing
- Maintenance needs

Many studies have been conducted and continue to be conducted on the design and installation of permeable asphalt systems. Most studies focus on full-depth permeable asphalt designed to manage stormwater from low to moderate traffic parking areas, sidewalks, pathways and driveways. Studies and implementation of full-depth permeable asphalt systems on low-speed/low volume roadways and highways is lacking. Further research is needed on roadway and highway installations of full-depth permeable asphalt to obtain a greater depth of knowledge on the cost and operation and maintenance required for these systems long term. Additionally, the water quality benefits of full-depth permeable asphalt require further research; specifically water quality information from underdrained permeable asphalt systems. These systems act as underground detention systems but preliminary research indicates greater water quality benefits than many traditional stormwater detention systems. Potential additional research topics include:

- Evaluation of permeable asphalt mix designs and admixtures
- Development and adaptation of a field QC standard for compaction testing of permeable asphalt
- Development and adaptation of a laboratory QC standard for pavement durability testing
- Evaluation of compaction methods for permeable asphalt pavements
- Development of vacuum sweepers specifically for the high volume maintenance of permeable pavements
- Recommended curing time for permeable asphalt as a function of pavement strength and temperature
- Study of road sanding and permeable pavements

6 Summary

Permeable pavements have a number of potentially compelling benefits in the urban highway environment. However, they also have a number of key limitations that may preclude their use in many conditions. As with all stormwater controls, the long-term success of permeable pavements depends on (1) using permeable pavements only in locations that are suitable for use, (2) careful design, specification, and construction QA/QC of permeable pavements to meet structural and hydrologic objectives, and (3) effective ongoing maintenance. If project circumstances do not allow for these requirements to be met, then permeable pavements are not a good choice for the project. However, where these barriers do not exist or can be overcome, permeable pavements may provide cost and effectiveness benefits (compared to traditional stormwater controls) that justify their usage.

7 References

AASHTO (1993). *Guide for Design of Pavement Structures*, Washington, D.C.

American Concrete Institute (ACI) (2008). "Specification for Permeable Concrete Pavements." 522.1-08, Committee 522.

American Concrete Paving Association (ACPA) (2012). *Pervious Pave – Background, Purpose, Assumptions and Equations*, Washington, D.C.

Alvarez, A.E., and A.E. Martin. (2009). "Permeable Friction Course Mixtures are Different." Rocky Mountain Asphalt Conference & Equipment Show, Colorado State University, Fort Collins, CO.

ASCE (2013 in draft). *Recommended Design Guidelines for Permeable Pavements. Manual of Practice on Recommended Design Guidelines for Permeable Pavements*, B. Eisenberg, K. Lindow, and D. Smith, eds., American Society of Civil Engineers, The Permeable Pavements Technical Committee, Low Impact Development Standing Committee, Urban Water Resources Research Council, Environment and Water Resources Institute. Co-authors: Bean, E. Z., Braga, A. M., Clar, M. L., Lindow, K. C., Early, K., Eisenberg, B. E., Fassman, E. A., Gwilym, K. M., Haselbach, L. M., III, W. F. H., Karimipour, S., Kevern, J. T., Kunkler, L., Lowry, T. P., Perry, S. D., Potts, A., Putz, R., Roseen, R., Rowe, A., and Smith, D. R.

ASTM (2012). Standard C-1754/1754M, "Standard Test Method for Density and Void Content of Hardened Permeable Concrete," Annual Book of ASTM Standards Vol. 4(2), ASTM International, ASTM International, West Conshohocken, PA.

Backström, M. (2009). *Permeable Pavement in a Cold Climate*, Licentiate Thesis, Lulea University of Technology, Lulea, Sweden. http://epubl.luth.se.

Barrett, M. E. (2008). "Effects of Permeable Friction Course (PFC) on Highway Runoff." 11th International Conference on Urban Drainage, Edinburgh, Scotland, UK, 2008.

Barrett, M.E. (2006). "Stormwater Quality Benefits of a Permeable Friction Course." World Environmental and Water Resources Congress 2006: Examining the Confluence of Environmental and Water Concerns, American Society of Civil Engineers, Reston, VA.

Barrett, M.E. and C.B. Shaw (2007). "Benefits of Porous Asphalt Overlay on Storm Water Quality." *Transportation Research Record: Journal of the Transportation Research Board, No. 2025,* Transportation Research Board of the National Academies, Washington, D.C.

Bean, E., W. Hunt, and D. Bidelspach (2007). "Field Survey of Permeable Pavement Surface Infiltration Rates." *Journal of Irrigation and Drainage Engineering*. May/June 2007

Briggs, J., A. Gong, E. Hadley, R. Roseen, and T. Ballestero (2008). "Permeable Pavement Sites In New England: 2007 Assessment Of Infiltration Capacity & Condition." University of New Hampshire Stormwater Center, Durham, NH.

Cackler, T., J. Alleman, J. Kevern, and J. Sikkema (2012). *Technology Demonstrations Project: Environmental Impact Benefits with "TX Active" Concrete Pavement in Missouri DOT Two-Lift Highway Construction Demonstration*. Final Report I, October 2012. Federal Highway Administration (DTFH-61-06-H-00011 (Work Plan 22)). http://www.intrans.iastate.edu/research/documents/research-reports/TX_Active_for_FHWA_w_cvr.pdf

Chai, L., M. Kayhanian, B. Givens, J. Harvey, and D. Jones (2012). “Hydraulic Performance of Fully Permeable Highway Shoulder for Storm Water Runoff Management.” *J. Environ. Eng.*, Vol. 138 Issue 7, pages 711–722. http://dx.doi.org/10.1061/(ASCE)EE.1943-7870.0000523.

Cooley, L.A., J.W. Brumfield, R.B. Mallick, W.S. Mogawer, M. Partl, L. Poulikakos, and G. Hicks (2009). *NCHRP Report 640: Construction and Maintenance Practices for Permeable Friction Courses*, Transportation Research Board of the National Academies, Washington, D.C.

Crouch, L., J. Pitt, and A. Hewitt (2007). “Aggregate Effects on Pervious Portland Cement Concrete Static Modulus of Elasticity,” *Journal of Materials in Civil Engineering*, Vol. 19, No. 7.

Delatte, N. (2007). “Structural Design of Permeable Concrete Pavements,” Presented at 86th Annual Meeting of the Transportation Research Board Annual Meeting, Washington, D.C., 16 pages.

Estakhri, C.K., A.E. Alvarez, A.E. Martin (2008). “Guidelines and Construction and Maintenance of Porous Friction Courses in Texas.” Report 0-5262-1, Texas Transportation Institute, Texas A&M University.

Ferguson, B. (2005). *Permeable Pavements*, CRC Press, Boca Raton, FL.

Federal Highway Administration (FHWA) (1990). *Open Graded Friction Courses.* FHWA Technical Advisory T5040.31, Washington D.C.

Gunderson J. (2010). “Boulder Hills LID Economic Case Study.” Forging the Link Between Research-Based Institutions, Watershed Assistance Groups, and Municipal Land Use Decisions, UNH Stormwater Center, Durham.

Hansen K. (2008). *Permeable Asphalt Pavements for Stormwater Management; Design, Construction and Maintenance Guide*. National Asphalt Pavement Association, pages 19–21.

Hein D. (2006). *Resilient Modulus Testing of Open Graded Drainage Layer Aggregates for Interlocking Concrete Block Pavement*, 8th International Conference on Concrete Block Paving, San Francisco, CA.

Hein, D., E. Strecker, A Poresky, R. Roseen and M. Venner (2013) *Permeable Shoulders with Stone Reservoirs,* Draft Report, Project Number NCHRP 25-25/Task 82. http://apps.trb.org/cmsfeed/TRBNetProjectDisplay.asp?ProjectID=3315

Hodgman, R. (2012). "Meeting notes regarding costs of 2009 installation of the Maine Mall Road, South Portland, ME." R.M. Roseen, ed., Maine DOT Transportation Research Program, Augusta, ME.

Hossain, M., L.A. Scofield, and R.W. Meier Jr., (1992). “Porous Pavement For Control Of Highway Runoff In Arizona: Performance To Date.” *Transportation Research Record 1354*, TRB, National Research Council, Washington, D.C., pages 45–54.

Houle, J.J., R.M. Roseen, T.P. Ballestero, T. Puls, and J. Sherrard, J. (2013 in press). “A Comparison of Maintenance Cost, Labor Demands, and System Performance for LID and Conventional Stormwater Management.” *Journal of Environmental Engineering,* American Society of Civil Engineers.

Jones, D., J. Harvey, H. Li, and B. Campbell (2010). *Summary of Laboratory Tests to Assess Mechanical Properties of Permeable Pavement Materials*. Caltrans Document No.: CTSW-TM-10-249.01, November 30, 2010. http://www.ucprc.ucdavis.edu/PDF/UCPRC-TM-2009-05.pdf.

Kandhal, P. and R. Mallick (1999). *Design of new-generation open-graded friction courses*. NCAT Rep. No. 99-3, National Center for Asphalt Technology, Auburn, AL.

Kayhanian, M. (2012). "Adapting Full-Depth Permeable Pavement for Highway Shoulders and Urban Roads for Stormwater Runoff Management." *Stormwater: The Journal for Surface Water Quality Professionals*, http://www.stormh2o.com/SW/Articles/Adapting_FullDepth_Permeable_Pavement_for_Highway_18309.aspx, August 13, 2012, accessed May 2013.

Kevern, J.T. and C. Farney (2012). "Reducing Curing Requirements for Permeable Concrete Using a Superabsorbent Polymer for Internal Curing." *Transportation Research Record: Journal of the Transportation Research Board, No. 2290,* Transportation Research Board of the National Academies, Washington D.C., page 115–121

Liu, K., A. Alvarez, A.E. Martin, T. Dossey, A. Smit, and C. Estakhri (2010). *Synthesis of Current Research on Permeable Friction Courses: Performance, Design, Construction, and Maintenance.* Report 0-5836-1, Texas Transportation Institute, Texas A&M University.

MacMullan, E. (2007). "Economics of LID." *EcoNorthwest*, Eugene, OR.

MaineDOT. (2010). *Technical Report 10-05: Performance of a Permeable Pavement System on the Maine Mall Road in South Portland.* Maine Department of Transportation, Transportation Research Division, Augusta, ME.

McGhee, K., T. Clark, and C. Hemp (2009). *Research Report: A Functionally Optimized Hot-Mix Asphalt Wearing Course: Part 1: Preliminary Results*. Virginia Transportation Research Council, Final Report VTRC 09-R20, Charlottesville, VA. http://www.virginiadot.org/vtrc/main/online_reports/pdf/09-r20.pdf

Muench, S.T. and J.P. Mahoney (2002). "Washington Asphalt Pavement Association Asphalt Pavement Guide: Pavement Types." Washington Asphalt Pavement Association, Inc., http://www.asphaltwa.com/wapa_web/ index.htm (December 16, 2009).

Muench, S., C. Weiland, J. Hatfield, and L. Wallace (2011). Open-Graded Wearing Courses in the Pacific Northwest, Final Report, SPR 680. Sponsored by Oregon DOT. http://www.wsdot.wa.gov/NR/rdonlyres/3EB8E603-819F-476F-A54B-70F71D517EFF/0/OregonDOTClassFReportJune2011.pdf

National Asphalt Pavement Association (NAPA) (2002). "Design, Construction, and Maintenance of Open-Graded Friction Courses."

NAPA. (2003). "Design, Construction, and Maintenance Guide for Permeable Asphalt Pavements." Information Series 131.

NAPA. (2008). "Design, Construction, and Maintenance Guide for Permeable Asphalt Pavements." Information Series 131.

Pagotto, C., M. Legret, and P. Le Cloirec (2000). "Comparison of the Hydraulic Behaviour and the Quality of Highway Runoff Water According to the Type of Pavement." *Water Resources*, Vol. 34, No. 18, pages 4446–4454.

Pitt, R., S. Clark, and R. Field (1999). "Groundwater contamination potential from stormwater infiltration practices." *Urban Water*, Vol. 1, Issue 3, pages 217–236.

Powell, L.M., E.S. Rohr, M.E. Canes, J.L. Cornet, E.J. Dzuray, and L.M. McDougle (2005). "Low-Impact Development Strategies and Tools for Local Governments." LMI Governmental Consulting.

Rasmussen, R.O., R.J. Bernhard, U. Sandberg, and E.P. Mun (2007). "The Little Book of Quieter Pavements," FHWA-IF-08-004, U.S. Department of Transportation, Federal Highway Administration, Washington, D.C.

Roque, R., C. Koh, Y. Chen, X. Sun, and G. Lopp (2009). "Introduction of Fracture Resistance to the Design and Evaluation of Open Graded Friction Courses in Florida." Florida Department of Transportation and University of Florida, Tallahassee, FL.

Roseen, R.M. (2013). "Project meeting notes with materials costs from Aggregate Industries for placement of 800 tons of PG76-22 for Commercial Street, Provincetown, MA," Stratham, NH.

Roseen, R.M. and T.P. Ballestero (2008). "Permeable Asphalt Pavements for Stormwater Management in Cold Climates." HMAT, Vol. 13, Issue 3, National Asphalt Pavement Association.

Roseen, R.M., T.P. Ballestero, J.J. Houle, J.F. Briggs, and J.P. Houle (2011) "Water Quality and Hydrologic Performance of a Permeable Asphalt Pavement as a Stormwater Treatment Strategy in a Cold Climate," *ASCE Journal of Environmental Engineering*.

Roseen, R.M., T.P. Ballestero, K.M. Houle, D. Heath, and J.J. Houle (2014). "Assessment of Winter Maintenance of Permeable Asphalt and Its Function for Chloride Source Control," *Journal of Transportation Engineering*, Vol. 140, Issue 2.

San Diego County (2008). Personal communication from Dane Clingan, San Diego County, California. February.

Schaefer, V.R., J.T. Kevern and K. Wang. (2011). *An Integrated Study of Permeable Concrete Mixture Design for Wearing Course Applications*, Final Report, sponsored through Federal Highway Administration DTFH61-06-H-00011 (Work Plan 10) and the Ready Mixed Concrete (RMC) Research & Education Foundation.

Schaefer, V., K. Wang, M. Suleiman, and J. Kevern (2006). Mix Design Development for Pervious Concrete In Cold Weather Climates, Sponsored by: Iowa Department of Transportation. http://www.ctre.iastate.edu/reports/mix_design_pervious.pdf

Smith D.R. (2011). *Permeable Interlocking Concrete Pavements: Selection, Design, Construction, Maintenance,* 4th ed., Interlocking Concrete Pavement Institute, Herndon, VA.

St. John, M. and R. Horner (1997): *Effect of Road Shoulder Treatments on Highway Runoff Quality and Quantity*. http://www.wsdot.wa.gov/research/reports/fullreports/429.1.pdf

Stanard, C., R. Candaele, R. Charbeneau, and M. Barrett (2007). *State of the Practice: Permeable Friction Courses*, Report No: FHWA/TX-08/0-5220-1, sponsored by Texas DOT and FHWA. http://www.utexas.edu/research/ctr/pdf_reports/0_5220_1.pdf

Texas Department of Transportation. (2004). Standard Specifications for Construction and Maintenance of Highways, Streets, and Bridges, Item 342, 312-329. Retrieved March 20, 2005 from TxDOT Web Site: http://www.dot.state.tx.us/business/specifications.htm

Thelen, E. and L.F. Howe (1978). *Permeable Pavement*, Franklin Institute Press, Philadelphia, PA.

University of New Hampshire Stormwater Center (UNHSC) (2009). *University of New Hampshire Stormwater Center 2009 Annual Report*, Durham, N.H. (http://www.unh.edu/erg/cstev/2009_stormwater_annual_report.pdf).

UNHSC (2009). “Fact Sheet: Permeable Asphalt Pavement for Stormwater Management.” http://www.unh.edu/civil-engineering/research/erg/cstev/pubs_specs_info/porous_ashpalt_fact_sheet.pdf

UNHSC, J. Houle, R. Roseen, and T. Ballestero (2012). “UNH Stormwater Center 2012 Biennial Report.” Cooperative Institute for Coastal and Estuarine Environmental Technology, Durham, NH.

United States Environmental Protection Agency (U.S. EPA) (2000). “Low Impact Development, a Literature Review”, EPA-841-B-00-005, Office of Water (4203). Environmental Protection Agency Washington, D.C.

Wang, T., J. Harvey, and D. Jones (2010). *A Framework for Life-Cycle Cost Analyses and Environmental Life-Cycle Assessments for Fully Permeable Pavements Technical Memorandum*, Caltrans Document No.: CTSW-TM-09-249.03, UCPRC Document No.: UCPRC-TM-2010-05, June 30, 2010 http://www.ucprc.ucdavis.edu/PDF/UCPRC-TM-2010-05.pdf

Wei, I.W. (1986). “Installation and evaluation of permeable pavement at Walden Pond State Reservation: Final Report to Commonwealth of Massachusetts.” Department of Civil Engineering, Northeastern University, Commonwealth of Massachusetts, Division of Water Pollution Control Research Project 77-12 & 80-22, Boston, MA.

Washington State Department of Transportation (WSDOT) (2011). WSDOT Pavement Policy. http://www.wsdot.wa.gov/nr/rdonlyres/d7971b81-5443-45b9-8b9b-bfc0d721f5a1/0/wsdotpavementpolicyfinal71211.pdf

WSDOT (unpublished). Unpublished research proposal for study of permeable pavement shoulders within state highways. http://www.ecy.wa.gov/programs/wq/psmonitoring/effectiveness/PermeableShouldersGuttersv3.docx

Younger K., R. Hicks, and M. Partl, (1994). *Evaluation of Permeable Pavement for Road Surfaces*, Report No. FHWA-OR-RD-95-03, sponsored by Oregon DOT and FHWA.