Developing a Practical Tool for Integrating Green Infrastructure into Cost-Effective Stormwater Management Plans

by

Jackson Ross Ellis

A thesis submitted to the Graduate Faculty of
Auburn University
in partial fulfillment of the
requirements for the Degree of
Master of Science

Auburn, Alabama
August 8, 2020

Keywords: Stormwater Management, Green Infrastructure, Optimization,
Low Impact Development, Urban Hydrology

Approved by

Dr. Frances C. O'Donnell, Chair, Assistant Professor of Civil Engineering
Dr. Jose G. Vasconcelos, Associate Professor of Civil Engineering
Dr. Benjamin F. Bowers, Assistant Professor of Civil Engineering

Abstract

Urban stormwater management today aims to manage both the quantity and quality of stormwater runoff. Best management practices (BMPs) such as detention basins have long been used to address runoff quantity objectives, and green infrastructure practices (GIPs) have emerged recently has an effective means for addressing water quality issues. There is a wide variety of GIPs available to stormwater designers, each with unique cost considerations and design guidelines. Optimal combinations of BMPs and GIPs can maximize stormwater benefits and cost effectiveness, but the available tools for BMP optimization are designed to be used by experienced stormwater practitioners. A simplified tool designed for inexperienced practitioners is needed to promote widespread use of green infrastructure.

A spreadsheet-based decision-support tool was developed to equip designers with a means to develop cost-effective stormwater management plans that integrate GIPs with other stormwater BMPs. The tool was designed to be flexible and easy-to-use while still providing actionable stormwater designs and cost estimates. The tool allows the user to select intrinsic and structural GIPs to achieve a target runoff reduction for water quality objectives. The hydrologic impact of using green infrastructure is modeled and measured with rainfall-runoff simulations. An optimization model for a detention basin BMP was developed to be used in conjunction with the spreadsheet tool.

The tool was applied to a representative case study site for which the actual design and cost estimates of on-site BMPs were known. The tool provided realistic results for the case study analysis and revealed that successive applications of the tool could easily provide the user with a site design that maximized cost-effectiveness. A major benefit of the tool was that application of

the tool required little time or effort on the part of the user and use of the tool required no specialized computing or stormwater modeling expertise. A sensitivity analysis of the tool illustrated the critical trade-off relationship between GIP costs and detention basin BMP costs and identified the potential for improving local stormwater policies.

Acknowledgments

I would like to thank my advisor, Dr. Frances C. O'Donnell, for her sound advice and guidance throughout the research process, and for providing the opportunity to study something I am passionate about. I also want to thank Dr. Jose G. Vasconcelos for being a mentor to me. His mentorship is the reason I pursued graduate school and has been critical for my research success and career preparedness. Additionally, I want to thank Dr. Benjamin F. Bowers for serving on my committee and providing helpful feedback. I am grateful to the faculty and staff of the Department of Civil Engineering for educating and supporting me in my undergraduate and graduate studies, and I am thankful to have had Auburn as my home for six years. I would like to thank my friend and officemate, Michael Z. Izzo, for his support and solidarity, both academic and extracurricular. Special thanks also goes to Austin W. Harmon for being a sounding board and valuable asset to my research process. I want to thank my partner, Robin M. Liever, for her unwavering support and encouragement. I could not have done this without her. Finally, I want to thank my parents, John T. Ellis and Elizabeth B. Ellis, for believing in me and always lending an ear.

Table of Contents

Abstract 2
Acknowledgments 4
Table of Contents 5
List of Tables 7
List of Figures 8
1. Introduction 10
1.1 Background 10
1.2 Research Objectives 13
1.3 Research Scope 13
1.3.1 Target User 14
1.4 Organization of Thesis 15
2. Literature Review 17
2.1 Stormwater Management 17
2.2 Green Infrastructure 21
2.3 Optimization in Water Resources 24
2.4 Available Tools 27
3. Methodology 30
3.1 Tool Overview 30
3.2 Green Infrastructure Design 31
3.2.1 Intrinsic GIPs 32
3.2.2 Structural GIPs 33
3.2.3 User Inputs and Workflow 35
3.3 Hydrologic Modeling 38
3.4 Detention Basin Optimization Model 50
3.5 Cost Estimation Methods 54
3.6 Case Study 55
3.7 Sensitivity Analysis 61
4. Results 63
4.1 User Interface 63
4.2 Case Study Results 70

4.3 Sensitivity Analysis Results 76
5. Conclusions 83
5.1 Research Conclusions 83
5.2 Recommendations for Future Research 84
References 86
Appendix A: Detention Basin Optimization Model GAMS Code 93
Appendix B: Case Study GAMS Output 96
Appendix C: Case Study Pre-development SWMM Input File 104
Appendix D: Case Study Post-development SWMM Input File 112

List of Tables

Table 3.1 Runoff Reduction Credits for Intrinsic GIPs (City of Birmingham, 2019) 33
Table 3.2 Default values for subcatchment properties (Rossman, 2009).................................... 45
Table 3.3 Default unit costs for construction of green infrastructure (US EPA, 2017)............... 55
Table 3.4 Detention basin construction unit costs (Lacy, 2016)... 55
Table 3.5 Pre-development conditions for a case study site in Lee County, AL 56
Table 3.6 Post-development case study site subcatchment properties.. 57
Table 3.7 Case study infiltration trench design volume and cost estimate 58
Table 3.8 Case study detention basin dimensions and cost estimate 60
Table 3.9 Unit costs developed for case study site 61
Table 3.10 Sensitivity analysis conditions for first 25 trials representing a small storm............ 62

Table 4.1 Comparison of original infiltration trench design and design calculated with tool..... 70
Table 4.2 Comparison of original detention basin design and design calculated with tool......... 71
Table 4.3 Case study detention basin design parameters for optimization model 72
Table 4.4 Optimized design for case study detention basin... 73
Table 4.5 Summary of cost estimates calculated with tool compared to original cost estimates 74
Table 4.6 Infiltration trench design calculated by tool for new subcatchment discretization...... 75
Table 4.7 Optimized detention basin design for new subcatchment discretization 75
Table 4.8 Summary of cost estimates for new subcatchment discretization............................... 76
Table 4.9 Sensitivity analysis results 77

List of Figures

Figure 1.1 The effect of urbanization on hydrology (National Research Council, 2008) 10

Figure 3.1 Process flow diagram for the developed tool 31
Figure 3.2 Green infrastructure selection and design process flow diagram 37
Figure 3.3 Synthetic temporal rainfall mass distribution functions for each rainfall distribution type (NRCS, 1986) 39
Figure 3.4 Rainfall distribution types for different geographic regions (NRCS, 1986) 40
Figure 3.5 Comparison of original NRCS synthetic rainfall distribution in 6-minute increments and calculated synthetic rainfall distribution in 5-minute increments 41
Figure 3.6 Rainfall hyetograph calculated from a Type III synthetic rainfall distribution 42
Figure 3.7 Rainfall hyetograph calculated for a 2-hour storm of 4.25 inches 43
Figure 3.8 Conceptual model in SWMM for subcatchment geometry 44
Figure 3.9 Idealized subcatchment for calculating characteristic width of overland flow 45
Figure 3.10 Example of triangular approximation of basin outflow hydrograph (Mays, 2010).. 47
Figure 3.11 Example of hydrographs generated from SWMM models and calculated required basin storage volume 49
Figure 3. 12 Profile view of conceptual basin design geometry 51
Figure 3. 13 Outlet structure design geometry 52
Figure 3.14 Post-development site design for a case study site in Lee County, AL 57
Figure 3.15 Post-development case study site pervious and impervious (hatched) subcatchments 57
Figure 3.16 Case study site infiltration trench design 58
Figure 3.17 Case study site detention basin design, (a) length dimensions, (b) width dimensions 59
Figure 3.18 Case study site detention basin outlet structure design 60

Figure 4.1 User interface: Cost data worksheet 63
Figure 4.2 User interface: Pre-development site conditions worksheet 64
Figure 4.3 User interface: Pre-development subcatchment data worksheet 65
Figure 4.4 User interface: Post-development site characteristics worksheet 66
Figure 4.5 User interface: Post-development subcatchment data worksheet 67
Figure 4.6 User interface: Intrinsic GI selection worksheet 68
Figure 4.7 User interface: Structure GI design worksheet 69
Figure 4.8 User interface: Results summary worksheet 69
Figure 4.9 Hydrographs calculated with the developed tool for case study site 71
Figure 4.10 Optimized detention basin design, (a) length dimensions, (b) width dimensions 73
Figure 4.11 Optimized detention basin outlet structure design 74
Figure 4.12 Green infrastructure cost as a function of water quality design rainfall variation ... 78
Figure 4.13 Detention basin cost as a function of water quality design rainfall variation 79
Figure 4.14 Total cost as a function of water quality design rainfall variation 79

Figure 4.15 Green infrastructure cost as a function of variation in imperviousness 80
Figure 4.16 Detention basin cost as a function of variation in imperviousness.......................... 80
Figure 4.17 Total cost as a function of variation in imperviousness 81
Figure 4.18 Local minima for detention basin costs as a function of green infrastructure costs. 82

1. Introduction

1.1 Background

When a natural area is developed into an urbanized watershed, the amount of impervious land cover in the watershed is drastically increased due to the construction of rooftops, parking lots, and roads. When it rains in an undeveloped watershed, the rainfall is intercepted by plant cover, evaporated into the atmosphere, transpired by plants, infiltrated into the soil, or becomes surface runoff and flows downhill. In a developed watershed, the increase in impervious area, as well as the decrease in plant cover, decreases the amount of rainfall that is infiltrated, intercepted, and evapotranspired and increases the amount of rainfall that becomes surface runoff (Figure 1.1).

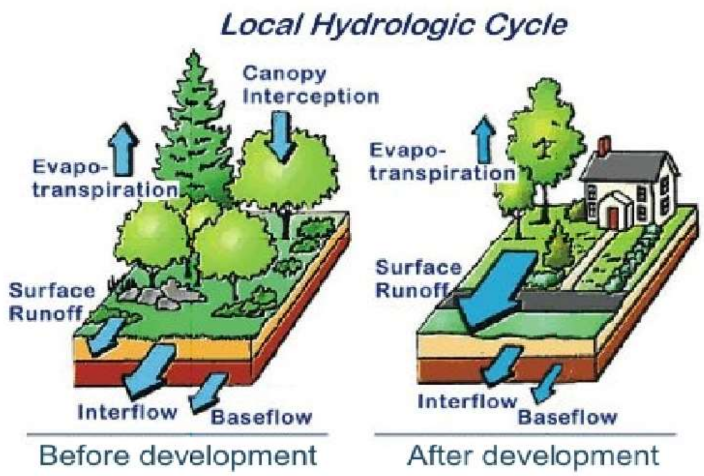


Figure 1.1 The effect of urbanization on hydrology (National Research Council, 2008)

Stormwater management is the practice of controlling excess runoff in the urban environment. Historically, stormwater has been managed by collecting runoff and conveying it quickly away from urban areas (US EPA, 2000). Detention basins are the most commonly used stormwater control measure for reducing peak post-development runoff flows to pre-developments levels (Mays, 2010). Modern stormwater management must address water quality concerns in conjunction with controlling the quantity of runoff discharged from urban catchments. Urban stormwater runoff is a significant source of nonpoint source pollution, and stormwater discharges to receiving bodies of water are regulated under the National Pollution Discharges Elimination System (NPDES) (National Research Council, 2008). Best management practices (BMPs) are stormwater control measures that are used to meet the requirements of stormwater regulations. Low impact development (LID) is based on the principle of capturing and treating stormwater at its source through the natural processes of infiltration, storage, and evapotranspiration (ADEM, 2016). Green infrastructure (GI) uses the principles of low impact development to treat stormwater in distributed, small scale facilities rather than discharging runoff to a drainage system. Green infrastructure and low impact development are effective for improving the water quality of runoff by capturing runoff produced by small, frequent storms and a portion of the runoff produced by larger storms (Collins et al., 2009). The runoff from small storms is responsible for most of the average annual pollutant mass loading to receiving bodies of water, and runoff reduction has become an important part of meeting stormwater management objectives (Pitt, 1987; US EPA, 2009b). Research has also shown that combining infiltration-based green infrastructure practices (GIPs) with conventional storage-based detention basin BMPs can help to preserve the pre-development hydrology of a site undergoing urbanization, which has become a common objective for urban stormwater management (Atlanta Regional Commission, 2016; Damodaram et al., 2010; US EPA, 2009b).

Low impact development techniques have been found to reduce lifecycle costs for stormwater infrastructure and can reduce the size and number of required BMPs in a watershed (Collins et al., 2009; US EPA, 2000). A case study on a Green Infrastructure Plan to be implemented in Lancaster, PA found that a $77 million investment in green infrastructure projects would reduce capital costs for other stormwater infrastructure by $120 million. Additionally, it was determined that the investment in green infrastructure would save the city $4.8 million per year in energy, air quality, and climate-related costs (US EPA, 2014). Green infrastructure can also benefit communities by increasing surrounding property values. A case study of green infrastructure projects in neighborhoods of Madison, WI used regression models to illustrate the connection between GI development projects and increases in property values (Madison, 2013). Integrating GIPs with local stormwater management infrastructure can bring about economic, environmental, and social benefits, but the wide variety of GIPs available to designers makes it difficult to determine the best practices, or combination of practices, to use. Through personal communication with local municipal stormwater practitioners (Scott Rogers, Dan Ballard, Leslie Gahagan, Ashley Campbell), the research team identified uncertainty associated with cost-effectiveness of GIPs to be a barrier in the wide-spread implementation of green infrastructure.

As green infrastructure becomes more integral to urban stormwater management, the need becomes greater for methods that aid designers in selecting GIPs that yield least-cost stormwater designs. Optimization techniques are frequently applied to identify and analyze cost-effective solutions to the complex problems of water resources management. Rainfall-runoff modeling tools, such as the Storm Water Management Model (SWMM), can be combined with optimization techniques to develop minimum-cost designs for using GIPs and/or detention basin BMPs to treat stormwater in an urban watershed (Damodaram & Zechman, 2013; Giacomoni & Joseph, 2017). Optimization techniques can also be applied to individual GIPs and BMPs to analyze cost-effectiveness of individual devices (Baptista & Paz, 2018; Stafford et al., 2015). Tools have been developed that use optimization techniques to aid in the cost-effective design of GIPs and BMPs for watershed-scale stormwater management (US EPA, 2017). However, these tools often require a high level of expertise in watershed management and modeling (Shoemaker et al., 2009), or they are focused only on low impact development and do not provide the analysis necessary for optimization of storage-based BMPs (Center for Neighborhood Technology, 2009). There is a need for a practical tool for integrating green infrastructure into cost-effective stormwater management plans.

1.2 Research Objectives

This research aims to develop a practical decision-support tool for assisting stormwater designers in integrating green infrastructure practices into cost-effective stormwater management plans. The objectives for this study are:

- to utilize and combine common hydrologic and hydraulic methods such as the SCS Curve Number method, the Small Storm Hydrology Model, the Runoff Reduction Method, and the Storm Water Management Model (SWMM);
- to develop an optimization model for a simple detention basin to allow analyses of combinations of LID and BMPs to meet stormwater management goals;
- to simplify planning-level design procedures for low impact development and green infrastructure practices and aid designers in selecting between a variety of diverse practices; and
- to develop a user-friendly tool that is easily obtained, utilized, and understood by stormwater practitioners at any level of expertise.

1.3 Research Scope

This research is concerned with permanent post-construction stormwater management practices. Stormwater management practices for erosion and sediment control during construction activities are not included in the scope of this study, however the principles of stormwater management during and after construction are similar. This research is also primarily concerned with the use of on-site stormwater control measures. Regional and sub-regional detention basins and other best management practices (BMPs) are not considered within the scope of this study. The methods and conclusions of this study are considered applicable for urban catchments between one and ten acres in size.

The product of this research is a decision-support tool for selecting appropriate green infrastructure practices to be integrated into cost-effective stormwater management plans for urban developments. This tool serves as a proof-of-concept device for combining a variety of theories and methods in stormwater design, appropriate for planning-level analyses and estimates. The described tool was developed for this research using Microsoft Excel. A spreadsheet-based approach was selected to ensure flexibility, practicality, and accessibility for all possible users. The tool includes algorithms for process automation written in Visual Basic for Applications (VBA). The VBA language was chosen so that future researchers can easily access and adapt the code that controls the tool. The tool uses the Storm Water Management Model (SWMM) as the computational engine for basic hydrologic modeling processes. Many assumptions were made about catchment characteristics, precipitation, and runoff routing to ensure flexibility of the developed tool; thus, the tool is not appropriate for calibrated hydrologic simulation.

1.3.1 Target User

The tool resulting from this research was developed with consideration for several categories of end user. The target user for the tool is stormwater practitioners with little to no expertise or experience in watershed management and modeling. The target user for the tool could also be stormwater designers with little to no experience with design and construction of green infrastructure practices. The developed tool could be used by municipalities as part of a site review process for new developments, and by developers in performing engineering estimates of stormwater management plans that include green infrastructure. Finally, the tool could be used by university extension programs as an instructional device for promoting the use of green infrastructure.

1.4 Organization of Thesis

This thesis contains five chapters, organized as follows:

Chapter 1: Introduction provides background on the effect of urbanization on hydrologic processes, objectives for urban stormwater management, the use of low impact development and green infrastructure, and the need for practical planning and design tools for stormwater best management practices. This chapter also contains a statement of the objectives for the research described in this thesis, as well as a discussion of the scope of this research.

Chapter 2: Literature Review summarizes research and developments in urban stormwater issues, the use of low impact development and green infrastructure practices, optimization techniques for stormwater management, and the tools that have been developed for stormwater practitioners to improve stormwater management practices.

Chapter 3: Methodology outlines the processes used in the creation of a decision-support tool for green infrastructure and stormwater best management practices. This chapter also describes the procedures used to conduct a case study and a sensitivity analysis using the developed tool.

Chapter 4: Results describes and discusses the structure and function of the user interface for the developed decision-support tool. This chapter also compiles, presents, and discusses the relevant results of the case study and sensitivity analysis.

Chapter 5: Conclusions contains a summary of the conclusions drawn from this study and a discussion of how the research objectives were satisfied. Also in Chapter 5 are recommendations for future research. Following Chapter 5 are references to works cited and appendices which provide supplementary data and methodological details.

2. Literature Review

2.1 Stormwater Management

Urbanization affects the hydrology of developing areas by increasing the amount of impervious land cover in a watershed. The increase in impervious cover causes an increase in peak discharge and volume of stormwater runoff, as well as a decrease in the storage, infiltration, and evapotranspiration of rainfall and runoff that occurs in undeveloped areas (Mays, 2010). This results in post-development stormwater runoff hydrographs with greater peak flows, shorter times to peak flow, and shorter flow durations than runoff hydrographs for pre-development conditions (Wanielista & Yousef, 1993). The traditional objectives of stormwater management are to prevent flooding caused by design storms defined by specific return periods, to reduce mass loading of pollutants to receiving bodies of water, to reduce post-development peak runoff discharge to pre-development conditions or below, and to maintain a percentage of rainfall excess on-site for groundwater recharge (Wanielista & Yousef, 1993). The stormwater control facilities typically used to meet these management objectives include underground storage, retention ponds (also called wet detention ponds), and extended detention basins (also called dry detention basins) installed both on-site and downstream.

The quantity of stormwater runoff from urban areas has long been an important consideration for stormwater management, but in recent decades the water quality of urban runoff has become the subject of stormwater management objectives and regulation. Stormwater accumulates pollutants as it moves through the urban environment including oil and grease, nutrients, heavy metals, and bacteria (Cook & DeBell, 2001). In 1987, the Clean Water Act (CWA) was amended to regulate stormwater under the National Pollutant Discharge Elimination System (NDPES). In 1990, the EPA issued the Phase I Stormwater Rules for operators of municipal separate storm sewer systems (MS4s) serving populations of greater than 100,000, and in 1999 the EPA issued the Phase II Stormwater Rules for MS4s serving populations of between 50,000 and 100,000 (National Research Council, 2008). These water quality regulations for urban stormwater runoff led to the widespread adoption of stormwater best management practices (BMPs), also called stormwater control measures (SCMs), as a part of municipal stormwater management plans (National Research Council, 2008; Roy-Poirier et al., 2010; US EPA, 1999). The most commonly used BMPs are multipurpose detention basins which help to meet the quantity and quality objectives of stormwater management, but there are many different types of BMPs available to stormwater designers which use storage, infiltration, and reuse practices.

The design of stormwater BMPs like detention basins is based on hydrologic analysis of developing watersheds. In 1986, the Natural Resources Conservation Service (NRCS) published Technical Release 55 (TR-55). TR-55 presents procedures for estimating runoff volumes in addition to peak discharges for small urban watersheds. Rainfall is converted to runoff in the TR-55 method by using a Curve Number (CN) which is based on soil, plant cover, imperviousness, interception, and surface storage in a watershed (NRCS, 1986). The Curve Number infiltration method is widely used to estimate runoff volumes from design rainfall because it does not require extensive input of soil characteristics. Curve Numbers are based on basic descriptions of the land cover conditions and the Hydrologic Soil Group (A, B, C, or D). Other infiltration models such as Horton's equation require input and calibration of several parameters (Equation 2.1) (Akan, 1992):

$$f_p = f_c + (f_o - f_c)e^{-kt} \qquad \text{Equation 2.1}$$

where f_p is the infiltration capacity of the soil in ft/sec, f_c is the equilibrium infiltration capacity in ft/sec, f_o is the initial infiltration capacity in ft/sec, k is a constant representing the rate of decreased infiltration capacity in sec^{-1}, and t is the time since the start of infiltration in seconds. The simplicity of the Curve Number method allows for flexible application and easily accessible inputs and results.

The runoff resulting from small frequent rain events is often not controlled or treated by conventional stormwater BMPs like detention basins, which are designed to control runoff from larger, infrequent flood events. Runoff volume is the most important hydraulic parameter for water quality, and runoff from small storms is responsible for the majority of annual pollutant loading to receiving bodies of water (Pitt, 1987, 1999). The hydrologic model described in TR-55 is suitable and commonly used for designing stormwater management infrastructure that controls runoff from large, infrequent "design" storms with return periods of 2-years, 10-years, 25-years, or 100-years. Research has shown that this model does not compare well with observed runoff volumes for small, frequent storms (Pitt, 1999). Rainfall and runoff were observed for numerous catchments across the United States with varying land uses and rainfall depths. It was found that for small rainfall depths, the Curve Number predicted by the NRCS TR-55 model was not representative of the actual Curve Number calculated from rainfall and runoff observations (Pitt, 1999). Pitt developed the Small Storm Hydrology Method as a more accurate model for calculating runoff volumes for small rainfall depths. It was found that accurate predictions of runoff volume could be calculated only with rainfall depth information and that other conditions such as antecedent moisture, rainfall durations, and rainfall intensities did not substantially improve estimates of runoff volumes from small storms (Pitt, 1987). The Small Storm Hydrology Method is defined by Equation 2.2 (ALDOT, 2014):

$$Q = P * Rv \qquad \text{Equation 2.2}$$

where Q is runoff in inches, P is small storm rainfall depth in inches, and Rv is a dimensionless volumetric runoff coefficient based on land cover conditions. Values for volumetric runoff coefficients have been determined for different rainfall depths and land cover conditions and are available in the literature (Pitt, 1987, 2013).

Small Storm Hydrology was not adopted as an integral part of stormwater management until the Runoff Reduction Method was developed and popularized. The Runoff Reduction Method was developed in 2008 by the Center for Watershed Protection and the Chesapeake Stormwater Network to assess the ability of stormwater control measures to reduce runoff volumes and treat stormwater quality (Collins et al., 2009). The Runoff Reduction Method allows comparison and combination of various stormwater management practices such as environmental site design (ESD), low impact development (LID), green infrastructure (GI), and conventional BMPs. Using LID and GI in combination with conventional BMPs can reduce the size and number of BMPs required to meet peak discharge and water quality objectives, as well as restore pre-development hydrologic conditions in terms of runoff volume, duration, velocity, frequency, infiltration, and stream protection (Collins et al., 2009). In 2007, the United States Congress enacted the Energy Independence and Security Act (EISA). Section 438 of the EISA requires federal developments to maintain or restore on-site pre-development hydrology to the maximum extent technically feasible (METF) (US EPA, 2009b). The EPA Technical Guidance Document for meeting the requirements of Section 438 suggests site designers use low impact development and green infrastructure practices to retain on-site the runoff resulting from small storms, less than or equal to the 95^{th} percentile rainfall depth for the site location (US EPA, 2009b).

Green infrastructure practices are well-suited for on-site capture and storage of stormwater runoff resulting from small storms. The Small Storm Hydrology Method makes hydrologic design for green infrastructure simpler than the design procedure required for flood-control BMPs such as detention basins. The Runoff Reduction Method suggests that combining green infrastructure and conventional stormwater best management practices can help designers meet stormwater quantity and quality objectives, as well as the requirements of Section 438 of the EISA to restore pre-development hydrologic conditions. There is a need for a methodology that integrates design of green infrastructure and detention basin BMPs for holistic site-specific stormwater management plans.

2.2 Green Infrastructure

The terms low impact development, best management practices, and green infrastructure are often used interchangeably, and the practices they refer to are often used together to meet stormwater management goals. The overlapping definitions of these terms has led to some uncertainty about how to use different LID and GI practices. There is a need for clarity and simplification in design guidance for low impact development and green infrastructure.

The principles of LID were developed in the 1990s in Prince George's County, MD and are based on controlling stormwater on-site rather than the conventional practice of quickly draining runoff to large facilities near watershed outlets (US EPA, 2000). The goal of low impact development is to mimic in developed watersheds the hydrologic conditions of natural, undeveloped areas (ADEM, 2016). LID practices are often small-scale stormwater controls distributed throughout a watershed that utilize the natural hydrologic processes of storage, infiltration, and evapotranspiration to reduce stormwater runoff. The use of low impact development has been shown to have economic, environmental, and social benefits. The EPA has conducted multiple case studies and found that LID can significantly reduce costs for stormwater management (US EPA, 2007, 2013), as well as bring aesthetic value to projects and communities (US EPA, 2000).

The term “low impact development” can refer to site design practices such as conservation of natural features, minimizing impervious surface, and disconnecting impervious areas from drainage system, or it can refer to constructed devices such as bioretention facilities, rain gardens, grass swales, vegetated rooftops, rain barrels, cisterns, vegetated filter strips, and permeable pavements (US EPA, 2000). Many LID practices are referred to in the literature as best management practices or BMPs, while some sources make a distinction between LID practices that control stormwater with natural processes and storage-based BMPs like detention and retention ponds (US EPA, 2000).

More recently, the term “green infrastructure” or GI has been used to refer to control practices that are designed and built to treat stormwater according to the principles of LID, such as infiltration trenches, bioretention cells, or permeable pavements (City of Birmingham, 2019). Some literature makes a further distinction between structural BMPs which directly control stormwater flows, and intrinsic BMPs which refers to management practices that protect stormwater quality such as street sweeping and better site design (National Research Council, 2008; Taylor et al., 2014). Green infrastructure practices (GIP) can also be referred to as structural or intrinsic. Examples of structural GIPs are bioretention, infiltration trenches, and permeable pavements, whereas examples intrinsic GIPs are green roofs, downspout disconnection, and filter strips (City of Birmingham, 2019).

There are different methods for modeling green infrastructure practices depending on the objective. Green infrastructure modeling efforts can aim to analyze the hydrologic impact of green infrastructure in a watershed or to simulate the physical processes that occur within GIPs. Perez-Pedini et al. developed a distributed watershed model of a small urban catchment to investigate the effect of location of infiltration-based BMPs on peak flows. The watershed was discretized into 120-meter squares and infiltration-based BMPs were modeled as a 5-unit reduction in Curve Number for the squares where they were applied (Perez-Pedini et al., 2005). This CN-based approach for modeling the impact of GIPs was adopted by Damodaram et al. (2010). Curve Numbers were calculated for permeable pavements, green roofs, and rainwater harvesting systems based on runoff reduction, and a developed watershed was modeled with distributed GIPs to investigate the effect of combining GIPs with a detention pond BMP under different design storm conditions. It was found that the pond BMP reduced peak flows better than the GIPs, and GIPs preserved the timing of the pre-development hydrograph, and combined use of GIPs and BMPs together resulted in the closest match to the timing and magnitude of the simulated pre-development hydrograph (Damodaram et al., 2010). In contrast to CN-based modeling approaches, the EPA Storm Water Management Model (SWMM) has the option to model LID controls. The model represents pollutant reduction, infiltration, and outflow processes in LID controls as continuous simulations of physically-based parameters of the LID controls (US EPA, 2019). The LID modeling capabilities in SWMM have been used in studies of optimal locations of LID in a catchment (Giacomoni & Joseph, 2017). Efforts to model the hydrologic impact of LID and GIPs are motivated by a need to develop a method for selecting the optimal GIP type, design, and location to meet stormwater management objectives.

Cost optimization of GIPs and BMPs requires analysis of the costs and benefits associated with different kinds of practices. Cost-effectiveness is defined as the ability of a specific practice to accomplish stormwater objectives for the lowest cost. There is little consensus in how researchers and practitioners define cost-effectiveness of BMPs. For example, Sample et al. (2003) includes opportunity cost of land occupied by BMPs in an assessment of cost-effectiveness (Sample et al., 2003). The National Cooperative Highway Research Program developed a whole life cost model for stormwater BMPs based on pollutant removal performance and extensive cost data analysis (Taylor et al., 2014). Other methods simplify analyses by estimating operation and maintenance costs as a percentage of construction cost (Urbonas et al., 2017; Weiss et al., 2007). There is a need to simplify and generalize methods for assessing BMP performance measures and lifecycle cost estimation methods so that BMPs can be compared with other stormwater control measures using a common value such as treatment volume.

2.3 Optimization in Water Resources

Optimization has long been a topic of research in water resources, particularly in the field of stormwater management. Researchers have used integrated simulation and optimization techniques to determine the optimal location and size of detention basin BMPs within a watershed. Yeh and Labadie developed hydrologic models for a watershed using HEC-1, an early computer-based hydrologic model (USACE, 1998), and applied successive reaching dynamic programming to determine minimum-cost design of a detention basin system that maintains desired peak flow reduction (Yeh & Labadie, 1997). A multi-objective genetic algorithm was also applied to optimize designs for other objectives such as providing water supply and minimizing sediment load reduction. Multi-objective optimization techniques are used to generate a nondominated set of solutions that form a trade-off curve known as a Pareto frontier (Yeh & Labadie, 1997). Multi-objective optimizations are especially usefully for water resources applications because there are often competing objectives in watershed management problems. For example, a watershed practitioner may use a multi-objective optimization technique to maximize flood control effects and minimize cost.

Genetic and evolutionary algorithms are increasingly popular in water resources optimization research, as they can be used to find solutions to highly non-linear problems (Park et al., 2012; Zhen et al., 2004). Genetic algorithms are a type of search algorithm applied to multi-objective optimization problems. Genetic algorithms are based on the principle of natural selection in evolutionary biology (Perez-Pedini et al., 2005). A genetic algorithm is initiated by the generation of a set of alternatives for a design scenario and evaluating the objective function for each alternative. The best alternative is used to create a new set of "offspring" alternatives by altering parameters of the "parent" alternative. This process is repeated until a global optimal solution is determined. Genetic algorithms are an efficient means for generating a non-dominated solution set, but the solution set must be further evaluated to determine practical solutions to the problem being optimized.

Other optimization techniques do not generate a Pareto solution frontier and thus produce one optimal solution. Scatter search techniques may be used to identify a single optima solution for watershed management problems. Zhen et al. integrated an agricultural nonpoint source pollution model (AnnAGNPS) with a BMPs simulation module and a scatter search heuristic optimization technique to determine the least-cost location and sizing of detention basins in a watershed to maintain maximum annual average pollutant load requirements (Zhen et al., 2004). Oxley and Mays developed a technique that used a simulated annealing procedure to optimize size, location, and outlet structure design of detention basins based on hydrologic data extracted from repeated HEC-HMS watershed simulations (Oxley & Mays, 2014). This combination of optimization techniques with rainfall-runoff models such as HEC-HMS or SWMM has recently become a topic of interest in water resources research.

Simulation-optimization techniques have been used in studying optimization of LID usage in a watershed. Perez-Pedini used a genetic algorithm to determine optimal locations of infiltration-based BMPs in a distributed watershed model and generated a Pareto solution set illustrating the trade-off between number of BMPs and peak flow reduction (Perez-Pedini et al., 2005). Other studies have applied genetic algorithms to watersheds modeled in SWMM and determined optimal solutions to minimize cost and peak flow alteration (Damodaram & Zechman, 2013; Giacomoni & Joseph, 2017). Optimization studies also have been conducted for the design of individual on-site best management practices. Like simulation-optimization techniques for watershed-scale analyses, genetic algorithms are popular for solving optimization models of individual BMPs. Park et al. developed an optimization model for the design of a detention basin and outlet structure and determined optimal designs using a genetic algorithm. The methods were applied to two existing detention ponds in South Korea, and the model generated feasible solutions with smaller basins and outlet structures (Park et al., 2012). Cost-effectiveness studies for detention basins by Baptista and Paz used an iterative design process to generate 32 alternate basin designs for a rooftop catchment in Brazil, varying basin and outlet structure geometry. The results indicated that a 3.2% reduction in basin efficiency (meaning peak basin outflow was simulated to be 3.2% greater than peak pre-development runoff) correlated to a 36.4% reduction in basin cost (Baptista & Paz, 2018). Non-linear programming (NLP) optimization models have been developed for infiltration-based BMPs. Stafford et al. developed an NLP model for minimum cost designs of infiltration basins. Rather than employing a continuous simulation of hydrologic processes, runoff and infiltration were modeled using the Rational Method and Green-Ampt infiltration method (Stafford et al., 2015). This model was adapted for cost optimization of bioretention basins with dry wells, using a mixed-integer non-linear programming (MINLP) model and calculations to model evapotranspiration (Lacy, 2016).

Optimization techniques have been used in many applications for stormwater management and modeling research. Most applications of optimization in water resources research use an integrated simulation-optimization approach that requires development of a calibrated hydrologic and/or hydraulic model and continuous simulation. There is a need for LID and BMP optimization techniques that do not require extensive hydrologic and hydraulic simulation, but still improve design practices by decreasing costs, time, and labor.

2.4 Available Tools

There are several tools available to designers that perform hydrologic calculations, sizing of BMPs or GIPs, cost estimation for stormwater infrastructure, and optimization of BMP design and usage. The available tools vary in which of these processes they include, as well as their functionality, flexibility, and ease-of-use. This section includes discussion of some of the design tools available to stormwater practitioners.

The National Stormwater Calculator (SWC) was developed by the EPA to allow stormwater designers to estimate runoff, consider climate change projections, and assess effectiveness and cost of LID controls. The SWC accesses national databases for historic precipitation data and soils maps for a user-specified location. The SWC uses SWMM 5 as its computational engine for rainfall-runoff calculations. Hydrologic modeling is performed using 24-hour design storms for the user-specified location with return periods of 5, 10, 15, 30, 50, and 100 years. Land cover is defined as percentages of total area, and all SWC results are expressed in terms of unit area. The user can select any combination of disconnection, rain harvesting, rain gardens, green roofs, street planters, infiltration basins, and porous pavement LID controls. LID controls are sized to retain a design rainfall depth for the area they are draining. A planning-level estimation of capital and maintenance cost for LID controls is calculated and adjusted for location (Rossman & Bernagros, 2019). The SWC presents a simplified approach to the rainfall-runoff modeling practices of SWMM. The user interface is easy to use and well documented. The many assumptions made about watershed and LID parameters can potentially affect the accuracy of the results but are required for the tool to be flexible. The SWC provides runoff results, but there is no design component for detention basin BMPs that may be required for on-site stormwater treatment, nor is there a cost estimation or optimization included for detention basin design.

The National Green Values Calculator (GVC) was developed to aid decision-makers in applying green infrastructure for stormwater management and is available as a webpage. The National GVC assesses green infrastructure performance only in terms of volume capture and provides no peak runoff calculations, so detention basin sizing is not possible using the National GVC. The National GVC accesses precipitation data from the Hourly Precipitation Dataset and determines the size of a design storm between the 85^{th} and 99^{th} percentile for the user's site. Green infrastructure BMPs are assessed only for volume capture capacity and infiltration is not modeled. Modeling infiltration would require extensive knowledge of the user's soil conditions, so infiltration is neglected to simplify the tool. Lifecycle costs for green infrastructure BMPs are calculated based on unit costs compiled from many sources. The National GVC calculates lifecycle costs for a conventional stormwater design and compares the cost estimate to that of a green infrastructure design (Center for Neighborhood Technology, 2009). The National GVC is designed to be accessible to decision-makers that are not strictly stormwater designers, and thus is relatively easy to use. There is no rainfall-runoff analysis for flooding design storms and no peak flow calculation, thus the National GVC is not suitable for developing stormwater management plans that combine LID with detention basin BMPs.

The EPA System for Urban Stormwater Treatment and Analysis Integration (SUSTAIN) is a comprehensive decision-support and modeling framework for cost-effective implementation of BMPs throughout a watershed to meet stormwater management goals. SUSTAIN is a publicly available tool that uses process-based simulations to determine the optimal location, type, and cost of BMPs. SUSTAIN is highly specialized and is intended for users with practical understanding of watershed and BMP modeling processes. SUSTAIN has a user interface in ArcGIS and the framework consists of a BMP siting tool, a watershed runoff and routing module, a BMP simulation module, a BMP cost database, a post-processor, and an optimization module (Shoemaker et al., 2009). The optimization module uses meta-heuristic and evolutionary algorithms to solve the non-linear, multi-objective, complex optimization model developed in SUSTAIN (Lee et al., 2012).

The development of the EPA Opti-Tool for Stormwater and Nutrient Management made the SUSTAIN framework more accessible to practitioners. Opti-Tool is a spreadsheet-based BMP optimization tool with planning-level and implementation-level analysis options. The planning-level analysis simply uses the Solver add-in for Microsoft Excel to calculate optimal design storage for BMPs. The implementation-level analysis utilizes the SUSTAIN module to provide optimized BMP performance and cost-effectiveness results (US EPA, 2017). Opti-Tool requires rainfall-runoff modeling data, but does not perform simulations internally, requiring the user to generate hydrologic simulation results externally. Opti-Tool was designed for, and currently only supports, EPA Region 1 (New England) (US EPA, 2017).

There is a need for a decision-support tool that assists stormwater practitioners in selecting and designing green infrastructure practices to meet their stormwater management objectives. Such a tool should be accessible to all designers and require little expertise in advanced hydrologic and hydraulic modeling. A design tool should be flexible and provide results at a planning-level accuracy for green infrastructure sizing, green infrastructure cost estimates, and optimized detention basin sizing and cost estimation.

3. Methodology

3.1 Tool Overview

A spreadsheet-based tool was developed using Microsoft Excel that assists users in developing site-specific stormwater management plans for urban catchments using green infrastructure practices (GIPs). The developed tool allows users to describe their site design and select appropriate structural and intrinsic GIPs to reduce stormwater runoff and mitigate the effects of increased imperviousness. The hydrologic impact of development on the user's site is determined through hydrologic modeling. The Excel tool accesses the EPA Storm Water Management Model (SWMM) to model rainfall and runoff under pre-development and post-development site conditions for a user-specified design storm. The results of the hydrologic model are used to adapt a detention basin cost-optimization model for the user's flood protection requirements. The detention basin optimization model was developed to represent a multi-purpose basin that provides stormwater quality and flood protection. This section describes in detail the methods employed in the tool to perform a full planning-level analysis that includes green infrastructure selection and design, stormwater modeling, cost estimation, and detention basin design and optimization. The methods used in the tool were developed to meet the research objective of creating a tool that is accessible to any stormwater practitioner and allows flexibility in site design and stormwater management objectives. Figure 3.1 illustrates how the user interacts with the internal processes of the developed tool and how these processes interact with each other.

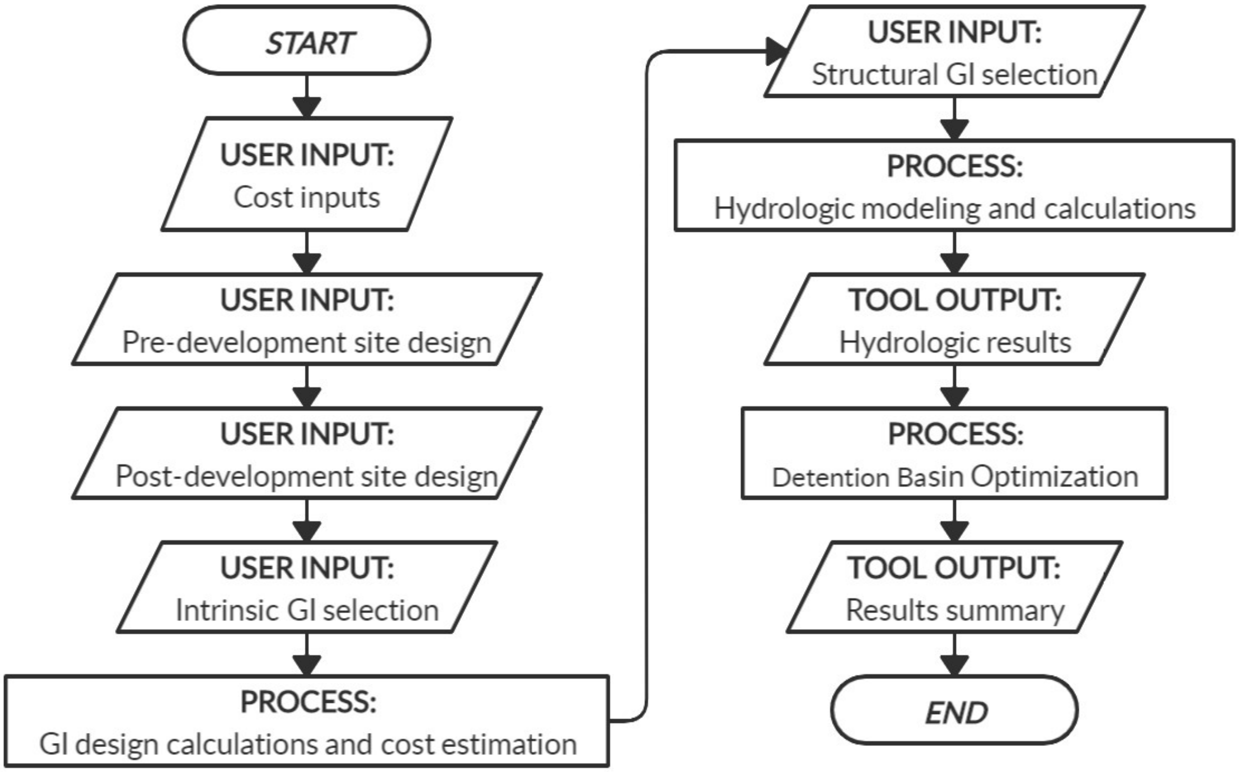


Figure 3.1 Process flow diagram for the developed tool

3.2 Green Infrastructure Design

The green infrastructure design process consists of selecting structural and intrinsic (also called non-structural) practices for reducing and capturing runoff generated on impervious surfaces. The tool makes a distinction between intrinsic GIPs (such as grass channels or sheet flow) and structural GIPs (such as infiltration trenches or permeable pavements). Intrinsic GIPs are used to reduce the amount of runoff that must be treated by structural GIPs. Structural GIPs are media-based engineered devices that capture and store a volume of runoff in the void space of a porous media. The Microsoft Excel spreadsheet user interface allows the user to easily select green infrastructure practices based on their desired site design.

The objective of the green infrastructure design process is to find a middle ground between accuracy of the results and usability of the developed tool. To reduce data requirements and simplify the green infrastructure design process, abstractions from stormwater runoff such as infiltration, evapotranspiration, and outflow from drains were neglected in the sizing calculations for structural GIPs. Similar BMP modeling assumptions have been made in BMP optimization and modeling research efforts (Damodaram & Zechman, 2013). Other available tools such as the EPA Opti-Tool use process-based simulations of stormwater best management practices (BMPs) and thus have higher data requirements for simulating BMP performance (US EPA, 2017). SWMM has physically-based modeling capabilities for GIPs (called LID Controls in SWMM) but using these models requires input of GIP geometry, which is unknown to the user of the tool developed in this research.

3.2.1 Intrinsic GIPs

It is assumed that treatment volume is the main driver of cost for structural GIPs. The runoff reduction due to intrinsic GIPs is quantified by adjusting the volumetric runoff reduction coefficient for the impervious areas where GIPs are applied. The volumetric runoff reduction coefficient (*Rv*) can be defined as the proportion of average annual runoff to average annual rainfall and depends on rainfall intensity and land cover conditions (Pitt, 1987). The volumetric runoff reduction coefficient for impervious areas is assumed to be 0.95 (City of Birmingham, 2019). Volumetric runoff reduction coefficients are adjusted with Equation 3.1 (City of Birmingham, 2019):

$$Rv' = Rv * \left(1 - \frac{RRC}{100}\right) \qquad \text{Equation 3.1}$$

where *Rv'* is the adjusted volumetric runoff reduction coefficient for an impervious area where an intrinsic GIP is applied, *Rv* is the unadjusted volumetric runoff reduction coefficient for an impervious area (equal to 0.95), and *RRC* is the runoff reduction credit associated with an applied intrinsic GIP. Table 3.1 contains runoff reduction credits for the intrinsic GIPs available to the user in the developed tool. Runoff reduction credits depend on GIP design level; for more information on low impact development and green infrastructure design, see the Low Impact Development Handbook for the State of Alabama (ADEM, 2016).

Table 3.1 Runoff Reduction Credits for Intrinsic GIPs (City of Birmingham, 2019)

Intrinsic GIP	Runoff Reduction Credit	
	Level 1	Level 2
Downspout Disconnection	17	45
Grass Channel w/o compost amended soil	1	20
Grass Channel with compost amended soil	12	30
Green Roof	78	89
Sheet Flow to pervious area	45	72
Sheet Flow to filter strip	45	50

3.2.2 Structural GIPs

The structural GIPs available to the user in the tool are bioretention basins, infiltration trenches, and permeable pavement systems. Each structural GIP has unique design criteria; for more information see the Low Impact Development Handbook for the State of Alabama (ADEM, 2016). The treatment volume is calculated with Equation 3.2 to determine the required size of structural GIP (City of Birmingham, 2019):

$$Tv = Rv' * A_{imp} * P_{wq} * \frac{43560}{12} \qquad \text{Equation 3.2}$$

where Tv is the structural GIP treatment volume in ft^3, Rv' is the dimensionless adjusted volumetric runoff reduction coefficient for an impervious area where a structural GIP is applied, A_{imp} is the impervious area in acres from which a structural GIP will receive runoff, P_{wq} is the water quality design rainfall in inches, and the last term is a conversion from acre-inches to ft^3. The water quality rainfall depth (P_{wq}) is typically defined as a small but frequent rain event, often quantified as the

85th, 90th, or 95th percentile rainfall for a location (ALDOT, 2014; Atlanta Regional Commission, 2016; City of Birmingham, 2019).

The SCS Curve Number method is used to calculate the runoff volume in watershed-inches produced by a design rainfall accounting for reductions due to structural GIPs. The Curve Number (CN) is an empirical parameter describing the rainfall-runoff relationship that depends on land cover conditions, antecedent soil moisture, and hydrologic soil group (HSG) classification (NRCS, 1986). The use of green infrastructure reduces the amount of rainfall that becomes runoff which decreases the CN value for impervious areas where GIPs are applied (Perez-Pedini et al., 2005). An adjusted CN that accounts for the treatment volume stored by structural GIPs is calculated with Equation 3.3 to Equation 3.6 (City of Birmingham, 2019):

$$Q = \frac{(P_2 - 0.2S)^2}{(P_2 + 0.8S)} \qquad \text{Equation 3.3}$$

$$S = \frac{1000}{CN} - 10 \qquad \text{Equation 3.4}$$

$$Q' = Q - \frac{12 * Tv}{43560 * A_{imp}} \qquad \text{Equation 3.5}$$

$$CN' = \frac{1000}{10 + 5P_2 + 10(Q') - 10(Q'^2 + 1.25Q'P_2)^{\frac{1}{2}}} \qquad \text{Equation 3.6}$$

where Q is unadjusted runoff volume in watershed-inches, P_2 is flood protection design rainfall in inches, S is maximum soil retention in inches, CN is unadjusted Curve Number for an impervious area where a structural GIP is applied (assumed to be 98 for impervious areas), Q' is adjusted runoff volume in inches for impervious areas were a structural GIP is applied, , and CN' is the adjusted Curve Number for the impervious area where a structural GIP is applied. The flood protection design rainfall (P_2) represents the user's design standard for flood protection stormwater infrastructure such as detention basins, typically a 2-year, 10-year, or 25-year return period storm of a 24-hour duration (City of Auburn, 2019; City of Birmingham, 2019). These calculations reflect the reduction in runoff from impervious areas associated with capturing a portion of the flood protection design rainfall via structural GIPs. The adjusted Curve Number for impervious areas are used in the creation of a hydrologic model representing the user's site design.

3.2.3 User Inputs and Workflow

The tool requires minimal input from the user for the green infrastructure design processes. In addition to the depths for water quality design rainfall (P_{wq}) and flood protection design rainfall (P_2), the user must specify the number of discrete areas of impervious cover in their post-development site design, defined as impervious areas with differing average slopes or impervious cover type. The user then must enter the size in acres for each area of impervious cover (as well as average slope for hydrologic calculations) and select a description for each area as parking, building, or other. The user can elect to apply intrinsic GIPs to their site design for each impervious area. The tool uses Visual Basic for Applications (VBA) algorithms to lock or unlock applicable GIP options in the Excel spreadsheet user interface based on the selected impervious cover type. For example, if an impervious area is described as parking, the green roof intrinsic GIP option will not be available, and the permeable pavement structural GIP will be available. The tool calculates the adjusted volumetric runoff reduction coefficient (Rv') for each impervious area based on the selected intrinsic GIPs and design levels. The user can also elect to apply structural GIPs to their site design for each impervious area. If structural GIPs are applied, the developed tool calculates structural GIP treatment volume (Tv), adjusted Curve Number for each impervious area where GIPs are applied (CN'), and estimated construction cost for bioretention, infiltration trench, and permeable pavement GIPs. The user can then select which structural GIP will be applied to each area of impervious cover. This process allows a maximum of one intrinsic GIP and one structural

GIP be applied for each area of impervious cover. The process flow diagram for green infrastructure selection and design is shown in Figure 3.2.

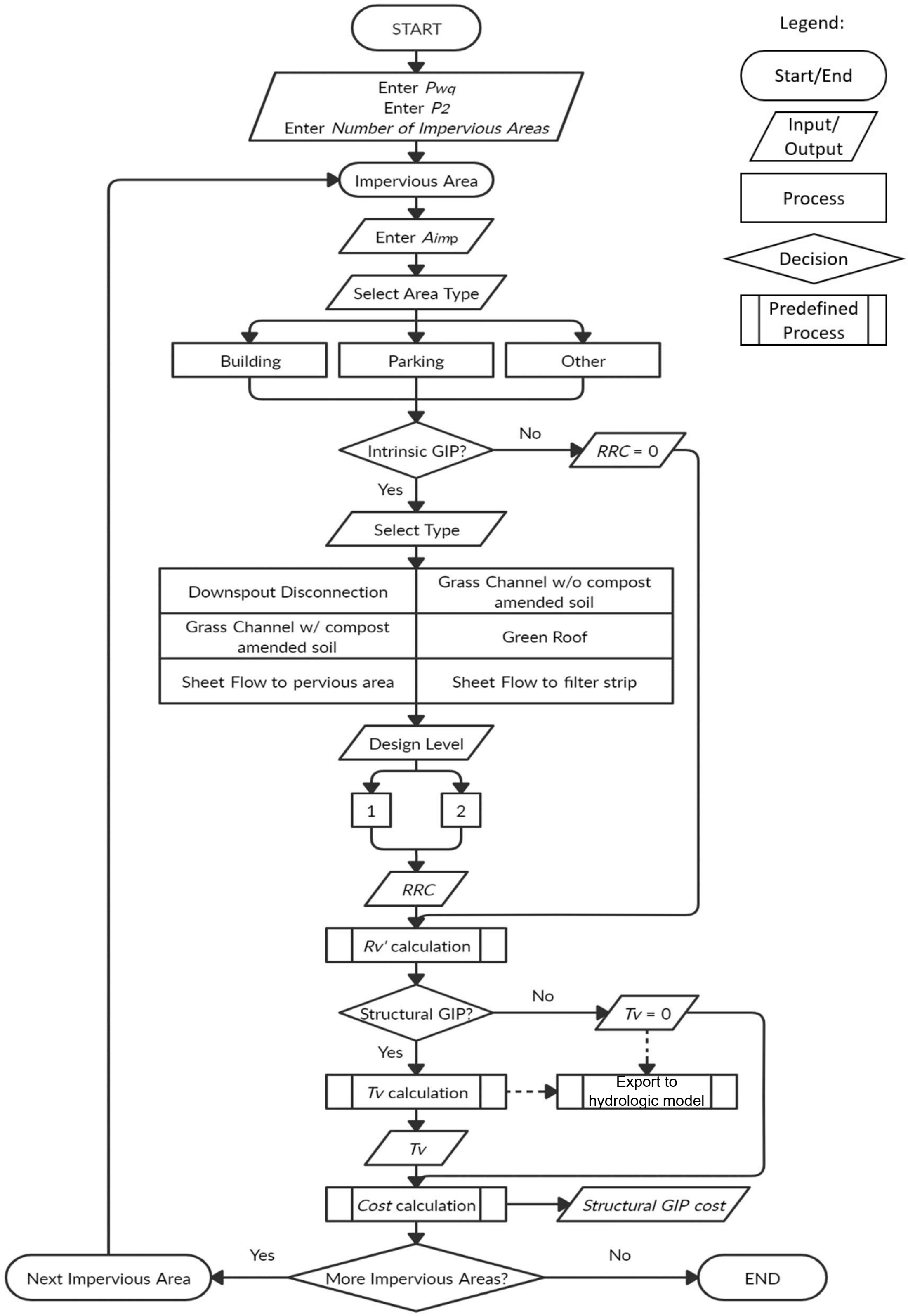


Figure 3.2 Green infrastructure selection and design process flow diagram

3.3 Hydrologic Modeling

The objectives of the hydrologic model employed in the tool are to simulate rainfall-runoff for the user's site design and quantify the hydrologic impact of using GIPs. Additionally, the hydrologic methods generate pre-development and post-development runoff hydrographs for a flood protection design storm and calculate the required storage volume for a detention basin design. The EPA Storm Water Management Model (SWMM) is for hydrologic modeling. The version of SWMM that is used (SWMM 5.1) is freely available for download from the EPA and has extensive documentation and technical support.

SWMM is a physics-based rainfall-runoff modeling software that is used to simulate stormwater quantity and quality in urban areas (Rossman, 2009). The input and output files for SWMM models have a standardized format that is conducive to parsing and editing by the Visual Basic for Applications (VBA) algorithms used in the developed tool. The input files for pre-development and post-development SWMM models are automatically edited and generated based on user-input site characteristics and hydrologic parameters. The tool uses a VBA script to execute pre-development and post-development SWMM models in the background via the Command Prompt application. This allows the rainfall-runoff models to be created and the simulations to be executed without any action or intervention on the part of the user. The output files for the pre-development and post-development SWMM simulations are automatically opened and the time-series runoff data are imported into the tool's Excel spreadsheet, where figures illustrating the pre-development and post-development runoff hydrographs are generated.

Design storms are represented in SWMM by a 5-minute increment time series distribution of rainfall intensities (called a rainfall hyetograph) over a storm duration. The user can select one of two design storm durations: a 2-hour storm or a 24-hour storm. A rainfall hyetograph is calculated based on the user-input flood protection design rainfall depth and the user-selected design storm duration.

Hyetographs are developed for 24-hour design storms using (City of Auburn, 2019; City of Birmingham, 2019)the NRCS synthetic rainfall distributions. These provide the cumulative fraction of a design rainfall over a 24-hour period (Figure 3.3). The NRCS developed four synthetic distributions to represent different geographic areas of the United States (Figure 3.4):

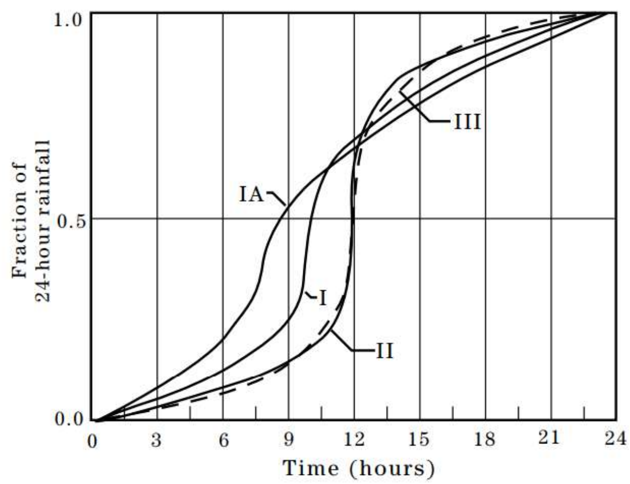


Figure 3.3 Synthetic temporal rainfall mass distribution functions for each rainfall distribution type (NRCS, 1986)

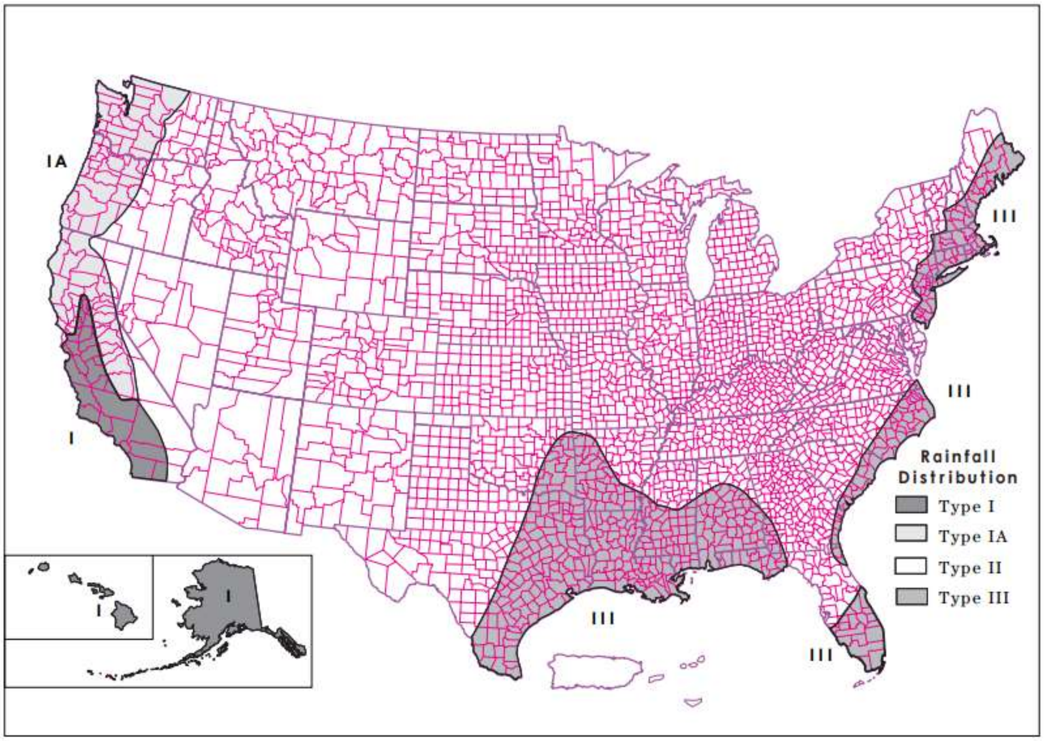


Figure 3.4 Rainfall distribution types for different geographic regions (NRCS, 1986)

The Type III synthetic rainfall distribution, which is representative of most of Alabama was used. The NRCS provides cumulative fractions of a 24-hour design rainfall depth in 6-minute (0.1 hour) increments (NRCS, 2015). To be compatible with SWMM, the data was converted to a 5-minute increment series ($X_{5(1)}, X_{5(2)},\ldots, X_{5(288)}$) by creating a series of points from the 6-minute series at 30-min intervals ($X_{30(1)}, X_{30(2)},\ldots, X_{30(48)}$) and interpolating between these points (Equation 3.7):

$$X_{5(i+1)} = X_{5(i)} + \frac{X_{30(j+1)} - X_{30(j)}}{6} \qquad \text{Equation 3.7}$$

where $X_{5(i)}$ is the ith value in the 5-minute series, and $X_{30(j)}$ is the jth value in the 30-minute series. The 30-minute series points are divided by six because that is the number of 5-minute increment data points in each 30-minute interval. Figure 3.5 illustrates that there is little difference between the synthetic rainfall distribution data in 6-minute increments and 5-minute increments.

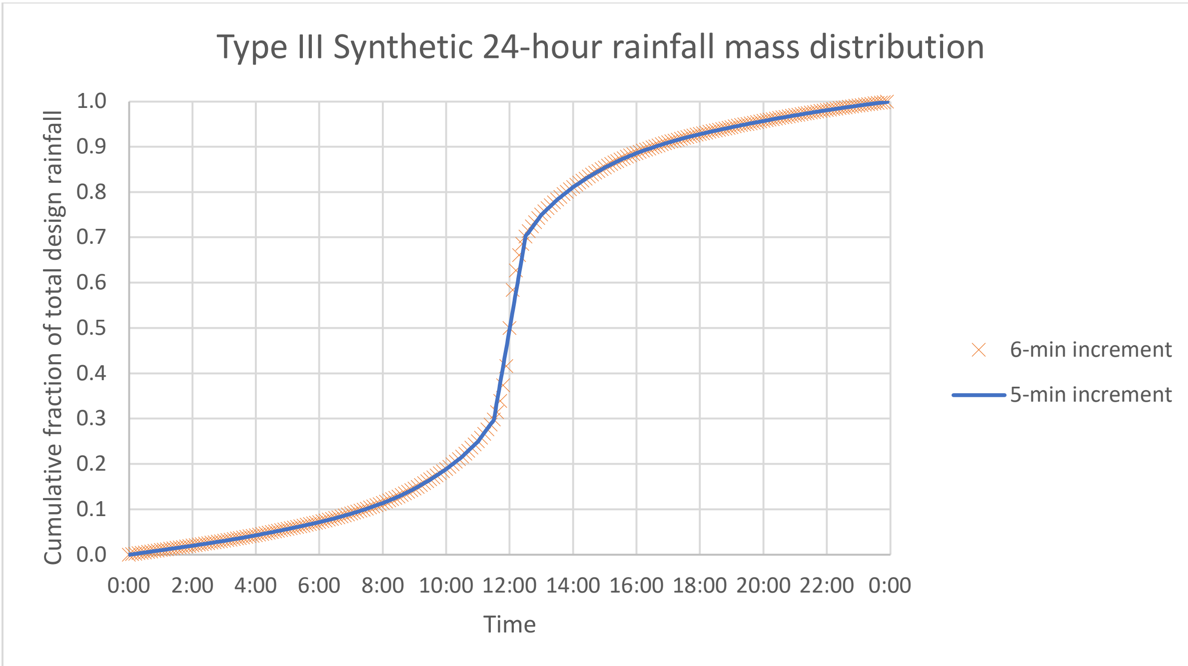


Figure 3.5 Comparison of original NRCS synthetic rainfall distribution in 6-minute increments and calculated synthetic rainfall distribution in 5-minute increments

The 5-minute increment data series was used to calculate the rainfall intensity for each 5-minute time step of a synthetic 24-hour design storm using Equation 3.8 through Equation 3.10:

$$C_{5(i)} = X_{5(i)} * P_2 \qquad \text{Equation 3.8}$$

$$V_{5(i)} = C_{5(i+1)} - C_{5(i)} \qquad \text{Equation 3.9}$$

$$I_{5(i)} = V_{5(i)} * \frac{60\,\frac{min}{hr}}{5\,min} \qquad \text{Equation 3.10}$$

where $C_{5(i)}$ is the cumulative rainfall depth in inches in the ith 5-minute time step, $X_{5(i)}$ is the cumulative fraction of the total design rainfall for the ith 5-minute time step, P_2 is the user-input flood protection design rainfall in inches, $V_{5(i)}$ is the incremental additional rainfall in inches for the ith 5-minute time step, and $I_{5(i)}$ is rainfall intensity in in/hr for the ith 5-minute time step. An example of the hyetograph resulting from these calculations is shown in Figure 3.6. The 24-hour time series hyetograph for the user's flood protection design storm is calculated and written to the input files for the pre-development and post-development SWMM models.

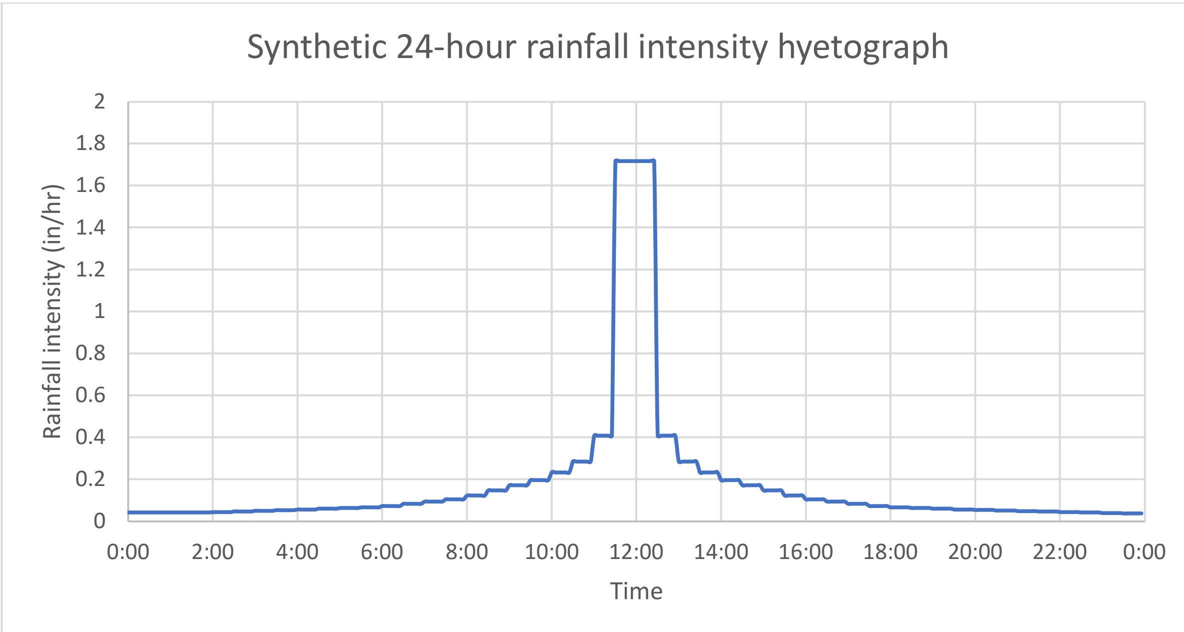


Figure 3.6 Rainfall hyetograph calculated from a Type III synthetic rainfall distribution

A 2-hour rainfall time series was also developed for users needing to determine the impact of design storms with shorter durations. The hyetograph for a representative 2-hour storm was obtained from the SWMM Applications Manual (US EPA, 2009a). The hyetograph obtained from the SWMM Applications Manual represented a 100-year 2-hour storm in Fort Collins, CO with a total rainfall depth of 3.67 inches (US EPA, 2009a). Rainfall intensities for each 5-minute time step are scaled to represent the user-input flood protection design rainfall depth using Equation 3.11 through Equation 3.13:

$$V_{SWMM(i)} = I_{SWMM(i)} * \frac{5\ min}{60\ min/hr} \qquad \text{Equation 3.11}$$

$$V_{user(i)} = V_{SWMM(i)} * \frac{P_2}{P_{SWMM}} \qquad \text{Equation 3.12}$$

$$I_{user(i)} = V_{user(i)} * \frac{60\ min/hr}{5\ min} \qquad \text{Equation 3.13}$$

where $V_{SWMM(i)}$ is the incremental rainfall depth in inches in the ith 5-minute time step of the example storm, $I_{SWMM(i)}$ is the rainfall intensity in in/hr in the ith 5-minute time step of the example storm, $V_{user(i)}$ is the incremental rainfall depth in inches in the ith 5-minute time step of the user's design storm, P_2 is the user-input flood protection design rainfall in inches, P_{SWMM} is 3.67 inches, and $I_{user(i)}$ is the rainfall intensity in in/hr in the ith 5-minute time step of the user's design storm. Figure 3.7 shows an example of the rainfall hyetograph resulting from these calculations.

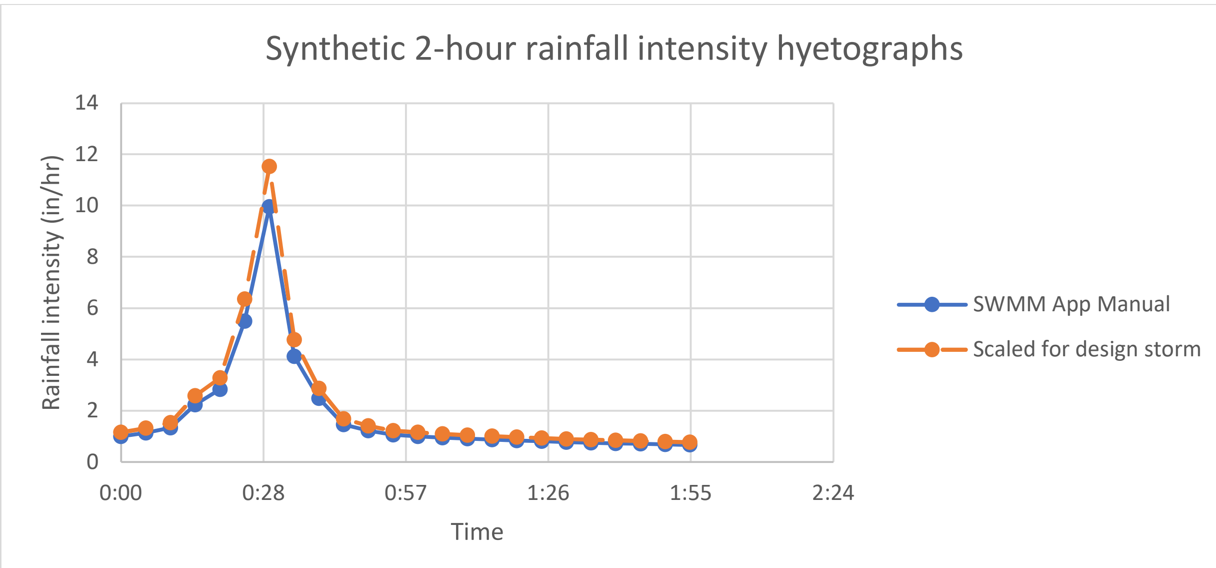


Figure 3.7 Rainfall hyetograph calculated for a 2-hour storm of 4.25 inches

The tool allows the user to define a maximum of 10 subcatchments for pre-development site characteristics. Subcatchments are discretized based on hydrologic soil group (HSG) and land cover conditions. Curve Numbers for pre-development conditions are input for each subcatchment by the user.

Several assumptions and simplifications were made to allow the tool to create planning-level SWMM models of the user's pre-development and post-development site designs. All infiltration is modeled using the Curve Number infiltration option. The area in acres and average slope percentage for each modeled subcatchment are input by the user and automatically written to SWMM input files by the tool. SWMM requires that a characteristic width for overland flow be specified for each subcatchment. The conceptual model for all subcatchments in SWMM is a rectangle with a representative subcatchment width and a length that represents the maximum length for overland flow (Figure 3.8). This width parameter is related to the maximum length of overland flow before shallow channelized flow begins, and is typically calculated by dividing the subcatchment area by the longest flow path in the subcatchment (US EPA, 2009a).

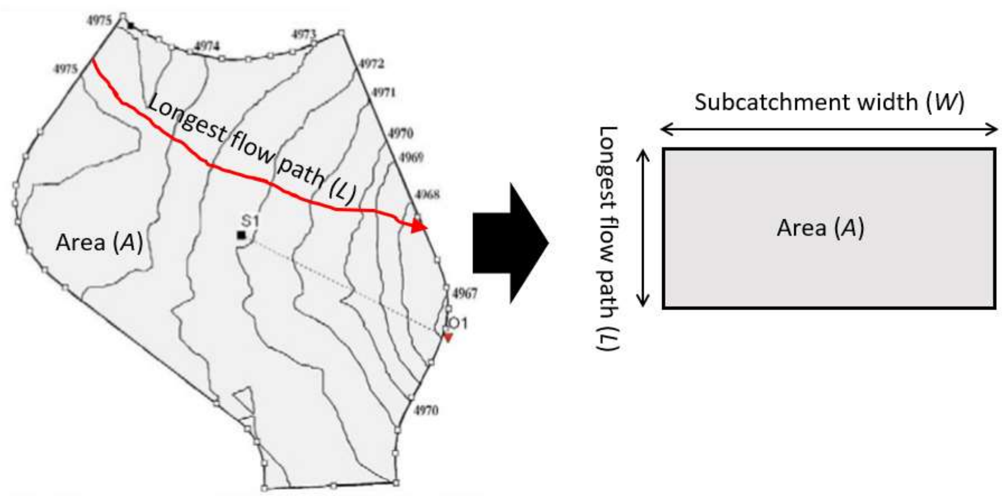


Figure 3.8 Conceptual model in SWMM for subcatchment geometry

In cases where the longest flow path in a subcatchment is greater than 500 feet, it is recommended that the width be calculated by dividing the subcatchment area by 500 feet (US EPA, 2009a). The developed tool uses a VBA function to calculate subcatchment widths. The subcatchment is idealized as a square and the longest flow path is defined as the length of the diagonal as shown in Figure 3.9.

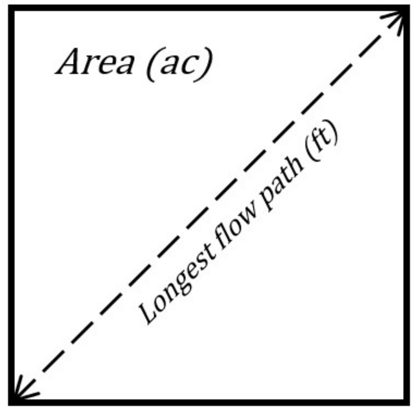


Figure 3.9 Idealized subcatchment for calculating characteristic width of overland flow

The width for each subcatchment is calculated based on user input subcatchment areas (Equation 3.14):

$$Width = \frac{A * 43560\frac{ft^2}{ac}}{\min\left(\left(A * 43560\frac{ft^2}{ac} * \sqrt{2}\right); 500\ ft\right)} \qquad \text{Equation 3.14}$$

where *Width* is the characteristic width of overland flow in feet, and *A* is user-input subcatchment area in acres. All subcatchments are assigned the default values for impervious area roughness coefficient, pervious area roughness coefficient, impervious area depression storage, pervious area depression storage, and percent of impervious area with no depression storage (Table 3.2).

Table 3.2 Default values for subcatchment properties (Rossman, 2009)

Subcatchment Property	Default Value
Impervious area roughness coefficient	0.015
Pervious area roughness coefficient	0.24
Impervious area depression storage (in)	0.06
Pervious area depression storage (in)	0.3
Percent of impervious area with zero depression storage	25

The SWMM model for post-development conditions is developed similarly. The tool allows the user to define a maximum of 10 impervious areas and 10 pervious areas for their post-development site design. Impervious subcatchments are discretized by type of impervious cover (building, parking, or other). Pervious subcatchments are discretized based on hydrologic soil group (HSG) and land cover conditions. Area, average slope, and Curve Numbers for pervious subcatchments are input for each subcatchment by the user. Area, average slope, impervious cover type, and whether to apply intrinsic GIPs are specified by the user for each impervious subcatchment and the unadjusted Curve Numbers are assumed to be 98.

To preserve the simplicity and flexibility of the tool, all modeled subcatchments are routed to one outfall and drainage infrastructure is not modeled. Runoff results are representative of overland flow from each subcatchment generated by the simulated rainfall event.

The SWMM graphical user interface can launch external applications using the add-in tools function. This function was used to create correctly formatted templates for SWMM input files in Excel worksheets within the tool. A VBA algorithm edits the worksheets containing SWMM input files based on user-input subcatchment information, calculated adjusted Curve Numbers, and calculated rainfall time series. The VBA program writes the edited SWMM input files to text files for pre-development and post-development models and saves the files in a specific location. The tool calls the SWMM executable using the VBA Shell function and specifies where the input files are located and where to save the output files from SWMM. The VBA script opens each output file and writes the content line-by-line to a specific worksheet in the Excel tool. The catchment outfall time-series flow data are parsed from both the pre-development and post-development SWMM output files and are used to generate runoff hydrographs for pre-development and post-development conditions.

The pre-development and post-development hydrographs are used to calculate the required storage volume for a detention basin that is designed to capture excess runoff from the modeled site and attenuate peak flows for the simulated design storm. The minimum required volume of runoff a detention basin must store can be defined as the time integral of the difference between the basin inflow hydrograph and the basin outflow hydrograph, as described by Equation 3.15 (Mays, 2010):

$$V_{req} = \int_0^t (Q_{in} - Q_{out})dt \quad \text{Equation 3.15}$$

where V_{req} is required basin volume, Q_{in} is the inflow hydrograph function, Q_{out} is the outflow hydrograph function, and t is the time at which the values of Q_{in} and Q_{out} are equal in the falling limb of the inflow hydrograph. The calculation of a basin outflow hydrograph typically requires an iterative routing procedure, but can be simplified by approximating the basin outflow hydrograph as triangular or linear as shown in Figure 3.10 (Mays, 2010).

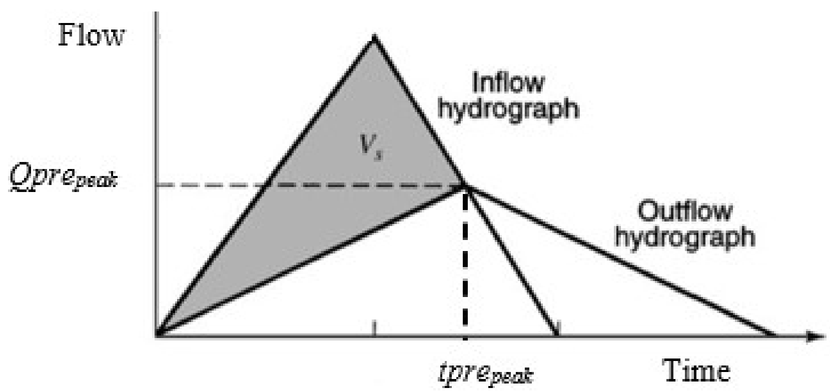


Figure 3.10 Example of triangular approximation of basin outflow hydrograph (Mays, 2010)

It is assumed that the post-development runoff hydrograph is equal to the basin inflow hydrograph. A linear basin outflow hydrograph is calculated and used to determine required basin volume (Equation 3.16 through Equation 3.17):

$$m = \frac{Q_{pre_{peak}}}{t_{pre_{peak}}} \quad \text{Equation 3.16}$$

$$Q_{out(i)} = m * t_i \quad \text{Equation 3.17}$$

where m is the slope of the linear basin outflow hydrograph approximation, $Qpre_{peak}$ is the peak pre-development runoff, $tpre_{peak}$ is the time where post-development flow equals peak pre-development flow in the falling limb of the post-development hydrograph, $Q_{out(i)}$ is basin outflow in the ith time step of the hydrograph calculation, and t_i is the time value in the ith time step of the hydrograph. Equation 3.17 is repeated for each time step until t_i is equal to $tpre_{peak}$.

A numerical integration of the difference in post-development runoff hydrograph and basin outflow hydrograph is performed using Equation 3.18:

$$V_{req} = \sum_i ((Q_{post(i)} - Q_{out(i)}) * (t_i - t_{i-1}) * 84600 \frac{sec}{day}) \qquad \text{Equation 3.18}$$

where V_{req} is required basin volume in ft^3, $Q_{post(i)}$ is post-development runoff in ft^3/sec in the ith time step, $Q_{out(i)}$ is basin outflow in ft^3/sec in the ith time step, t_i is time value in days in the ith time step, and the last term converts time units from days to seconds. Equation 3.18 is repeated for each time step until Q_{post} and Q_{out} are equal. Figure 3.11 Example of hydrographs generated from SWMM models and calculated required basin storage volume shows an example illustrating these calculations.

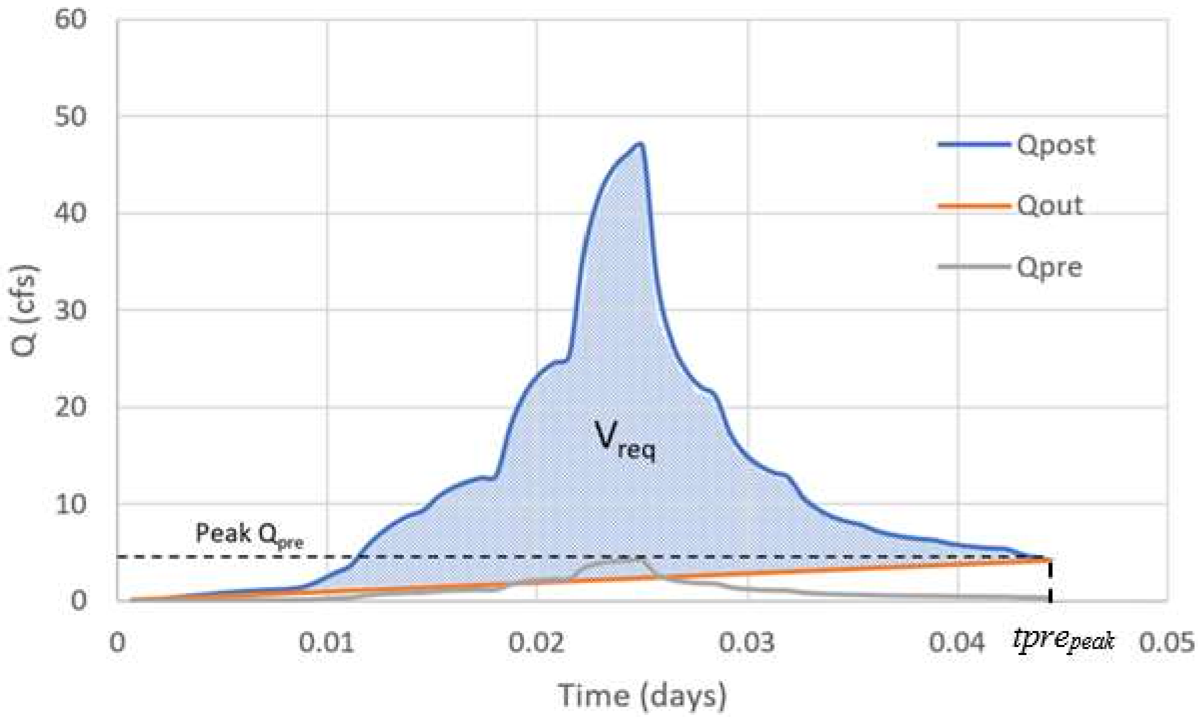


Figure 3.11 Example of hydrographs generated from SWMM models and calculated required basin storage volume

Detention basins act as stormwater quality infrastructure by capturing and storing the water quality capture volume for an extended period of time to allow pollutant removal to occur (US EPA, 2009). The WQv is typically released over a period of 24 hours (Atlanta Regional Commission, 2016). The water quality capture volume is calculated for the tool using Equation 3.19 and Equation 3.20:

$$WQv = P_{wq} * R * A_{imp} * \frac{43560\,\frac{ft^2}{ac}}{12\,\frac{in}{ft}} \qquad \text{Equation 3.19}$$

$$R = 0.05 + 0.009 * I \qquad \text{Equation 3.20}$$

where WQv is water quality capture volume in ft^3, P_{wq} is water quality design rainfall in inches, R is a dimensionless runoff coefficient, A_{imp} is total impervious area in acres, and I is percent imperviousness for a site. (US EPA, 2009a)(Atlanta Regional Commission, 2016)

The peak pre-development runoff flow, required detention basin storage volume, and water quality capture volume are used in an optimization model for the design of a detention basin to provide water quality and flood protection for the user's post-development site design.

3.4 Detention Basin Optimization Model

Detention basins and similar store-and-release stormwater best management practices (BMPs) are used in urban stormwater management to capture excess runoff from impervious areas and release it at attenuated peak flow rates. Modern detention basins typically have trapezoidal cross sections, and outlet structures that have a combination of orifices and weirs at different stages to treat stormwater for different design storms. They also provide stormwater quality protection by detaining a water quality capture volume (*WQv*), representing the runoff generated by small storms and the initial portion of runoff generated by larger storms. Extended storage of the water quality capture volume in detention basins reduces pollutant loads by preventing pollutant-laden runoff from entering receiving water bodies and allowing suspended solids to settle (US EPA, 2009a). The design of detention basin outlet structures is typically an iterative process using manual routing calculations or a rainfall-runoff simulation tool such as SWMM. The objective of detention basin design procedures is to minimize detention basin dimensions and maximize outlet structure dimensions while meeting design criteria concerning maximum basin outflow, maximum water depth, and minimum detention time for water quality.

A conceptual design of a multi-purpose detention basin was developed to simplify the optimization model. The basin was modeled as having an upper pond and a lower pond, of which the lower pond would be designed to treat the water quality capture volume (*WQv*), and the combined volume of both ponds would be sized to store the calculated required storage volume for a design storm (V_{req}). The basin was modeled as having vertical side slopes and rectangular cross sections. The outlet structure for the basin was modeled as having two orifices. A lower orifice restricts outflow from the lower pond and detains the water quality capture volume for a minimum detention time. An upper orifice was modeled so that combined peak outflow from the upper and lower orifices would not exceed peak pre-development flows. Both orifices are located in the side of the basin and circular in shape. The lower orifice has no vertical offset from the bottom of the lower pond and the upper orifice has no vertical offset from the maximum height of water stored in the lower pond. Figure 3. 12 and Figure 3. 13 illustrate the geometry of the conceptual basin design, where V_1 is volume of the lower pond, V_2 is volume of the upper pond, h_1 is height of the lower pond, h_2 is height of the upper pond, Ao_1 is orifice area for the lower orifice, Ao_2 is orifice area for the upper orifice, A_1 is plan area of the lower pond, and A_2 is total plan area of the basin (also plan area of the upper pond).

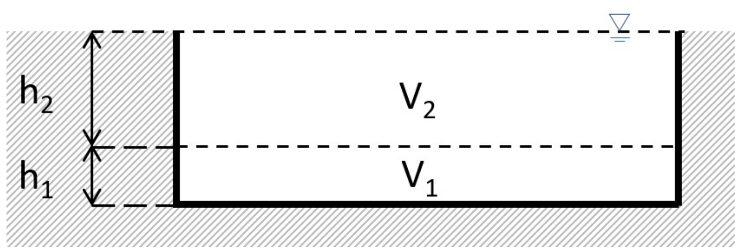


Figure 3. 12 Profile view of conceptual basin design geometry

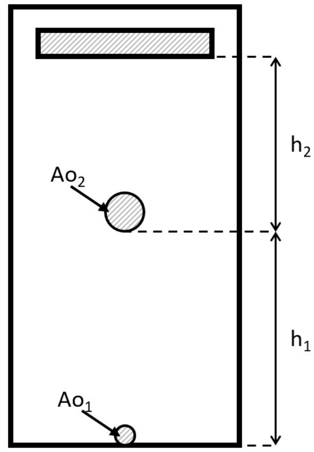


Figure 3. 13 Outlet structure design geometry

A nonlinear programming (NLP) optimization model was developed for this conceptual basin design. Optimization models consist of an objective function, constraints, and decision variables. The objective function for the optimization model was to minimize the total cost of construction for the detention basin (Equation 3.21):

$$Minimize\ Z = U_A * A_2 + U_V * (A_1 * h_1 + A_2 * h_2) \qquad \text{Equation 3.21}$$

where Z is total cost of construction in dollars, U_A is unit cost in dollars per ft^2 of land occupied by the basin, A_2 is basin surface area in ft^2, U_V is unit cost in dollars per ft^3 of excavated basin volume, A_1 is plan area of the lower pond in ft^2, h_1 is height of the lower pond in feet, and h_2 is height of the upper pond in feet. The optimization was bound by the following constraints:

1. The volume of the lower pool must be greater than or equal to the calculated water quality capture volume (Equation 3.22):

$$WQv \ \leq A_1 * h_1 \qquad \text{Equation 3.22}$$

where WQv is water quality capture volume in ft^3.

2. The volume of the basin must be greater than or equal to the calculated required storage volume (Equation 3.23):

$$V_{req} \leq A_1 * h_1 + A_2 * h_2 \qquad \text{Equation 3.23}$$

where V_{req} is required basin storage volume in ft^3.

3. The plan area of the lower pond must be less than or equal to the total plan area of the basin (Equation 3.24).

$$A_1 \leq A_2 \qquad \text{Equation 3.24}$$

4. The total plan area of the basin is less than a user-specified maximum (Equation 3.25):

$$A_2 \leq A_{lim} \qquad \text{Equation 3.25}$$

where A_{lim} is maximum basin area in ft^2.

5. The total height of the basin must be less than a user-specified maximum (Equation 3.26):

$$h_{lim} \geq h_1 + h_2 \qquad \text{Equation 3.26}$$

where h_{lim} is maximum basin height in feet.

6. The combined peak outflow from both orifices must be less than or equal to peak pre-development runoff (Equation 3.27):

$$Q_{pre} \geq C_{d1} * Ao_1 * \sqrt{2 * g * (h_1 + h_2)} + C_{d2} * Ao_2 * \sqrt{2 * g * h_2} \qquad \text{Equation 3.27}$$

where Q_{pre} is peak pre-development runoff in ft^3/sec, C_{d1} is the discharge coefficient for the lower orifice (assumed to be 0.65), Ao_1 is orifice area for the lower orifice in ft^2, g is gravitational acceleration (assumed to be 32.2 ft/sec^2), C_{d2} is the discharge coefficient for the upper orifice (assumed to be 0.65), and Ao_2 is orifice area for the upper orifice in ft^2.

7. The water quality capture volume is detained in the lower pond for the user-specified minimum detention time (Equation 3.28):

$$T_{det} \leq \frac{A_1}{Ao_1 * C_{d1}} \sqrt{\frac{2 * h_1}{g}} \qquad \text{Equation 3.28}$$

where T_{det} is minimum detention time in seconds.

The nonlinear programming model was developed and solved using the General Algebraic Modeling Software (GAMS, 2020). GAMS has been used in similar optimization models for infiltration basins (Lacy, 2016; Stafford et al., 2015). GAMS is a modeling system capable of modeling highly complex mathematical programming problems and has extensive documentation and support materials (GAMS, 2020). The above NLP model was solved using the multi-start heuristic search nonlinear programming solver. The free version of GAMS is readily available for download but has a maximum of 10 equations and 10 variables. The conceptual basin model could be adapted to represent a more realistic detention basin and an optimization model could be solved using a paid version of GAMS or a similar modeling software.

3.5 Cost Estimation Methods

The objective of the cost estimation methods in the tool is to provide the user with a value to evaluate and compare the cost-effectiveness of stormwater plans created using the tool. Construction cost of green infrastructure practices and stormwater BMPs is commonly estimated based on the size or design volume of the BMP (Sample et al., 2003; Taylor et al., 2014; Urbonas et al., 2017; Weiss et al., 2007). Construction cost of green infrastructure practices (GIPs) are calculated in the tool by multiplying the calculated treatment volume by a unit cost (Equation 3.29):

$$Con\ Cost = U_V * T_v \qquad \text{Equation 3.29}$$

where *Con Cost* is construction cost of a GIP in dollars, U_V is unit cost of a GIP in dollars per ft^3, and T_v is GIP treatment volume in ft^3 calculated via the methods described in Section 3.2.

The user has the option to enter their own unit costs for GIPs or use the default values. The default values were obtained from the EPA Opti-Tool User Guide (
Table **3.3**).

Table 3.3 Default unit costs for construction of green infrastructure (US EPA, 2017)

Green infrastructure type	Unit cost ($\$/ft^3$)
Bioretention	15.46
Infiltration trench	12.49
Permeable pavement	18.07

The construction cost for a detention basin is calculated as described in the cost-optimization function (Equation 3.21. The user has the option to enter their own unit costs for land cost and excavation cost or use the default values in Table 3.4, which were adapted from values used for infiltration basins (Lacy, 2016). Because costs can vary significantly due to location and site conditions, it is recommended that the user develop and enter their own unit costs to improve accuracy of all cost calculations.

Table 3.4 Detention basin construction unit costs (Lacy, 2016)

Unit cost	Default value
Land ($\$/ft^2$)	0.69
Excavation ($\$/ft^3$)	0.74

3.6 Case Study

A case study was performed using the developed tool. The objectives of the case study were to illustrate proper use of the tool, to compare tool results to actual stormwater designs, and to analyze and discuss the limitations of the tool. The developed tool was used to model the case study site based on the pre-development (

Table **3.5**) and post-development subcatchment properties (Table 3.6), the 2-year 24-hour rainfall depth for Lee County, AL, and a water quality design rainfall of 1.2 inches (City of Auburn, 2019). This analysis compares the actual designs and cost estimates for an infiltration trench and a detention basin calculated with the tool to those calculated for the actual site design.

The site selected for the case study was a newly developed property for a small building and two adjacent parking lots in Lee County, AL. The pre-development land cover condition for the 5.62-acre property was wooded with fair grass cover. The soil was determined to be Hydrologic Soil Group B using the NRCS Web Soil Survey and a Curve Number of 60 was assigned for pre-development conditions (NRCS, 1986, 2020). The NOAA Atlas 14 Precipitation Frequency Estimates were used to determine the 2-year 24-hour rainfall depth of 4.15 inches (NWS, 2020).

Table 3.5 Pre-development conditions for a case study site in Lee County, AL

Case study site pre-development conditions	
Area (ac)	5.62
Hydrologic Soil Group	B
Land cover condition	Woods, fair condition
Curve Number	60
Average slope (%)	5
2-year 24-hour rainfall (in)	4.15

The post-development site plan included a 0.8-acre building, a 1.1-acre parking lot, and an additional 1.4-acre parking lot. The remaining 2.32 acres were developed into grassed open area. An on-site detention basin was designed for the purpose of storing excess runoff and reducing peak runoff flows from a 2-year 24-hour design storm. The site design also included an infiltration trench to capture runoff from the 1.1-acre parking lot. The post-development site design is shown in Figure 3.14. This site design was discretized into five subcatchments, two pervious and three impervious (Figure 3.15). The properties for each subcatchment are shown in Table 3.6.

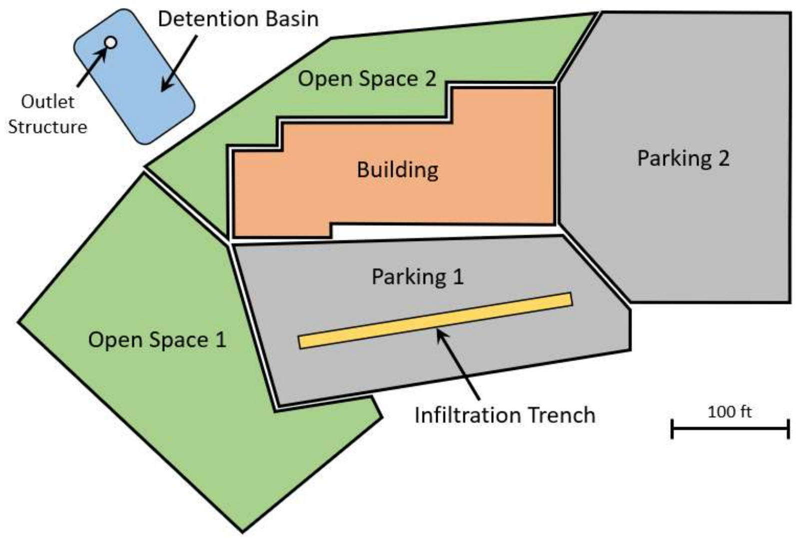


Figure 3.14 Post-development site design for a case study site in Lee County, AL

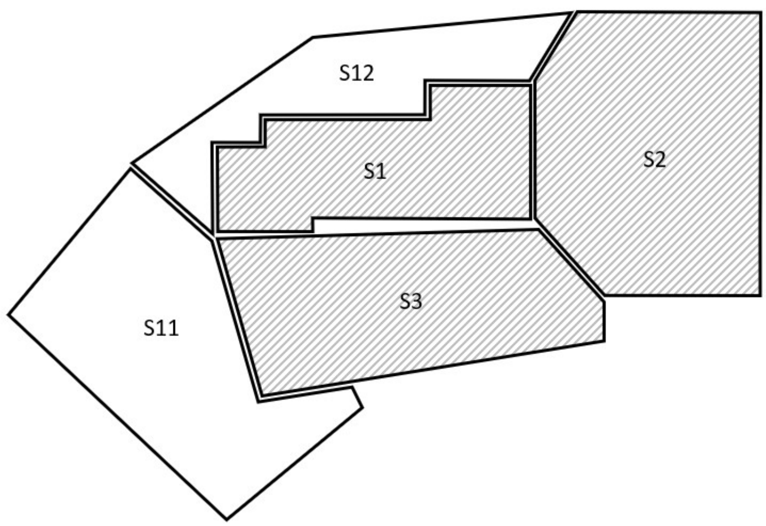


Figure 3.15 Post-development case study site pervious and impervious (hatched) subcatchments

Table 3.6 Post-development case study site subcatchment properties

Case study site post-development conditions					
Subcatchment name	S11	S12	S1	S2	S3
Land cover condition	Open space, poor condition	Open space, fair condition	Impervious, building	Impervious, parking	Impervious, parking
Area (ac)	1.16	1.16	0.8	1.4	1.1
Curve Number	79	69	98	98	98
Average slope (%)	2	3	0.5	3	1.5

The infiltration trench in subcatchment S3 was designed with the dimensions shown in Figure 3.16. The porous media filling the infiltration trench was assumed to have a porosity of 50%, thus the treatment volume for the infiltration trench was calculated to be 1185 ft^3. The infiltration trench was estimated to cost \$5,000 to construct (Table 3.7).

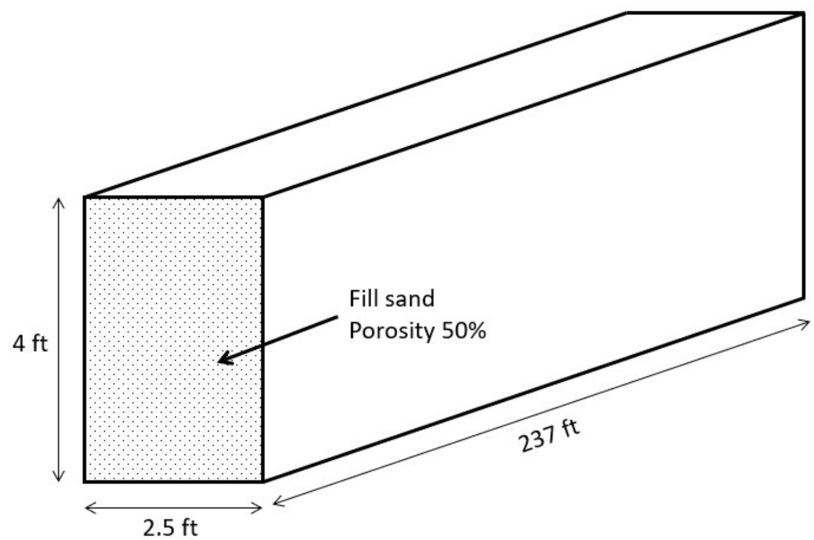


Figure 3.16 Case study site infiltration trench design

Table 3.7 Case study infiltration trench design volume and cost estimate

Infiltration trench design	
Treatment volume (ft^3)	1185
Construction cost	\$ 5,000.00

The on-site detention basin was shaped as a trapezoidal prism with side slopes of 3:1 and a depth of 6 ft (Figure 3.17). Basin geometry was calculated using Equation 3.32 through Equation 3.34:

$$L_{top} = L_{bot} + 2 * z * D \quad \text{Equation 3.30}$$

$$W_{top} = W_{bot} + 2 * z * D \quad \text{Equation 3.31}$$

$$A_{top} = L_{top} * W_{top} \qquad \text{Equation 3.32}$$

$$A_{bot} = L_{bot} * W_{bot} \qquad \text{Equation 3.33}$$

$$V = \frac{D}{3}\left(A_{top} + A_{bot} + \sqrt{A_{top} * A_{bot}}\right) \qquad \text{Equation 3.34}$$

where L_{top} is basin length in feet at the top of the basin, L_{bot} is basin length in feet at the bottom of the basin (84 ft), z is the side slope coefficient (3), D is basin depth in feet (6 ft), W_{top} is basin width in feet at the top of the basin, W_{bot} is basin width in feet at the bottom of the basin (24 ft), A_{top} is basin plan area in ft^2 at the top of the basin, A_{bot} is basin plan area in ft^2 at the bottom of the basin, and V is basin volume in ft^3.

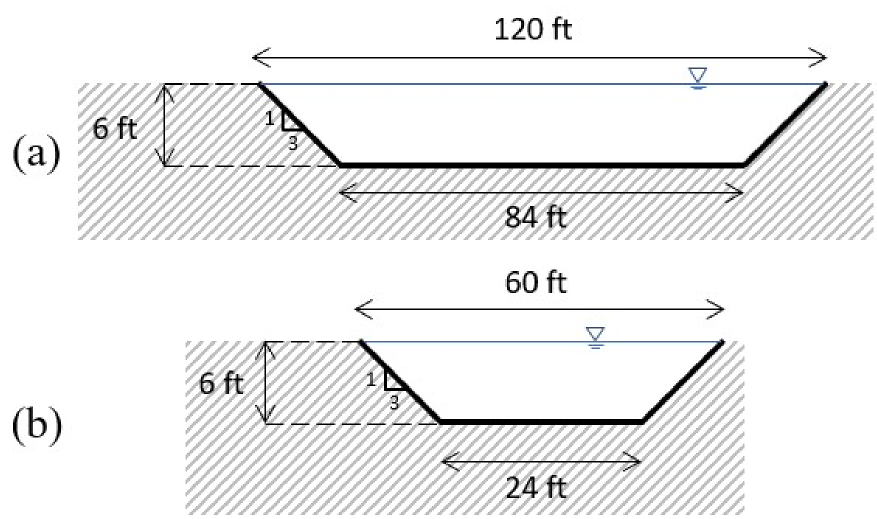


Figure 3.17 Case study site detention basin design, (a) length dimensions, (b) width dimensions

The detention basin construction cost estimate was calculated using Equation 3.35:

$$Cost_{basin} = U_V * V + U_A * A \qquad \text{Equation 3.35}$$

where $Cost_{basin}$ is detention basin construction cost in dollars, U_V is excavation unit cost in \$/ft^3 (Table 3.9), V is basin volume in ft^3, U_A is land unit cost in \$/ft^2 (Table 3.9), and A is basin plan area in ft^2. The basin volume and plan area used in the cost calculation were calculated with Equation 3.30 through Equation 3.34 using a basin depth D of 6 feet.

The detention basin is drained by a pre-cast circular concrete outlet structure. The outlet structure had an 8-inch circular orifice with a 0.5-ft vertical offset and a weir spillway with a 5-ft vertical offset (Figure 3.18).

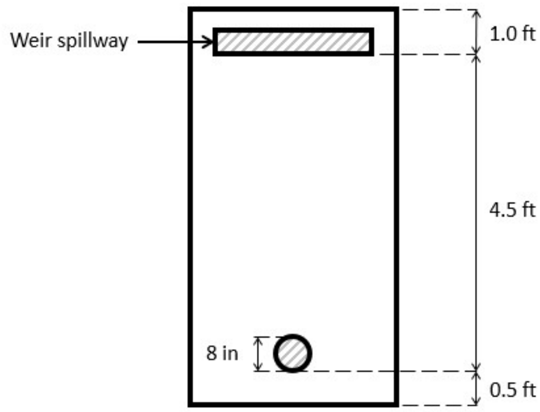


Figure 3.18 Case study site detention basin outlet structure design

When water in the basin exceeds a stage of 5 feet, runoff is discharged through the high-flow weir spillway. The volume of water stored in the basin when it reaches a stage of 5 feet is the maximum storage volume of the basin. This volume was calculated with Equation 3.30 through Equation 3.34 using a basin depth *D* of 5 feet. The vertical offset of the 8-inch orifice allows a small amount of water to be detained in the bottom 0.5 feet of the basin. This is referred to as "dead" storage and was calculated with Equation 3.30 through Equation 3.34 using a basin depth *D* of 0.5 feet. Table 3.8 contains the calculated dimensions for the case study site detention basin.

Table 3.8 Case study detention basin dimensions and cost estimate

Detention basin design	
Basin plan area (ft^2)	7200
Basin depth (ft)	6
Basin volume (ft^3)	26052
Basin max storage volume (ft^3)	19491
Basin "dead" storage volume (ft^3)	1090
Orifice diameter (in)	8
Orifice area (ft^2)	0.349
Basin total construction cost	$ 11,011.62

Unit costs were developed for the cost estimates of an infiltration trench, detention basin land cost, and detention basin excavation. The infiltration trench unit cost was calculated by dividing the treatment volume of the case study infiltration trench by the estimated construction cost (Equation 3.36):

$$U_{IT} = \frac{Tv}{Cost_{IT}} \qquad \text{Equation 3.36}$$

where U_{IT} is infiltration trench unit cost in \$/ft^3, Tv is infiltration trench treatment volume (1185 ft^3), and $Cost_{IT}$ is infiltration trench cost (\$5000). Detention basin excavation unit cost was calculated assuming \$5 per yd^3 of soil excavated. Land unit cost was calculated assuming a value of \$37,500 per acre for the case study property (Alabama GIS, 2019). The unit costs used for this analysis are shown in Table 3.9.

Table 3.9 Unit costs developed for case study site

Unit cost	Value
Infiltration trench ($/ft^3)	4.22
Land ($/ft^2)	0.86
Excavation ($/ft^3)	0.19

3.7 Sensitivity Analysis

A sensitivity analysis was performed to examine the relationships between the many variables used in calculations within the tool. Many values can be directly calculated without using the tool, but the hydrologic modeling and optimization calculations cannot be directly examined without applying the tool. The tool was applied for 36 design scenarios, varying imperviousness and water quality design rainfall for each trial. Table 3.10 illustrates the systematic variation of design variables and shows reference numbers assigned for each unique design scenario.

Table 3.10 Sensitivity analysis conditions for first 25 trials representing a small storm

Trial Numbers						
A_T (ac)	10					
P_2 (in)	4					
$\boldsymbol{A_{imp}/A_T}$	$\boldsymbol{P_{wq}/P_2}$					
	0	0.2	0.4	0.6	0.8	1.0
0	1	2	3	4	5	6
0.2	7	8	9	10	11	12
0.4	13	14	15	16	17	18
0.6	19	20	21	22	23	24
0.8	25	26	27	28	29	30
1.0	31	32	33	34	35	36

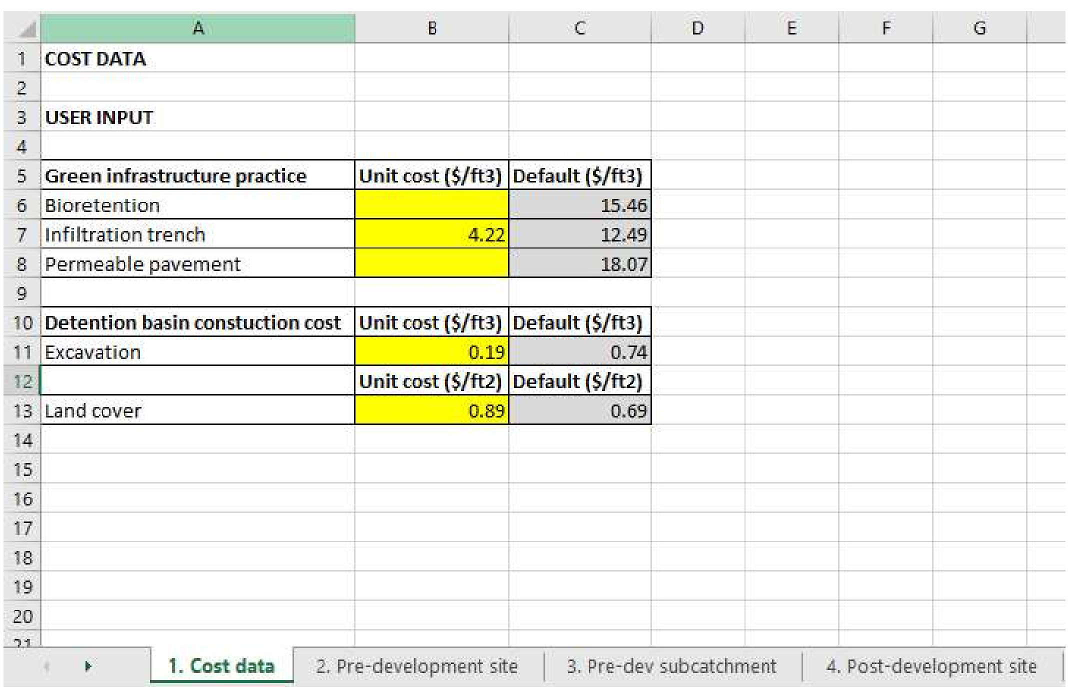
Impervious area (A_{imp}) was divided by a total area (A_T) and water quality design rainfall (P_{wq}) was divided by flood protection design rainfall (P_2) so that results of the sensitivity analysis can be interpreted for different stormwater design objectives. Calculations were based on a total area of 10 acres and a flood protection design rainfall of 4 inches. Each trial was modeled as having a post-development site plan of one impervious and one impervious subcatchment. Pervious subcatchments were models as having a Curve Number of 60 and all subcatchments were modeled as having a 2% slope. The 24-hour synthetic rainfall distribution was used as the design storm. Impervious subcatchments were modeled as having an infiltration trench. Cost estimates were calculated using the unit costs in Table 3.9.

4. Results

4.1 User Interface

The methods described in Chapter 3 were used to develop a spreadsheet-based decision-support tool for cost-effective stormwater management designs. The user interface of the resulting tool was developed in Microsoft Excel and uses separate worksheets to guide the user through the process of completing a design using the tool. The worksheets are ordered and described as follows:

1. Cost data (Figure 4.1) – This worksheet contains two tables in which the user can input unit costs in $\$/ft^3$ for bioretention, infiltration trench, and permeable pavement green infrastructure practices, as well as a unit cost in $\$/ft^2$ for land costs of a detention basin and a unit cost in $\$/ft^3$ for excavation costs of a detention basin. User input is not necessary on this worksheet; if no user-defined unit costs are entered, the tool will use the default values.



COST DATA

USER INPUT

Green infrastructure practice	Unit cost ($/ft3)	Default ($/ft3)
Bioretention		15.46
Infiltration trench	4.22	12.49
Permeable pavement		18.07

Detention basin constuction cost	Unit cost ($/ft3)	Default ($/ft3)
Excavation	0.19	0.74
	Unit cost ($/ft2)	Default ($/ft2)
Land cover	0.89	0.69

Figure 4.1 User interface: Cost data worksheet

2. Pre-development site (Figure 4.2) – On this worksheet, the user describes the basic pre-development site conditions. The user must enter a water quality design rainfall depth (usually obtained from a state or local design manual) and a flood protection design rainfall, which is usually specified by a state or local manual as a rainfall event with a specific return period (2-year, 25-year, etc.). Rainfall frequency estimates can be easily obtained using the NOAA Atlas 14 maps (NWS, 2020). The user must also select either a 2-hour or 24-hour rainfall duration. The user must input the number of discrete land usage areas in the pre-development site design and enter the total site area in acres. The CommandButton on this worksheet edits the SWMM input files for hydrologic modeling of the user's pre-development conditions.

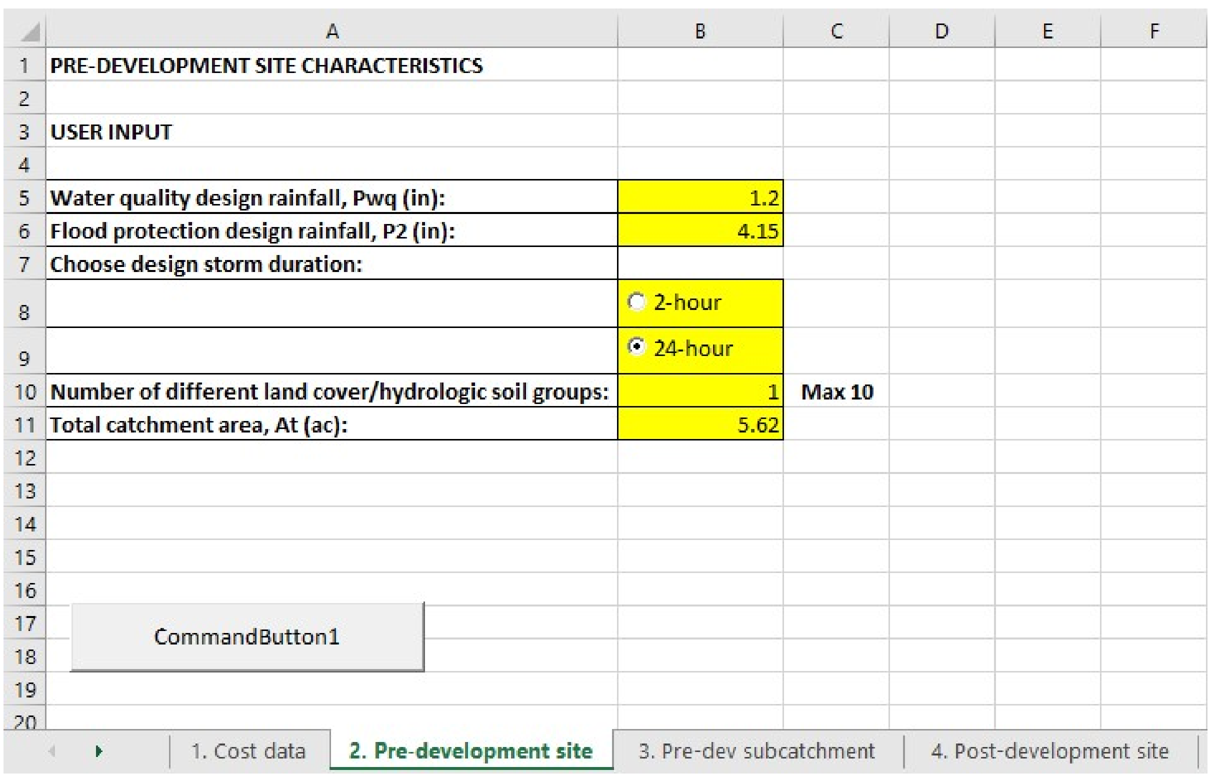


Figure 4.2 User interface: Pre-development site conditions worksheet

3. Pre-development subcatchments (Figure 4.3) – On this worksheet the user enters area in acres, average slope percentage, and Curve Number for each of the discrete subcatchments specified on Sheet 2. The CommandButton on this worksheet executes the pre-development SWMM model.

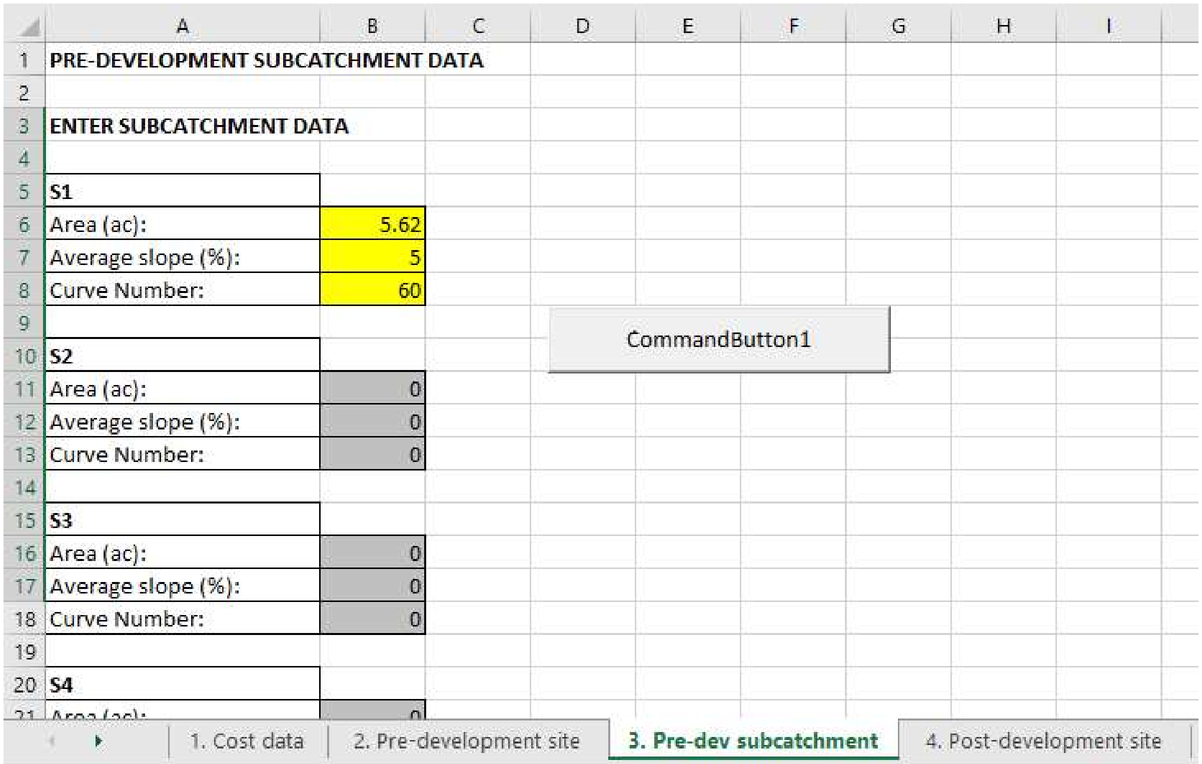


Figure 4.3 User interface: Pre-development subcatchment data worksheet

4. Post-development site (Figure 4.4) – On this worksheet the user specifies the number of impervious and pervious subcatchments comprising the post-development site plan. Discretization of different subcatchments should be based on land cover conditions. The CommandButton on this worksheet unlocks input boxes on Sheet 5 for the number of subcatchments specified.

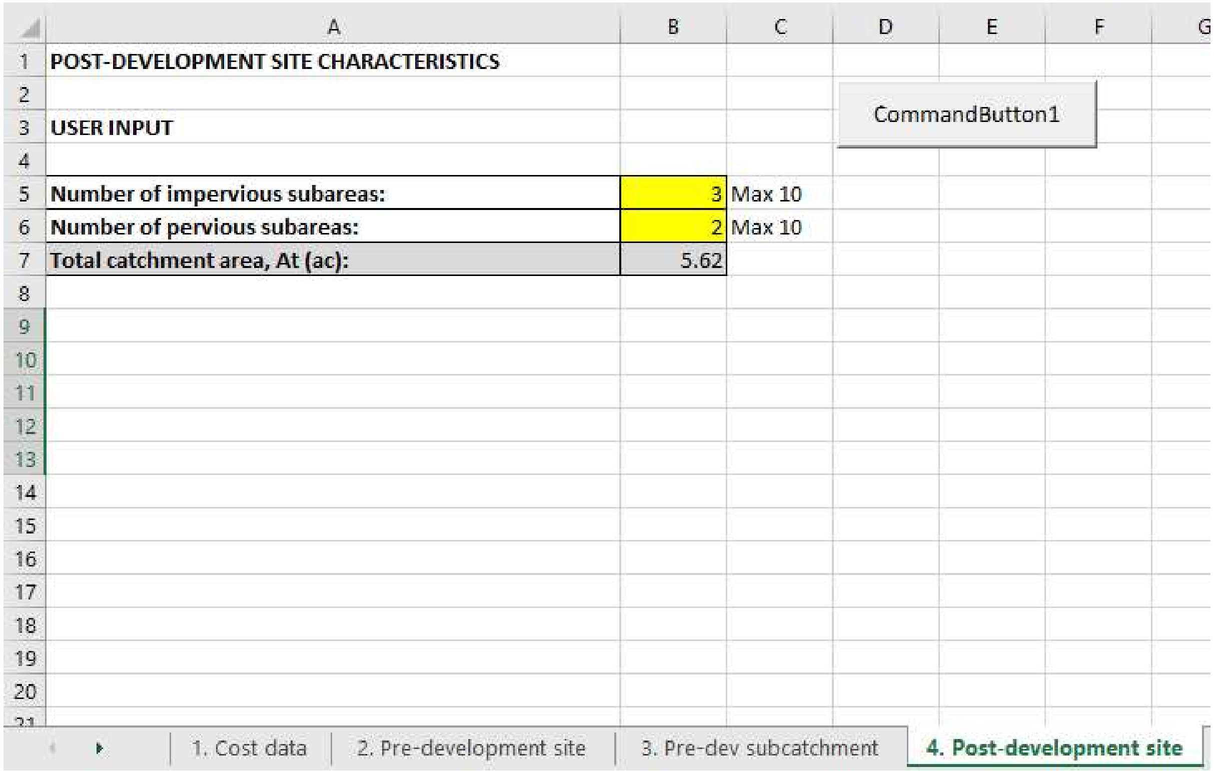


Figure 4.4 User interface: Post-development site characteristics worksheet

5. Post-development subcatchments (Figure 4.5) – On this worksheet, the user enters subcatchment properties for each subcatchment in the post-development site design. For each impervious subcatchment, the user must select a description for the subcatchment and select Yes or No for applying an intrinsic GIP. The CommandButton on this worksheet edits the post-development SWMM input file.

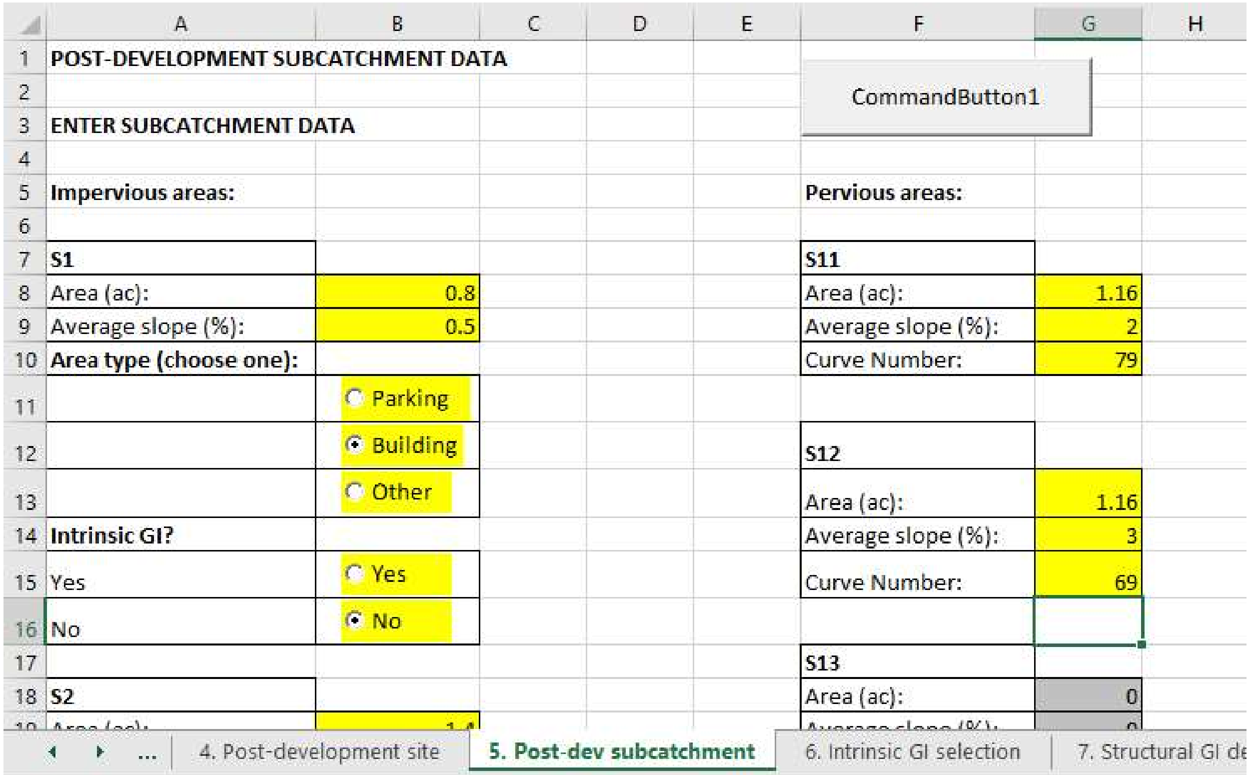


Figure 4.5 User interface: Post-development subcatchment data worksheet

6. Intrinsic GI selection (Figure 4.6) – On this worksheet, the user must select from the applicable intrinsic GIPs to be applied for each impervious subcatchment, as well as select the design level and choose Yes or No for application of a structural GIP. The CommandButton on this worksheet unlocks the applicable structural GIPs on Sheet 7.

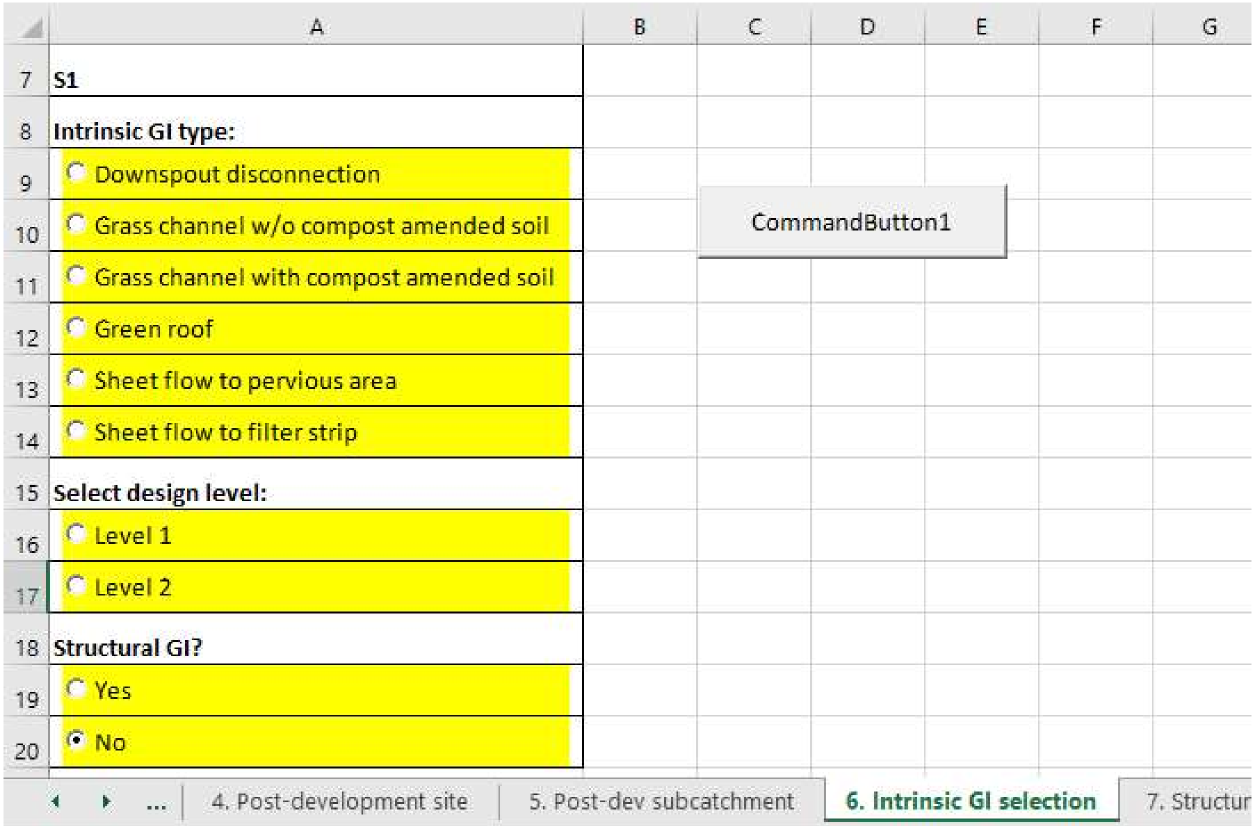


Figure 4.6 User interface: Intrinsic GI selection worksheet

7. Structural GI design (Figure 4.7) – On this worksheet, the user selects a structural GI to apply to each impervious subcatchment from the available options. The CommandButton on this worksheet executes the post-development SWMM model and calculates tool results.

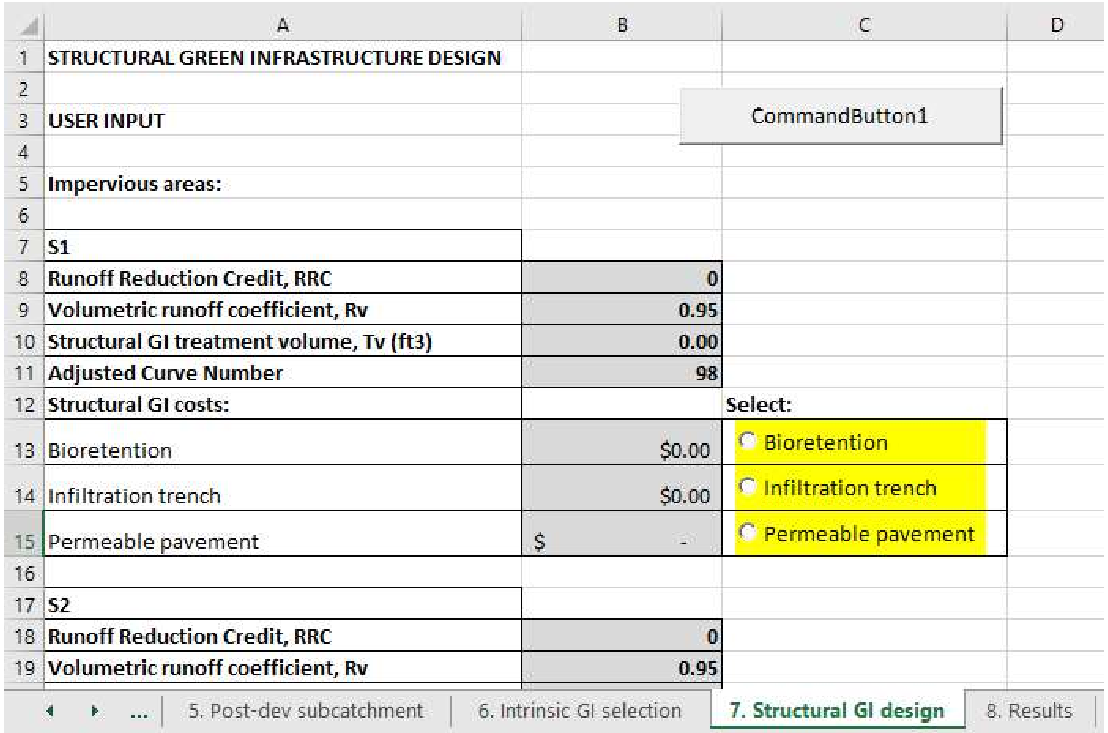


Figure 4.7 User interface: Structure GI design worksheet

8. Results (Figure 4.8) – This worksheet contains a summary of the results. The results summary can be used to develop implementation-level stormwater designs and contains the necessary information for adapting the developed detention basin optimization model.

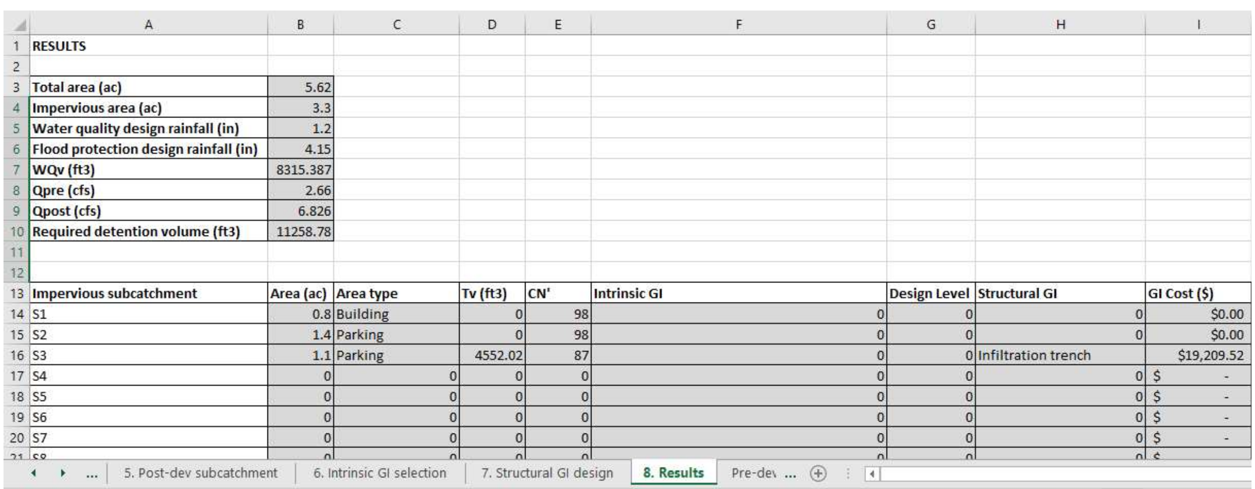


Figure 4.8 User interface: Results summary worksheet

The remaining worksheets in the Excel file are *Pre-development SWMM input, Post-development SWMM input, Pre-development SWMM output, Post-development SWMM output, Hydrographs,* and *Precipitation.* These worksheets are used for internal data handling but may contain helpful or supplementary information for experienced users.

4.2 Case Study Results

The developed tool was used to perform planning-level design and cost estimation for a site-specific stormwater management plan using green infrastructure for a 5.62-acre urban development case study site that employed an infiltration trench and an on-site detention basin (Figure 3.14).

The green infrastructure design process was applied for the infiltration trench in subcatchment S3. The infiltration trench designed with the tool had a treatment volume 3.84 times larger than the treatment volume of the infiltration trench designed for the case study site, resulting in a 284% increase in cost. The calculated cost is greater than the original estimate because the infiltration trench was designed to treat the entire runoff volume produced by 1.2 inches of rainfall on the 1.1-acre parking area. The original infiltration trench design is only large enough to treat a portion of this runoff volume. These results are shown in Table 4.1.

Table 4.1 Comparison of original infiltration trench design and design calculated with tool

Infiltration trench results	
Designed treatment volume (ft^3)	1185
Calculated treatment volume (ft^3)	4552
Designed cost estimate	$ 5,000.00
Calculated cost estimate	$ 19,209.52

The runoff reduction resulting from the use of an infiltration trench resulted in an adjusted Curve Number of 87 for subcatchment S3, an 11-unit reduction. The pre-development, post-development, and approximated basin outflow hydrographs were calculated and generated by the tool (Figure 4.9). The peak pre-development runoff was 2.66 cfs and the peak post-development runoff was 6.83 cfs.

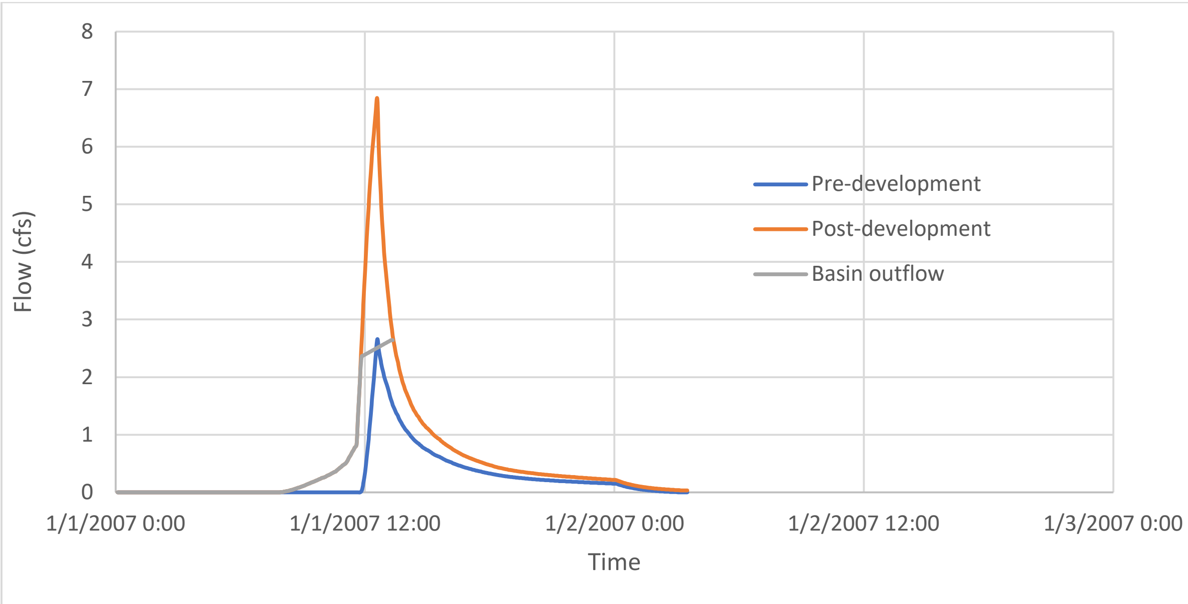


Figure 4.9 Hydrographs calculated with the developed tool for case study site

The required storage volume for a detention basin was calculated from the hydrographs in Figure 4.9. The tool also calculated the water quality volume (*WQv*) for the case study site. The tool-calculated basin volume was compared to the volume calculated from the original basin design, and the water quality volume was compared to the "dead" storage calculated for the original basin design (Table 4.2).

Table 4.2 Comparison of original detention basin design and design calculated with tool

Detention basin results	
Designed storage volume (ft^3)	19491
Calculated storage volume (ft^3)	11259
Designed "dead" storage (ft^3)	1090
Calculated WQv (ft^3)	8315

The required detention basin storage volume calculated by the tool was 42.2% smaller than the storage volume of the original basin design. Additionally, the "dead" storage provided by the vertical offset of the outlet structure in the original design did not provide enough storage to meet the requirement to detain the water quality volume calculated with the tool.

These results were used to adapt the detention basin optimization model. The calculated required basin storage volume, water quality volume, pre-development peak flow, and the unit costs in Table 3.9 were applied to the developed optimization model and the model was solved using the General Algebraic Modeling Software (GAMS). A maximum basin depth of 5 feet and a *WQv* detention time of 24 hours (86400 seconds) were used. The values for the relevant parameters are compiled in Table 4.3.

Table 4.3 Case study detention basin design parameters for optimization model

Detention basin optimization design parameters	
WQv (ft^3)	8315
Required basin volume (ft^3)	11259
Peak pre-development runoff (cfs)	2.66
WQv detention time (sec)	86400
Maximum basin depth (ft)	5
Excavation unit cost ($\$/ft^3$)	0.19
Land unit cost ($\$/ft^3$)	0.89

The results of the detention basin optimization are shown in Table 4.4. The estimated construction cost for the optimized basin design was $4,143.31. To compare the cost of the optimized basin design to the cost of the basin designed for the case study site, the additional construction cost of a 1-foot freeboard was calculated using Equation 3.35 and added to the value above, resulting in a total cost of $4,571.14. This optimized basin cost estimate was 58.5% lower than the cost of the original basin design.

Table 4.4 Optimized design for case study detention basin

Optimized detention basin design	
Lower pond area (ft^2)	2252
Lower pond height (ft)	3.69
Upper pond area (ft^2)	2252
Upper pond height (ft)	1.31
Lower pond orifice area (ft^2)	0.019
Upper pond orifice area (ft^2)	0.41
Basin construction cost	$ 4,143.31
Basin construction cost w/ freeboard	$ 4,571.14

Some of the cost reductions can be attributed to the rectangular design of the optimized basin. The trapezoidal design of the case study basin requires a larger plan area occupied by the basin compared to a rectangular design with vertical side slopes, resulting in greater land costs for a trapezoidal basin. Following the 2:1 length to width ratio of the original basin design, Figure 4.10 illustrates the dimensions of the optimized basin design. Figure 4.11 shows the resulting optimized outlet structure design.

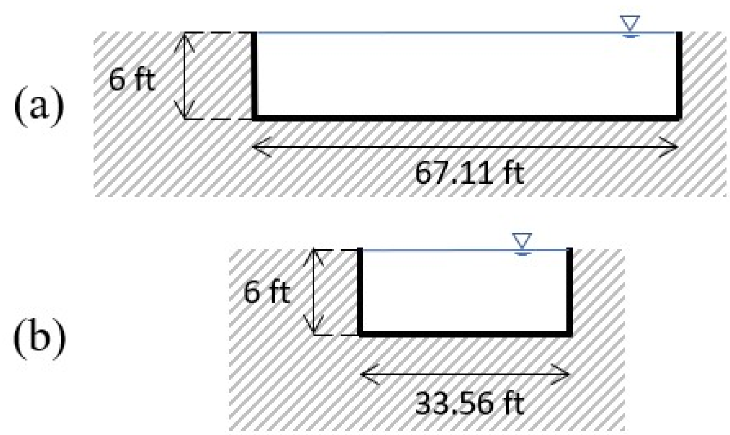


Figure 4.10 Optimized detention basin design, (a) length dimensions, (b) width dimensions

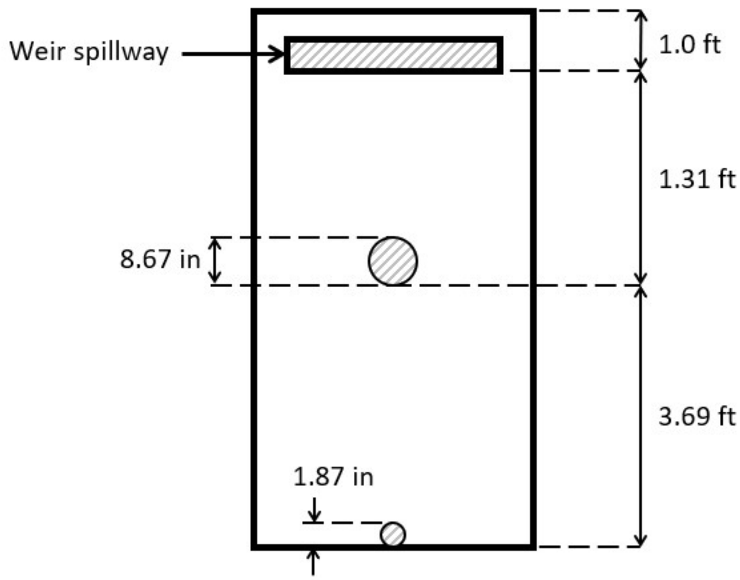


Figure 4.11 Optimized detention basin outlet structure design

The total cost of the infiltration trench and optimized detention basin design resulting from applying the developed tool for the case study site was $23,780.66, which is a 48.5% increase from the original cost estimate of $16,011.62. Table 4.5 shows a summary of the cost estimate results.

Table 4.5 Summary of cost estimates calculated with tool compared to original cost estimates

Cost estimate	Change from original
Infiltration trench	284% increase
Detention basin	58.5% decrease
Total	48.5% increase

These results illustrated the trade-off between green infrastructure costs and detention basin costs. Construction of larger green infrastructure facilities will reduce runoff volumes but can result in greater overall costs past a break-even point. Identification of the break-even point can be facilitated by iterative use of the tool developed in this study. For example, the above case study site was modeled again using the tool, but subcatchment S3 was arbitrarily split into two subcatchments of 0.78 acres and 0.32 acres. The 0.32-acre portion of S3 was modeled as having an infiltration trench. The resulting infiltration trench design is shown in Table 4.6.

Table 4.6 Infiltration trench design calculated by tool for new subcatchment discretization

Infiltration trench results	
Designed treatment volume (ft^3)	1185
Calculated treatment volume (ft^3)	1324
Designed cost estimate	$ 5,000.00
Calculated cost estimate	$ 5,588.23

Because the volume of runoff captured by the infiltration trench decreased compared to the first run of the tool for the case study site, the required volume for the detention basin increased to 12550 ft^3. The detention basin optimization model was solved for the new design. The results of the basin optimization are shown in Table 4.7.

Table 4.7 Optimized detention basin design for new subcatchment discretization

Optimized detention basin design	
Lower pond area (ft^2)	2510
Lower pond height (ft)	3.31
Upper pond area (ft^2)	2510
Upper pond height (ft)	1.69
Lower pond orifice area (ft^2)	0.02
Upper pond orifice area (ft^2)	0.36
Basin construction cost	$ 4,618.40
Basin construction cost w/ freeboard	$ 5,095.30

The new combined cost of the infiltration trench and detention basin was $10,683.53, a 3.0% decrease. This run of the tool resulted in a larger infiltration trench design and a lower total cost that the actual case study design, thus providing more treatment for stormwater quality and improving cost-effectiveness. These results also illustrate the sensitivities of the tool that are explored further in Section 4.3. Table 4.8 shows a summary of the new cost results.

Table 4.8 Summary of cost estimates for new subcatchment discretization

Cost estimate	Change from case study
Infiltration trench	11.8% increase
Detention basin	53.7% decrease
Total	3.0% decrease

There were several qualitative results from the case study analysis. Typically, a stormwater designer will perform trial-and-error design calculations for green infrastructure practices and detention basins, sometimes with the help of a rainfall-runoff modeling software like SWMM which can be time consuming and may require calibration of a watershed model. Both runs of the developed tool for the case study site took approximately 10 minutes to complete on a consumer laptop computer.

4.3 Sensitivity Analysis Results

The results for the 36 trials of the sensitivity analysis are compiled in Table 4.9. As the green infrastructure treatment volume increases as a result of varying the design rainfall and imperviousness, the required basin volume generally decreases. However, the water quality capture volume (*WQv*) requirement for the detention basin design is shown to surpass the calculated required basin volume as water quality design rainfall increases, becoming the main driver of basin cost. The results also illustrate the hydrologic impact of green infrastructure, reducing the required basin volume to zero in many cases. The green infrastructure cost, basin cost, and total cost were plotted as functions of the independent variables for the sensitivity analysis. The resulting plots are shown in Figure 4.12 through Figure 4.17.

Table 4.9 Sensitivity analysis results

Sensitivity analysis results								
Trial Number	A_{imp}/A_T	P_{wq}/P_2	Tv (ft^3)	GI cost ($)	WQv (ft^3)	Vreq (ft^3)	Basin cost ($)	Total cost ($)
1	0	0	0	0	0	0	0	0
2	0	0.2	0	0	0	0	0	0
3	0	0.4	0	0	0	0	0	0
4	0	0.6	0	0	0	0	0	0
5	0	0.8	0	0	0	0	0	0
6	0	1	0	0	0	0	0	0

Sensitivity analysis results								
Trial Number	A_{imp}/A_T	P_{wq}/P_2	Tv (ft^3)	GI cost ($)	WQv (ft^3)	Vreq (ft^3)	Basin cost ($)	Total cost ($)
7	0.2	0	0	0	0	4360	1604	1604
8	0.2	0.2	5518	23284	401	3377	1243	24527
9	0.2	0.4	11035	46569	802	2019	983	47552
10	0.2	0.6	16553	69853	1202	622	1475	71328
11	0.2	0.8	22070	93137	1603	18	1967	95104
12	0.2	1	27588	116421	2004	0	2458	118880

Sensitivity analysis results								
Trial Number	A_{imp}/A_T	P_{wq}/P_2	Tv (ft^3)	GI cost ($)	WQv (ft^3)	Vreq (ft^3)	Basin cost ($)	Total cost ($)
13	0.4	0	0	0	0	12694	4672	4672
14	0.4	0.2	11035.2	46569	1429	9284	3417	49985
15	0.4	0.4	22070.4	93137	2858	5321	3506	96643
16	0.4	0.6	33105.6	139706	4286	1621	5259	144965
17	0.4	0.8	44140.8	186274	5715	0	7012	193286
18	0.4	1	55176	232843	7144	0	8765	241608

Sensitivity analysis results								
Trial Number	A_{imp}/A_T	P_{wq}/P_2	Tv (ft^3)	GI cost ($)	WQv (ft^3)	Vreq (ft^3)	Basin cost ($)	Total cost ($)
19	0.6	0	0	0	0	22795	8388	8388
20	0.6	0.2	16552.8	69853	3084	16857	6203	76056
21	0.6	0.4	33105.6	139706	6168	9855	7568	147273
22	0.6	0.6	49658.4	209558	9252	3074	11352	220910
23	0.6	0.8	66211.2	279411	12336	0	15135	294547
24	0.6	1	82764	349264	15420	0	18919	368183

Sensitivity analysis results								
Trial Number	A_{imp}/A_T	P_{wq}/P_2	Tv (ft^3)	GI cost ($)	WQv (ft^3)	Vreq (ft^3)	Basin cost ($)	Total cost ($)
25	0.8	0	0	0	0	33930	12486	12486
26	0.8	0.2	22070.4	93137	5367	25445	9364	102501
27	0.8	0.4	44140.8	186274	10733	15106	13169	199443
28	0.8	0.6	66211.2	279411	16100	4889	19753	299164
29	0.8	0.8	88281.6	372548	21466	0	26337	398886
30	0.8	1	110352	465685	26833	0	32922	498607

Sensitivity analysis results								
Trial Number	A_{imp}/A_T	P_{wq}/P_2	Tv (ft^3)	GI cost ($)	WQv (ft^3)	Vreq (ft^3)	Basin cost ($)	Total cost ($)
31	1.0	0	0	0	0	44821	16494	16494
32	1.0	0.2	27588	116421	8276	33901	12476	128897
33	1.0	0.4	55176	232843	16553	20303	20309	253152
34	1.0	0.6	82764	349264	24829	6644	30463	379727
35	1.0	0.8	110352	465685	33106	0	40618	506303
36	1.0	1	137940	582107	41382	0	50772	632879

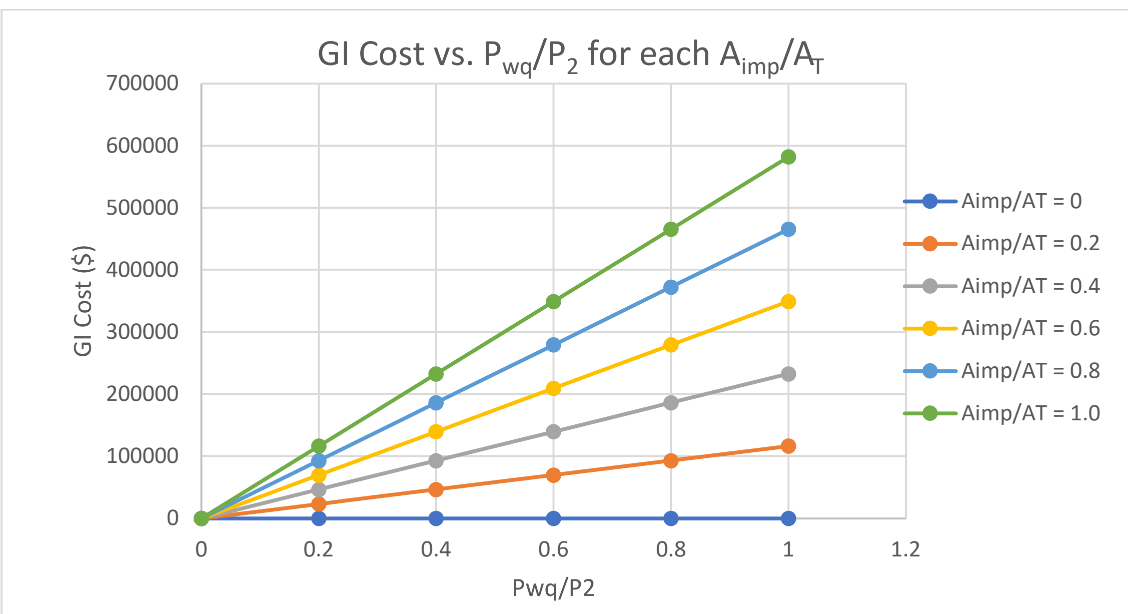


Figure 4.12 Green infrastructure cost as a function of water quality design rainfall variation

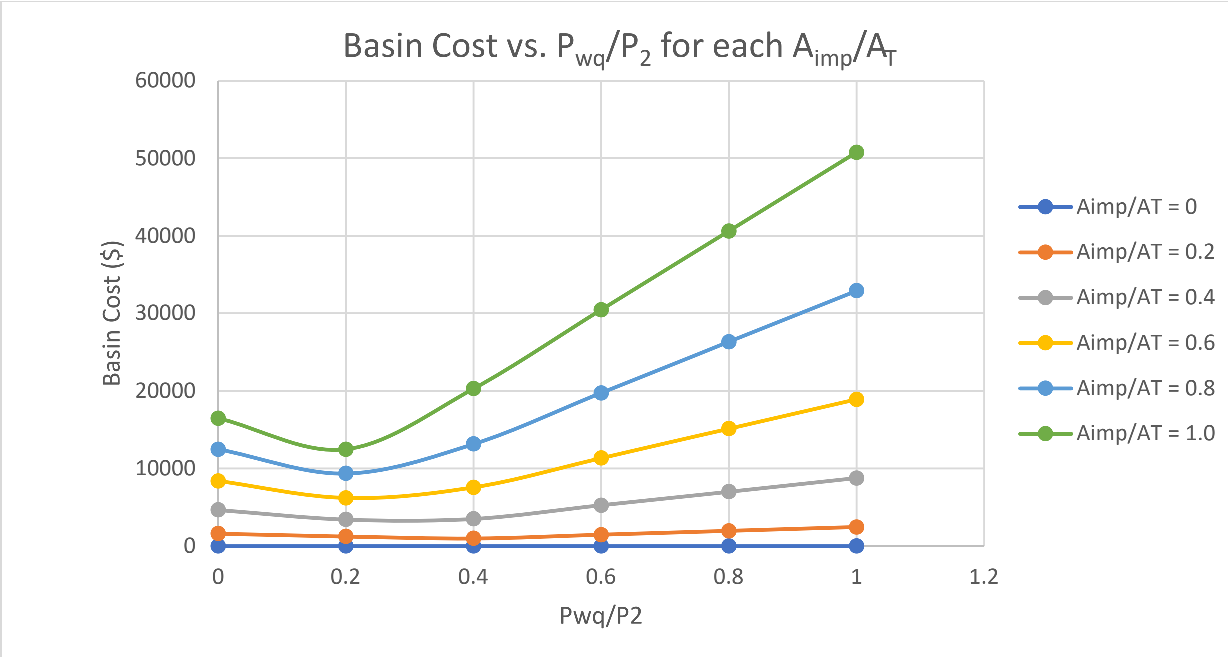


Figure 4.13 Detention basin cost as a function of water quality design rainfall variation

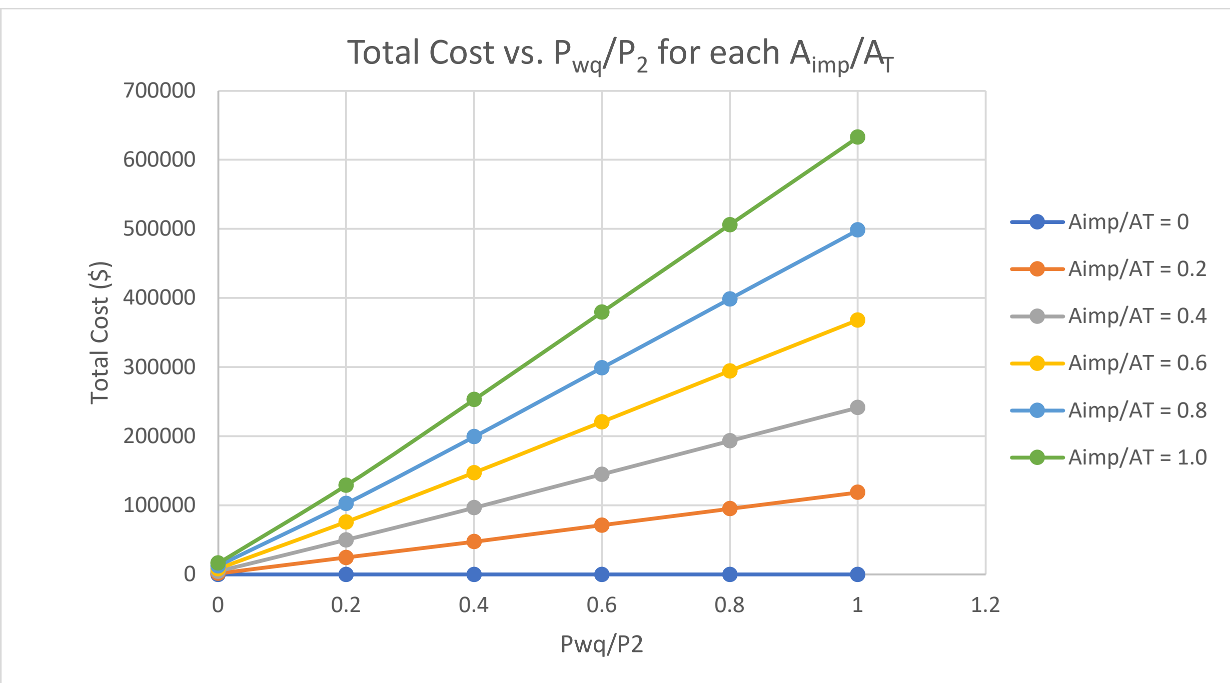


Figure 4.14 Total cost as a function of water quality design rainfall variation

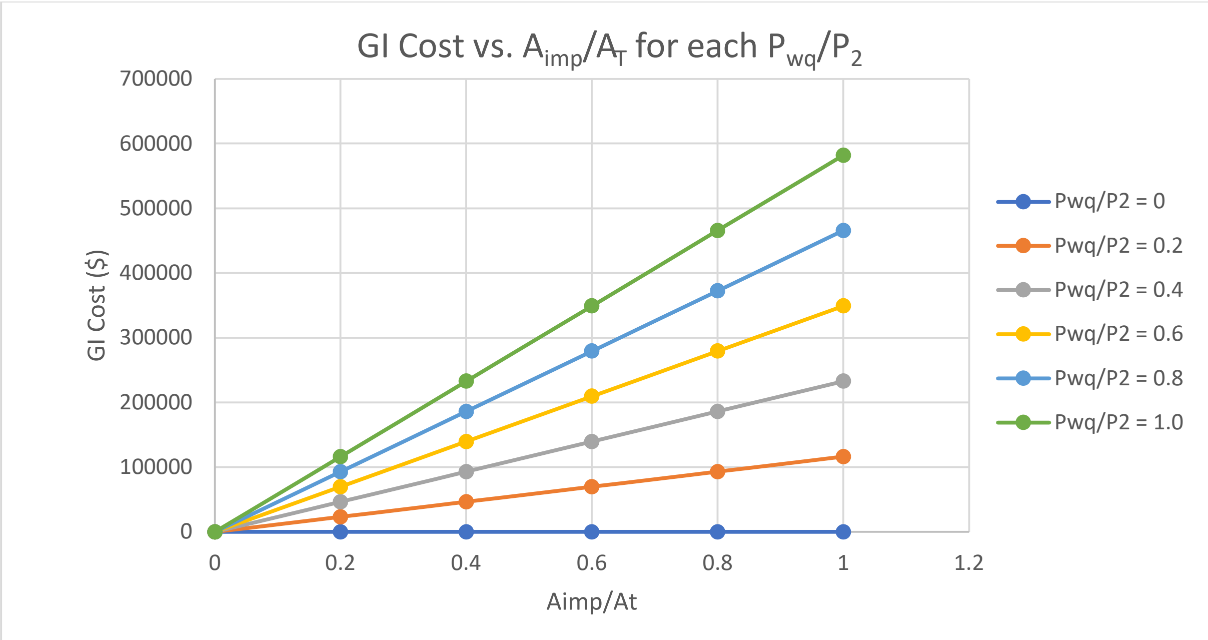


Figure 4.15 Green infrastructure cost as a function of variation in imperviousness

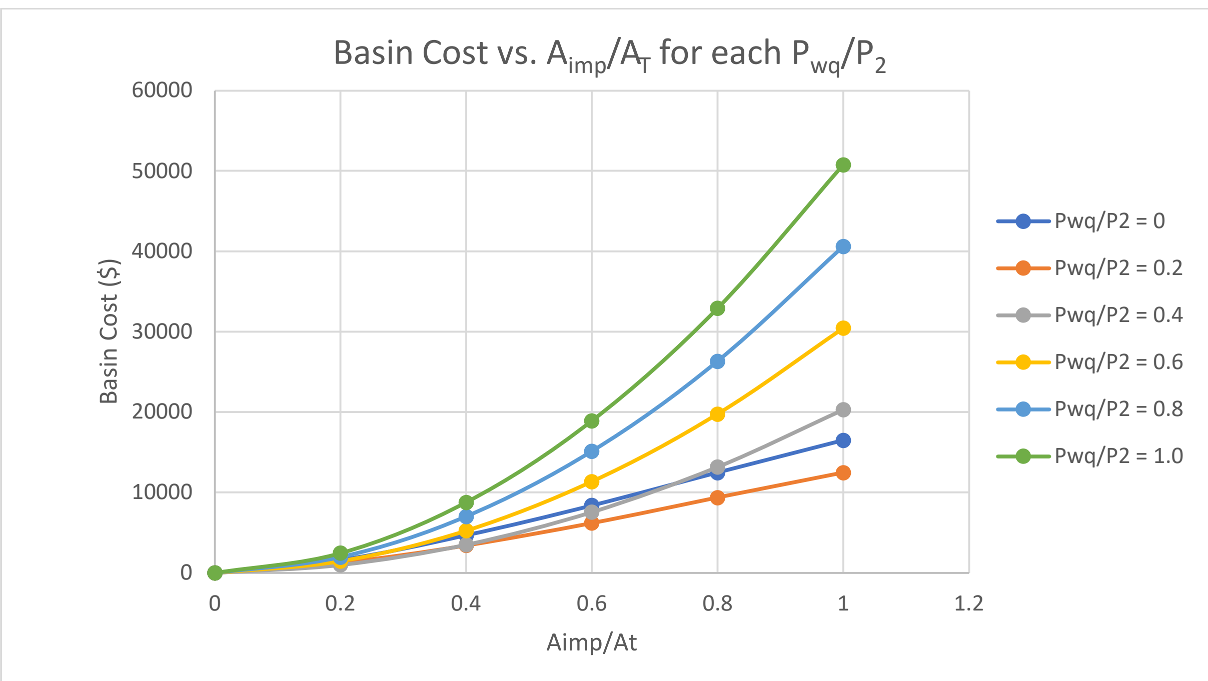


Figure 4.16 Detention basin cost as a function of variation in imperviousness

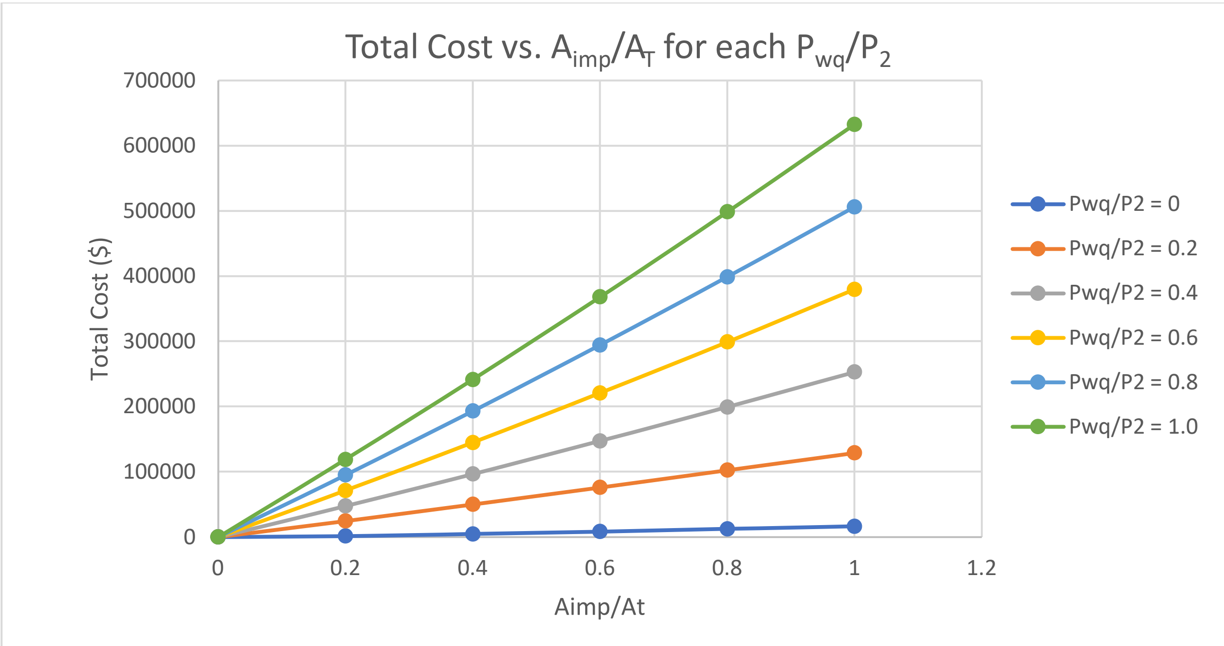


Figure 4.17 Total cost as a function of variation in imperviousness

The above figures illustrate the sensitivity of the developed tool to user-defined ratios of water quality design rainfall to flood protection design rainfall, as well as the sensitivity to the imperviousness of a site design. Figure 4.13 illustrates local minima in detention basin costs for any imperviousness where the ratio of water quality design rainfall to flood protection design rainfall is equal to 0.2. Figure 4.18 shows that these local minima exists for all design scenarios considered in the sensitivity analysis. This result suggests that for this tool, a water quality rainfall depth equal to 20% of the flood protection rainfall depth yields a minimum-cost basin design.

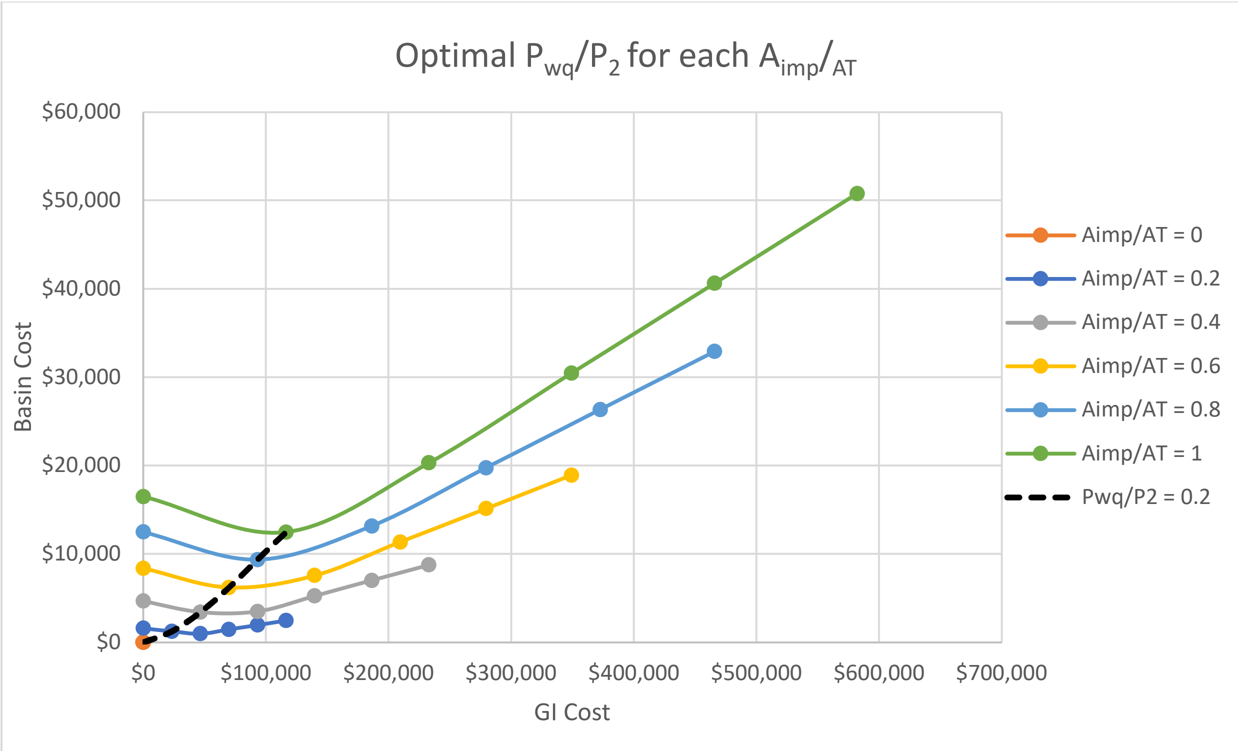


Figure 4.18 Local minima for detention basin costs as a function of green infrastructure costs

5. Conclusions

5.1 Research Conclusions

This study developed a decision-support tool to aid stormwater practitioners in selecting cost-effective designs for green infrastructure and detention basin BMPs. The tool was developed with the objective of reconciling the accuracy of the results with the accessibility of the methods. The tool was developed with specific regard to the most recent developments in the regulatory framework for stormwater quality, providing the user with a means to comply with local and federal regulations. The green infrastructure selection process and detention basin optimization model give the user a realistic starting point for planning and design, reducing the amount of time and labor required for developing stormwater management plans. The tool was developed using Microsoft Excel, which is widely available to stormwater practitioners at any level of expertise. The user interface facilitates planning-level site design for using green infrastructure based on minimal user input and intervention.

The case study performed in this research illustrated how the tool can be used to develop post-construction stormwater designs for a small urban catchment. The initial run of the tool for the case study was representative of how the tool would be applied by inexperienced users. The results generated for the first case study trial were realistic and reasonable, indicating that the calculation and modeling methods of the tool are valid for planning-level analyses. The second case study trial represented use by an experienced designer or a user more familiar with the tool. This second trial yielded a post-development stormwater design that provided more water quality treatment for a lower cost, resulting in better cost-effectiveness than the original design. The case study highlighted a common issue in using green infrastructure: cost-effective design of GIPs is highly dependent on the judgement and experience of the designer. Both case study trials were completed in approximately 10 minutes. Performing the same analysis manually, even with the help of modeling software, would take much more time and labor. This tool can help to bridge the gap in knowledge and experience in GI design by facilitating easy trials of alternative designs.

The sensitivity analysis performed in this study illustrated the hydrologic and economic trade-off relationship between green infrastructure and detention basin BMPs. The results illustrate that when only construction cost is considered, there is no design scenario where it is less costly to use green infrastructure than it is to not use any green infrastructure. Any cost reduction resulting from the use of green infrastructure must be calculated via a lifecycle cost analysis, which is consistent with the findings of various case studies on green infrastructure (US EPA, 2013, 2014). The sensitivity analysis illustrated that there exists an optimal proportion of runoff to be treated with green infrastructure to minimize detention basin costs. Given that many stormwater authorities now require the use of green infrastructure, this result could help developers identify least-cost designs for complying with stormwater requirements.

5.2 Recommendations for Future Research

The development of this research and the results discussed in this thesis have several implications for future stormwater management policy and research. The sensitivity analysis illustrated that the requirement for on-site detention basins to store a water quality volume (WQv) is a redundancy that can increase costs when enough green infrastructure is used to capture this volume before it is discharged to a basin. It is recommended that stormwater ordinances provide a waiver of the WQv design requirement for detention basins if green infrastructure provides capture of this quantity. The sensitivity analysis also revealed that the ratio of a water quality design rainfall to flood protection design rainfall can minimize costs when equal to 0.2. The water quality design rainfall used in much of the Southeastern United States is 1.2 inches, meaning that the optimal design storm is 6.0 inches. Based on this result, it is recommended that local stormwater policymakers evaluate whether changes to either the water quality design rainfall or the flood protection design storm can be made to reduce costs while still meeting stormwater management goals.

There were many topics not considered within the scope of this study that should be explored in future research efforts. It is recommended that the tool developed here be expanded in future versions to include a lifecycle cost analysis of the green infrastructure and detention basin designs, with the objective of minimizing net present value (NPV). The successive case study trials conducted in this study suggest that iterative application of the tool can yield a single optimal design based on user-defined subcatchment discretization. It is recommended that future research efforts develop an algorithm for automating iterative application of the tool, similar to a heuristic optimization technique. The General Algebraic Modeling System (GAMS) was used in this research to develop and solve the non-linear programming model for a detention basin optimization. GAMS is a powerful platform with numerous industry applications and the modeling capabilities are beyond what is required for this research. Additionally, the free version of GAMS limits models to 10 equations and 10 variables, which prevents models of more complex detention basin designs. It is recommended that future research develop detention basin models using a simpler tool with no restrictions that can be easily coupled with Excel spreadsheets. Finally, it is recommended that the tool developed here be adapted and applied to the similar problems in the field of erosion and sediment control, in which there is ample opportunity and need for cost-effectiveness optimization tools.

References

ADEM. (2016). *Low Impact Development Handbook for the State of Alabama*. Alabama Department of Environmental Management. http://adem.state.al.us/programs/water/constructionstormwater.cnt

Akan, A. O. (1992). Horton Infiltration Equation Revisited. *Journal of Irrigation and Drainage Engineering*, *118*(5), 828–830. https://doi.org/10.1061/(ASCE)0733-9437(1992)118:5(828)

Alabama GIS. (2019). *Lee County Public GIS*. https://www.alabamagis.com/Lee/frameset.cfm?cfid=957120&cftoken=59473796

ALDOT. (2014). *Determining Runoff for Small Storm Events*. Alabama Department of Transportation. https://www.dot.state.al.us/dsweb/divped/Stormwater/index.html

Atlanta Regional Commission. (2016). *Georgia Stormwater Management Manual*. https://atlantaregional.org/natural-resources/water/georgia-stormwater-management-manual/

Baptista, V. S. G., & Paz, A. R. da. (2018). Cost-efficiency analysis of a runoff detention reservoir with integrated hydraulic and structural dimensioning. *RBRH*, *23*(0). https://doi.org/10.1590/2318-0331.231820170168

Center for Neighborhood Technology. (2009). *National Green Values Stormwater Management Calculator*. http://greenvalues.cnt.org/national/calculator.php

City of Auburn. (2019). *Water Resources Management Design and Construction Manual*. auburnal.org

City of Birmingham. (2019). *Post Construction Storm Water Design Manual*. https://www.birminghamal.gov/storm-water-management/post-construction/

Collins, K. A., Hirschman, D., Hoffmann, G., Schueler, T., & Engineer, W. R. (2009). The Runoff Reduction Method. *World Environmental and Water Resources Congress*, 12.

Cook, M. B., & DeBell, K. M. (2001). Improving Water Quality in Urban Watersheds. *Linking Stormwater BMP Designs and Performance to Receiving Water Impact Mitigation*, 24–34. https://doi.org/10.1061/40602(263)3

Damodaram, C., Giacomoni, M. H., Prakash Khedun, C., Holmes, H., Ryan, A., Saour, W., & Zechman, E. M. (2010). Simulation of Combined Best Management Practices and Low Impact Development for Sustainable Stormwater Management. *JAWRA Journal of the American Water Resources Association*, *46*(5), 907–918. https://doi.org/10.1111/j.1752-1688.2010.00462.x

Damodaram, C., & Zechman, E. M. (2013). Simulation-Optimization Approach to Design Low Impact Development for Managing Peak Flow Alterations in Urbanizing Watersheds. *Journal of Water Resources Planning and Management*, *139*(3), 290–298. https://doi.org/10.1061/(ASCE)WR.1943-5452.0000251

GAMS. (2020). *User's Guide*. https://www.gams.com/latest/docs/UG_MAIN.html

Giacomoni, M. H., & Joseph, J. (2017). Multi-Objective Evolutionary Optimization and Monte Carlo Simulation for Placement of Low Impact Development in the Catchment Scale. *Journal of Water Resources Planning and Management*, *143*(9), 04017053. https://doi.org/10.1061/(ASCE)WR.1943-5452.0000812

Lacy, M. L. (2016). *Optimization Model for the Design of Bioretention Basins with Dry Wells* [Arizona State University]. http://hdl.handle.net/2286/R.I.38395

Lee, J. G., Selvakumar, A., Alvi, K., Riverson, J., Zhen, J. X., Shoemaker, L., & Lai, F. (2012). A watershed-scale design optimization model for stormwater best management practices.

Environmental Modelling & Software, *37*, 6–18. https://doi.org/10.1016/j.envsoft.2012.04.011

Madison, C. (2013). Impact of Green Infrastructure on Property Values within the Milwaukee Metropolitan Sewerage District Planning Area. *Center for Economic Development Publications*. https://dc.uwm.edu/ced_pubs/16

Mays, L. W. (2010). *Water Resources Engineering* (2nd ed.). Wiley. https://www.wiley.com/en-us/Water+Resources+Engineering%2C+2nd+Edition-p-9780470460641

National Research Council. (2008). *Urban Stormwater Management in the United States* [Overviews and Factsheets]. US EPA. https://www.epa.gov/npdes/npdes-stormwater-pollution-additional-documents

NRCS. (1986). *Urban Hydrology for Small Watersheds*.

NRCS. (2015). *Information on Rainfall, Frequency, & Distributions | NRCS | NEH Part 630 Ch. 4*. https://www.nrcs.usda.gov/wps/portal/nrcs/detailfull/national/water/?cid=stelprdb1044959

NRCS. (2020). *Web Soil Survey*. https://websoilsurvey.sc.egov.usda.gov/App/HomePage.htm

NWS. (2020). *NOAA Atlas 14 Maps, Volume 9, Version 2*. https://hdsc.nws.noaa.gov/hdsc/pfds/pfds_map_cont.html

Oxley, R. L., & Mays, L. W. (2014). Optimization – Simulation Model for Detention Basin System Design. *Water Resources Management*, *28*(4), 1157–1171. https://doi.org/10.1007/s11269-014-0552-z

Park, M., Chung, G., Yoo, C., & Kim, J.-H. (2012). Optimal design of stormwater detention basin using the genetic algorithm. *KSCE Journal of Civil Engineering*, *16*(4), 660–666. https://doi.org/10.1007/s12205-012-0991-0

Perez-Pedini, C., Limbrunner, J. F., & Vogel, R. M. (2005). Optimal Location of Infiltration-Based Best Management Practices for Storm Water Management. *Journal of Water Resources Planning and Management*, *131*(6), 441–448. https://doi.org/10.1061/(ASCE)0733-9496(2005)131:6(441)

Pitt, R. E. (1987). *Small Storm Urban Flow and Particulate Washoff Contributions to Outfall Discharges*. University of Wisconsin-Madison.

Pitt, R. E. (1999). Small Storm Hydrology and Why it is Important for the Design of Stormwater Control Practices. *Journal of Water Management Modeling*. https://doi.org/10.14796/JWMM.R204-04

Pitt, R. E. (2013). *WinSLAMM Version 10 Runoff Volume, Total Suspended Solids and Other Pollutant Calculations and Regional Calibration Files*.

Rossman, L. A. (2009). *Storm Water Management Model User's Manual Version 5.1*. 353.

Rossman, L. A., & Bernagros, J. T. (2019). *National Stormwater Calculator User's Guide—Version 2.0.0.1*. 99.

Roy-Poirier, A., Champagne, P., & Filion, Y. (2010). Review of Bioretention System Research and Design: Past, Present, and Future. *Journal of Environmental Engineering*, *136*(9), 878–889. https://doi.org/10.1061/(ASCE)EE.1943-7870.0000227

Sample, D. J., Heaney, J. P., Wright, L. T., Fan, C.-Y., Lai, F.-H., & Field, R. (2003). Costs of Best Management Practices and Associated Land for Urban Stormwater Control. *Journal* *of Water Resources Planning and Management*, *129*(1), 59–68. https://doi.org/10.1061/(ASCE)0733-9496(2003)129:1(59)

Shoemaker, L., Riverson, J., Alvi, K., Zhen, J. X., Paul, S., & Rafi, T. (2009). *SUSTAIN -- A Framework for Placement of Best Management Practices in Urban Watersheds to Protect Water Quality*. 202.

Stafford, N., Che, D., & Mays, L. W. (2015). Optimization Model for the Design of Infiltration Basins. *Water Resources Management*, *29*(8), 2789–2804. https://doi.org/10.1007/s11269-015-0970-6

Taylor, S., National Cooperative Highway Research Program, Transportation Research Board, & National Academies of Sciences, Engineering, and Medicine. (2014). *Long-Term Performance and Life-Cycle Costs of Stormwater Best Management Practices*. Transportation Research Board. https://doi.org/10.17226/22275

Urbonas, B., MacKenzie, K., Piza, H., Clary, J., Earles, A., Olson, C., & Roesner, L. (2017). *Best Management Practices—Rational Estimation of Actual Likely Costs of Stormwater Treatment*. Mile High Flood District. https://mhfd.org/resources/

US EPA. (1999). *Prelimiary Data Summary of Urban Stormwater Best Management Practices* (United States) [Reports and Assessments]. US EPA. https://www.epa.gov/eg/industrial-wastewater-studies-miscellaneous

US EPA. (2000). *Low Impact Development (LID): A Literature Review*.

US EPA. (2007). *Reducing Stormwater Costs through Low Impact Development (LID) Strategies and Practices, December 2007, EPA 841-F-07-006*. 37.

US EPA. (2009a). *Storm Water Management Model Applications Manual*. https://www.epa.gov/water-research/storm-water-management-model-swmm

US EPA. (2009b). *Technical Guidance on Implementing the Stormwater Runoff Requirements for Federal Projects under Section 438 of the Energy Independence and Security Act*. 63.

US EPA. (2013). *Case Studies Analyzing the Economic Benefits of Low Impact Development and Green Infrastructure Programs*. 142.

US EPA. (2014). *The Economic Benefits of Green Infrastructure: A Case Study of Lancaster, PA* (Wisconsin) [Reports and Assessments]. US EPA. https://www.epa.gov/green-infrastructure/economic-benefits-green-infrastructure-case-study-lancaster-pa

US EPA. (2019). *Storm Water Management Model (SWMM) version 5.1 User's Manual*. https://www.epa.gov/water-research/storm-water-management-model-swmm-version-51-users-manual

US EPA, O. (2017, July 11). *Opti-Tool: EPA Region 1's Stormwater Management Optimization Tool* [Overviews and Factsheets]. US EPA. https://www.epa.gov/tmdl/opti-tool-epa-region-1s-stormwater-management-optimization-tool

USACE. (1998). *HEC-1 Flood Hydrograph Package User's Manual*. https://www.hec.usace.army.mil/publications/ComputerProgramDocumentation/HEC-1_UsersManual_(CPD-1a).pdf

Wanielista, M. P., & Yousef, Y. A. (1993). *Stormwater Management*. John Wiley & Sons.

Weiss, W. P., Gulliver, J. S., & Erickson, A. J. (2007). Cost and Pollutant Removal of Storm-Water Treatment Practices. *Journal of Water Resources Planning and Management*, *133*(3), 218–229. https://doi.org/10.1061/(ASCE)0733-9496(2007)133:3(218)

Yeh, C.-H., & Labadie, J. W. (1997). Multiobjective Watershed-Level Planning of Storm Water Detention Systems. *Journal of Water Resources Planning and Management*, *123*(6), 336–343. https://doi.org/10.1061/(ASCE)0733-9496(1997)123:6(336)

Zhen, X.-Y. "Jenny," Yu, S. L., & Lin, J.-Y. (2004). Optimal Location and Sizing of Stormwater Basins at Watershed Scale. *Journal of Water Resources Planning and Management*, *130*(4), 339–347. https://doi.org/10.1061/(ASCE)0733-9496(2004)130:4(339)

Appendix A: Detention Basin Optimization Model GAMS Code

```
$ TITLE Optimization Model for Design of Multipurpose Detention Basin
*
*        Optimization model created by Ross Ellis as a component of a Master's Thesis project
*        Summer 2020, Auburn University Department of Civil Engineering
*        Thesis Advisor: Frances O'Donnell, Ph.D.
*
*        Model adapted from the optimization models developed for infiltration basins
*        by M. Lacy (2016) and Stafford et al. (2015).
*
*        Problem: Optimize the size of a detention basin given required basin storage volume
*        and pre-development peak discharge from a design storm and WQv retention requirements.
*
*        This optimization model is part of a tool that uses Microsoft Excel user inputs to
*        optimize the Green Infrastructure and detention basin for a development, using SWMM
*        for the hydrologic computational engine and GAMS for the optimization computational
*        engine.
*
*        Last update: June 29, 2020
*
*-----------------------------------------------------------------------------------------------
*
SCALARS
*
*        HYDRAULIC/HYDROLOGIC PARAMETERS
         g       GRAVITATIONAL ACCELERATION (FT PER S^2)                      /32.2/
         Cd1     WQCV ORIFICE DISCHARGE COEFFICIENT                           /0.65/
*                Assumes one orifice outlet structure with no vertical offset
         Cd2     SMALL STORM ORIFICE DISCHARGE COEFFICIENT                    /0.65/
*                Assumes one orifice outlet structure with no vertical offset
         Qpre    PRE-DEVELOPMENT PEAK DISCHARGE (CU.FT. PER S)                /2.66/
*                FROM PRE-DEVELOPMENT SWMM MODEL RESULTS
*                NOTE: Value will be changed by user input before GAMS model file is generated
         WQv     WATER QUALITY CAPTURE VOLUME (CU. FT.)                       /8315/
```

```
*               FROM WQv calculations
*               NOTE: Value will be changed by user input before GAMS model file is generated
        Vreq    REQUIRED BASIN STORAGE VOLUME (CU.FT)                           /11259/
*               FROM PRE-DEVELOPMENT SWMM MODEL RESULTS
*               NOTE: Value will be changed by user input before GAMS model file is generated

*       INSTITUTIONAL PARAMETERS
        Alim    AREA LIMITATION (SQ. FT.)                                       /1000000/
*               Maximum amount of land area available for a detention basin
*               Model returns INFFEASIBLE if limitation too small
        Hlim    BASIN DEPTH LIMITATION (FT)                                     /5/
*               Per design codes for local authority
        Tlim    WQV RETENTION TIME (SEC)                                        /86400/
*               Per design codes for local authority
*
*       COST PARAMETERS
        UA      COST PER UNIT LAND AREA ($ PER SQ. FT.)                         /0.89/
        UV      COST PER UNIT EXCAVATION VOLUME ($ PER CU. FT)                  /0.19/;
*
*-----------------------------------------------------------------------------------------------
*
VARIABLES
*
        Z       COST ($);
*               OBJECTIVE FUNCTION VARIABLE
*               Objective is to minimize cost while meeting storage and flow attenuation requirements
*
*-----------------------------------------------------------------------------------------------
*
POSITIVE VARIABLES
*
        A1      WQv POND SURFACE AREA (SQ. FT)
        A2      BASIN SURFACE AREA (SQ. FT)
        h1      WQv POND DEPTH (FT)
        h2      BASIN DEPTH (FT)
        Ao1     WQv ORIFICE AREA (SQ. FT)
        Ao2     UPPER ORIFICE AREA (SQ.FT);
*
*-----------------------------------------------------------------------------------------------
*
EQUATIONS
*
*       CONSTRAINTS
        Stor1   WQv POND VOLUME GREATER THAN WQv REQUIRED STORAGE
```

```
         Stor2   BASIN VOLUME GREATER THAN REQUIRED BASIN STORAGE
         Area1   WQv POND AREA LESS THAN BASIN AREA
         Area2   BASIN AREA LESS THAN AREA LIMIT
         Hght    TOTAL BASIN DEPTH LESS THAN DEPTH LIMIT
         Out1    BASIN OUTFLOW LESS THAN PEAK PRE-DEV FLOW
         Time    WQv RETENTION TIME GREATER THAN REQUIREMENT
*
         OBJ     OBJECTIVE FUNCTION - MINIMIZE COST;
*
*        CONSTRAINTS
         Stor1.. WQv     =E=     (A1 * h1);
         Stor2.. Vreq    =E=     (A1 * h1) + (A2 * h2);
         Area1.. A2      =G=     A1;
         Area2.. Alim    =G=     A2;
         Hght..  Hlim    =G=     h1 + h2;
         Out1..  Qpre    =E=     (Cd1 * Ao1 * sqrt(2 * g * (h1 + h2))) + (Cd2 * Ao2 * sqrt(2 * g * h2));
         Time..  Tlim * (Ao1 * Cd1)    =L=     A1 * sqrt(2 / g) * sqrt(h1);
*
*        OBJECTIVE FUNCTION
         OBJ..   Z       =E=     (UA * A2) + (UV * ((A1 * h1) + (A2 * h2)));
*
*        INITIAL VALUES
         A1.l    =       1;
         A2.l    =       1;
         h1.l    =       1;
         h2.l    =       1;
         Ao1.l   =       0.1;
         Ao2.l   =       0.1;
*
*        VARIABLE BOUNDS
         A1.lo   =       0.001;
         A1.up   =       +inf;
         A2.lo   =       0.001;
         A2.up   =       +inf;
         h1.lo   =       0.001;
         h1.up   =       +inf;
         h2.lo   =       0.001;
         h2.up   =       +inf;
         Ao1.lo  =       0.001;
         Ao1.up  =       +inf;
         Ao2.lo  =       0.001;
         Ao2.up  =       +inf;
*
*---------------------------------------------------------------------------------------------
*
MODEL MULTIPURPOSE_BASIN /ALL/;
*
*---------------------------------------------------------------------------------------------
*
OPTION NLP = MSNLP;
*
SOLVE MULTIPURPOSE_BASIN USING NLP MINIMIZING Z;
*
*---------------------------------------------------------------------------------------------
*
DISPLAY Z.l, A1.l, h1.l, A2.l, h2.l, Ao1.l, Ao2.l;
```

Appendix B: Case Study GAMS Output

```
GAMS 29.1.0  rbb4180b Released Nov 15, 2019 WEX-WEI x86 64bit/MS Windows 07/01/20 02:03:54 Page 1
Optimization Model for Design of Multipurpose Detention Basin
C o m p i l a t i o n


*
*        Optimization model created by Ross Ellis as a component of a Mast
er's Thesis project
*        Summer 2020, Auburn University Department of Civil Engineering
*        Thesis Advisor: Frances O'Donnell, Ph.D.
*
*        Model adapted from the optimization models developed for infilitr
ation basins
*        by M. Lacy (2016) and Stafford et al. (2015).
*
*        Problem: Optimize the size of a detention basin given required ba
sin storage volume
*        and pre-development peak discharge from a design storm and WQv re
tention requirements.
*
*        This optimization model is part of a tool that uses Microsoft Exc
el user inputs to
*        optimize the Green Infrastructure and detention basin for a devel
opment, using SWMM
*        for the hydrologic computational engine and GAMS for the optimiza
tion computational
*        engine.
*
*        Last update: June 29, 2020
*
*------------------------------------------------------------------------
----------------------
*
SCALARS
*
*        HYDRAULIC/HYDROLOGIC PARAMETERS
         g       GRAVITATIONAL ACCELERATION (FT PER S^2)
       /32.2/
         Cd1     WQCV ORIFICE DISCHARGE COEFFICIENT
       /0.65/
*                Assumes one orifice outlet structure with no vertical off
set
         Cd2     SMALL STORM ORIFICE DISCHARGE COEFFICIENT
       /0.65/
*                Assumes one orifice outlet structure with no vertical off
set
         Qpre    PRE-DEVELOPMENT PEAK DISCHARGE (CU.FT. PER S)
       /2.66/
*                FROM PRE-DEVELOPMENT SWMM MODEL RESULTS
*                NOTE: Value will be changed by user input before GAMS mod
el file is generated
         WQv     WATER QUALITY CAPTURE VOLUME (CU. FT.)
       /8315/
*                FROM WQv calculations
*                NOTE: Value will be changed by user input before GAMS mod
el file is generated
         Vreq    REQUIRED BASIN STORAGE VOLUME (CU.FT)
       /11259/
*                FROM PRE-DEVELOPMENT SWMM MODEL RESULTS
*                NOTE: Value will be changed by user input before GAMS mod
el file is generated

*        INSTITUTIONAL PARAMETERS
         Alim    AREA LIMITATION (SQ. FT.)
       /1000000/
*                Maximum amount of land area available for a detention bas
```

```
in
*                Model returns INFFEASIBLE if limitation too small
         Hlim    BASIN DEPTH LIMITATION (FT)
       /5/
*                Per design codes for local authority
         Tlim    WQV RETENTION TIME (SEC)
       /86400/
*                Per design codes for local authority
*
*        COST PARAMETERS
         UA      COST PER UNIT LAND AREA ($ PER SQ. FT.)
       /0.89/
         UV      COST PER UNIT EXCAVATION VOLUME ($ PER CU. FT)
       /0.19/;
*
*---------------------------------------------------------------------
---------------------
*
VARIABLES
*
         Z       COST ($);
*                OBJECTIVE FUNCTION VARIABLE
*                Objective is to minimize cost while meeting storage and f
low attenuation requirements
*
*---------------------------------------------------------------------
---------------------
*
POSITIVE VARIABLES
*
         A1      WQv POND SURFACE AREA (SQ. FT)
         A2      BASIN SURFACE AREA (SQ. FT)
         h1      WQv POND DEPTH (FT)
         h2      BASIN DEPTH (FT)
         Ao1     WQv ORIFICE AREA (SQ. FT)
         Ao2     UPPER ORIFICE AREA (SQ.FT);
*
*---------------------------------------------------------------------
---------------------
*
EQUATIONS
*
*        CONSTRAINTS
         Stor1   WQv POND VOLUME GREATER THAN WQv REQUIRED STORAGE
         Stor2   BASIN VOLUME GREATER THAN REQUIRED BASIN STORAGE
         Area1   WQv POND AREA LESS THAN BASIN AREA
         Area2   BASIN AREA LESS THAN AREA LIMIT
         Hght    TOTAL BASIN DEPTH LESS THAN DEPTH LIMIT
         Out1    BASIN OUTFLOW LESS THAN PEAK PRE-DEV FLOW
         Time    WQv RETENTION TIME GREATER THAN REQUIREMENT
*
         OBJ     OBJECTIVE FUNCTION - MINIMIZE COST;
*
*        CONSTRAINTS
         Stor1.. WQv     =e=     (A1 * h1);
         Stor2.. Vreq    =e=     (A1 * h1) + (A2 * h2);
         Area1.. A2      =G=     A1;
         Area2.. Alim    =G=     A2;
         Hght..  Hlim    =G=     h1 + h2;
         Out1..  Qpre    =E=     (Cd1 * Ao1 * sqrt(2 * g * (h1 + h2))) + (
Cd2 * Ao2 * sqrt(2 * g * h2));
         Time..  Tlim * (Ao1 * Cd1)    =L=     A1 * sqrt(2 / g) * sqrt(h1)
;
*
*        OBJECTIVE FUNCTION
         OBJ..   Z       =E=     (UA * A2) + (UV * ((A1 * h1) + (A2 * h2))
);
*
*        INITIAL VALUES
```

```
 100          A1.l    =       1;
 101          A2.l    =       1;
 102          h1.l    =       1;
 103          h2.l    =       1;
 104          Ao1.l   =       0.1;
 105          Ao2.l   =       0.1;
 106  *
 107  *       VARIABLE BOUNDS
 108          A1.lo   =       0.001;
 109          A1.up   =       +inf;
 110          A2.lo   =       0.001;
 111          A2.up   =       +inf;
 112          h1.lo   =       0.001;
 113          h1.up   =       +inf;
 114          h2.lo   =       0.001;
 115          h2.up   =       +inf;
 116          Ao1.lo  =       0.001;
 117          Ao1.up  =       +inf;
 118          Ao2.lo  =       0.001;
 119          Ao2.up  =       +inf;
 120  *
 121  *-------------------------------------------------------------------------
      --------------------
 122  *
 123  MODEL MULTIPURPOSE_BASIN /ALL/;
 124  *
 125  *-------------------------------------------------------------------------
      --------------------
 126  *
 127  OPTION NLP = MSNLP;
 128  *
 129  SOLVE MULTIPURPOSE_BASIN USING NLP MINIMIZING Z;
 130  *
 131  *-------------------------------------------------------------------------
      --------------------
 132  *
 133  DISPLAY Z.l, A1.l, h1.l, A2.l, h2.l, Ao1.l, Ao2.l;


COMPILATION TIME     =        0.000 SECONDS      2 MB  29.1.0 rbb4180b WEX-WEI
GAMS 29.1.0  rbb4180b Released Nov 15, 2019 WEX-WEI x86 64bit/MS Windows 07/01/20 02:03:54 Page 2
Optimization Model for Design of Multipurpose Detention Basin
Equation Listing    SOLVE MULTIPURPOSE_BASIN Using NLP From line 129


---- Stor1  =E=  WQv POND VOLUME GREATER THAN WQv REQUIRED STORAGE

Stor1..  - (1)*A1 - (1)*h1 =E= -8315 ; (LHS = -1, INFES = 8314 ****)


---- Stor2  =E=  BASIN VOLUME GREATER THAN REQUIRED BASIN STORAGE

Stor2..  - (1)*A1 - (1)*A2 - (1)*h1 - (1)*h2 =E= -11259 ;

      (LHS = -2, INFES = 11257 ****)


---- Area1  =G=  WQv POND AREA LESS THAN BASIN AREA

Area1..  - A1 + A2 =G= 0 ; (LHS = 0)


---- Area2  =G=  BASIN AREA LESS THAN AREA LIMIT

Area2..  - A2 =G= -1000000 ; (LHS = -1)


---- Hght  =G=  TOTAL BASIN DEPTH LESS THAN DEPTH LIMIT
```

```
Hght..  - h1 - h2 =G= -5 ; (LHS = -2)


---- Out1  =E=  BASIN OUTFLOW LESS THAN PEAK PRE-DEV FLOW

Out1..  - (0.184421392468444)*h1 - (0.44523262688905)*h2

      - (7.37685569873778)*Ao1 - (5.21622468841211)*Ao2 =E= -2.66 ;

      (LHS = -1.25930803871499, INFES = 1.40069196128501 ****)


---- Time  =L=  WQv RETENTION TIME GREATER THAN REQUIREMENT

Time..  - (0.249222393139613)*A1 - (0.124611196569807)*h1 + 56160*Ao1 =L= 0 ;

      (LHS = 5615.75077760686, INFES = 5615.75077760686 ****)


---- OBJ  =E=  OBJECTIVE FUNCTION - MINIMIZE COST

OBJ..  Z - (0.19)*A1 - (1.08)*A2 - (0.19)*h1 - (0.19)*h2 =E= 0 ;

      (LHS = -1.27, INFES = 1.27 ****)
```



```
---- Z  COST ($)

Z
                (.LO, .L, .UP, .M = -INF, 0, +INF, 0)
        1       OBJ


---- A1  WQv POND SURFACE AREA (SQ. FT)

A1
                (.LO, .L, .UP, .M = 0.001, 1, +INF, 0)
      (-1)      Stor1
      (-1)      Stor2
       -1       Area1
      (-0.2492) Time
      (-0.19)   OBJ


---- A2  BASIN SURFACE AREA (SQ. FT)

A2
                (.LO, .L, .UP, .M = 0.001, 1, +INF, 0)
      (-1)      Stor2
        1       Area1
       -1       Area2
      (-1.08)   OBJ


---- h1  WQv POND DEPTH (FT)

h1
                (.LO, .L, .UP, .M = 0.001, 1, +INF, 0)
      (-1)      Stor1
      (-1)      Stor2
       -1       Hght
      (-0.1844) Out1
      (-0.1246) Time
      (-0.19)   OBJ
```

```
---- h2  BASIN DEPTH (FT)

h2
                (.LO, .L, .UP, .M = 0.001, 1, +INF, 0)
      (-1)      Stor2
       -1       Hght
      (-0.4452) Out1
      (-0.19)   OBJ


---- Ao1  WQv ORIFICE AREA (SQ. FT)

Ao1
                (.LO, .L, .UP, .M = 0.001, 0.1, +INF, 0)
      (-7.3769) Out1
    56160       Time


---- Ao2  UPPER ORIFICE AREA (SQ.FT)

Ao2
                (.LO, .L, .UP, .M = 0.001, 0.1, +INF, 0)
      (-5.2162) Out1
```

```
MODEL STATISTICS

BLOCKS OF EQUATIONS           8     SINGLE EQUATIONS            8
BLOCKS OF VARIABLES           7     SINGLE VARIABLES            7
NON ZERO ELEMENTS            23     NON LINEAR N-Z             16
DERIVATIVE POOL              20     CONSTANT POOL              21
CODE LENGTH                  51


GENERATION TIME      =        0.031 SECONDS      3 MB  29.1.0 rbb4180b WEX-WEI


EXECUTION TIME       =        0.031 SECONDS      3 MB  29.1.0 rbb4180b WEX-WEI
```

```
               S O L V E      S U M M A R Y

     MODEL   MULTIPURPOSE_BASIN   OBJECTIVE  Z
     TYPE    NLP                  DIRECTION  MINIMIZE
     SOLVER  MSNLP                FROM LINE  129

**** SOLVER STATUS     1 Normal Completion
**** MODEL STATUS      2 Locally Optimal
**** OBJECTIVE VALUE             4143.3120

 RESOURCE USAGE, LIMIT          0.625      1000.000
 ITERATION COUNT, LIMIT         0    2000000000
 EVALUATION ERRORS              0             0

 MsNlp Terminated When Iteration Count Exceeded Limit of 1000



                       LOWER     LEVEL     UPPER    MARGINAL

---- EQU Stor1      -8315.000 -8315.000 -8315.000      .
---- EQU Stor2      -1.126E+4 -1.126E+4 -1.126E+4    -0.368
---- EQU Area1           .         .        +INF      0.657
---- EQU Area2      -1.000E+6 -2251.800     +INF       .
---- EQU Hght          -5.000    -5.000     +INF    400.820
---- EQU Out1          -2.660    -2.660    -2.660      .
---- EQU Time           -INF       .         .        EPS
---- EQU OBJ             .         .         .       1.000

  Stor1  WQv POND VOLUME GREATER THAN WQv REQUIRED STORAGE
  Stor2  BASIN VOLUME GREATER THAN REQUIRED BASIN STORAGE
  Area1  WQv POND AREA LESS THAN BASIN AREA
  Area2  BASIN AREA LESS THAN AREA LIMIT
  Hght  TOTAL BASIN DEPTH LESS THAN DEPTH LIMIT
  Out1  BASIN OUTFLOW LESS THAN PEAK PRE-DEV FLOW
  Time  WQv RETENTION TIME GREATER THAN REQUIREMENT
  OBJ  OBJECTIVE FUNCTION - MINIMIZE COST

                       LOWER     LEVEL     UPPER    MARGINAL

---- VAR Z              -INF   4143.312     +INF       .
---- VAR A1             0.001  2251.800     +INF       .
---- VAR A2             0.001  2251.800     +INF       .
---- VAR h1             0.001     3.693     +INF       .
---- VAR h2             0.001     1.307     +INF       .
---- VAR Ao1            0.001     0.019     +INF       .
---- VAR Ao2            0.001     0.408     +INF       .

  Z  COST ($)
  A1  WQv POND SURFACE AREA (SQ. FT)
  A2  BASIN SURFACE AREA (SQ. FT)
  h1  WQv POND DEPTH (FT)
  h2  BASIN DEPTH (FT)
  Ao1  WQv ORIFICE AREA (SQ. FT)
  Ao2  UPPER ORIFICE AREA (SQ.FT)


**** REPORT SUMMARY :        0     NONOPT
                             0 INFEASIBLE
                             0  UNBOUNDED
                             0     ERRORS
```

```
----    133 VARIABLE Z.L                  =     4143.312  COST ($)
            VARIABLE A1.L                 =     2251.800  WQv POND SURFACE AREA
                                                          (SQ. FT)
            VARIABLE h1.L                 =        3.693  WQv POND DEPTH (FT)
            VARIABLE A2.L                 =     2251.800  BASIN SURFACE AREA (S
                                                          Q. FT)
            VARIABLE h2.L                 =        1.307  BASIN DEPTH (FT)
            VARIABLE Ao1.L                =        0.019  WQv ORIFICE AREA (SQ.
                                                          FT)
            VARIABLE Ao2.L                =        0.408  UPPER ORIFICE AREA (S
                                                          Q.FT)


EXECUTION TIME       =        0.000 SECONDS      3 MB  29.1.0 rbb4180b WEX-WEI


USER: GAMS Development Corporation, USA              G871201/0000CA-ANY
      Free Demo, +1 202-342-0180, support@gams.com, www.gams.com DC0000


**** FILE SUMMARY

Input      C:\Users\Ross\Documents\gamsdir\projdir\Untitled_2.gms
Output     C:\Users\Ross\Documents\gamsdir\projdir\Untitled_2.lst
```

Appendix C: Case Study Pre-development SWMM Input File

```
[TITLE]
;;Project Title/Notes
GI Tool
Pre-Development Runoff

[OPTIONS]
;;Option  Value
FLOW_UNITS  CFS
INFILTRATION  CURVE_NUMBER
FLOW_ROUTING  KINWAVE
LINK_OFFSETS  DEPTH
MIN_SLOPE  0
ALLOW_PONDING  NO
SKIP_STEADY_STATE  NO

START_DATE  1/1/2007
START_TIME  0:00:00
REPORT_START_DATE  1/1/2007
REPORT_START_TIME  0:00:00
END_DATE  1/3/2007
END_TIME  0:00:00
SWEEP_START  1/1
SWEEP_END  12/31
DRY_DAYS  0
REPORT_STEP  0:05:00
WET_STEP  0:05:00
DRY_STEP  1:00:00
ROUTING_STEP  0:05:00
RULE_STEP  0:00:00

INERTIAL_DAMPING  PARTIAL
NORMAL_FLOW_LIMITED  SLOPE
FORCE_MAIN_EQUATION  H-W
VARIABLE_STEP  0.75
LENGTHENING_STEP  0
MIN_SURFAREA  12.566
MAX_TRIALS  8
HEAD_TOLERANCE  0.005
SYS_FLOW_TOL  5
LAT_FLOW_TOL  5
MINIMUM_STEP  0.5
THREADS  1

[EVAPORATION]
;;Data Source  Parameters
;;--------------  ----------------
CONSTANT  0
DRY_ONLY  NO

[RAINGAGES]
;;Name  Format  Interval  SCF  Source
;;--------------  ---------  ------  ------  ----------
RainGage  INTENSITY  0:05  1  TIMESERIES  Synth_24hr

[SUBCATCHMENTS]
;;Name  Rain Gage  Outlet  Area  %Imperv  Width  %Slope  CurbLen  SnowPack
;;--------------  ----------------  ----------------  --------  --------  --------  --------  --------  --
--------------
S1  RainGage  J1  5.62  0  489.614  5  0
S2  RainGage  J1  0  0  0  0  0
S3  RainGage  J1  0  0  0  0  0
S4  RainGage  J1  0  0  0  0  0
S5  RainGage  J1  0  0  0  0  0
S6  RainGage  J1  0  0  0  0  0
S7  RainGage  J1  0  0  0  0  0
S8  RainGage  J1  0  0  0  0  0
```

```
S9  RainGage  J1  0  0  0  0  0
S10  RainGage  J1  0  0  0  0  0


[SUBAREAS]
;;Subcatchment  N-Imperv  N-Perv  S-Imperv  S-Perv  PctZero  RouteTo  PctRouted
;;--------------  ----------  ----------  ----------  ----------  ----------  ----------  ----------
S1  0.015  0.24  0.06  0.3  25  OUTLET
S2  0.015  0.24  0.06  0.3  25  OUTLET
S3  0.015  0.24  0.06  0.3  25  OUTLET
S4  0.015  0.24  0.06  0.3  25  OUTLET
S5  0.015  0.24  0.06  0.3  25  OUTLET
S6  0.015  0.24  0.06  0.3  25  OUTLET
S7  0.015  0.24  0.06  0.3  25  OUTLET
S8  0.015  0.24  0.06  0.3  25  OUTLET
S9  0.015  0.24  0.06  0.3  25  OUTLET
S10  0.015  0.24  0.06  0.3  25  OUTLET


[INFILTRATION]
;;Subcatchment  CurveNum    DryTime
;;--------------  ----------  ----------  ----------
S1  60  0.2  6.5
S2  0  0.2  6.5
S3  0  0.2  6.5
S4  0  0.2  6.5
S5  0  0.2  6.5
S6  0  0.2  6.5
S7  0  0.2  6.5
S8  0  0.2  6.5
S9  0  0.2  6.5
S10  0  0.2  6.5


[JUNCTIONS]
;;Name  Elevation  MaxDepth  InitDepth  SurDepth  Aponded
;;--------------  ----------  ----------  ----------  ----------  ----------
J1  0  0  0  0  0

[OUTFALLS]
;;Name  Elevation  Type  Stage Data  Gated  Route To
;;--------------  ----------  ----------  ----------------  --------  ----------------
O1  0  FREE    NO

[CONDUITS]
;;Name  From Node  To Node  Length  Roughness  InOffset  OutOffset  InitFlow  MaxFlow
;;--------------  ----------------  ----------------  ----------  ----------  ----------  ----------  ----
------  ----------
C1  J1  O1  66.28  0.001  0  0  0  0

[XSECTIONS]
;;Link  Shape  Geom1  Geom2  Geom3  Geom4  Barrels  Culvert
;;--------------  ------------  ----------------  ----------  ----------  ----------  ----------  --------
--
C1  DUMMY  0  0  0  0  1

[TIMESERIES]
;;Name  Date  Time  Value
;;--------------  ----------  ----------  ----------
;Adapted form Applciations manual hyetograph
Synth_2hr    0:00  1.131
Synth_2hr    0:05  1.289
Synth_2hr    0:10  1.504
Synth_2hr    0:15  2.522
Synth_2hr    0:20  3.212
Synth_2hr    0:25  6.209
Synth_2hr    0:30  11.254
Synth_2hr    0:35  4.660
Synth_2hr    0:40  2.805
```

```
Synth_2hr    0:45  1.651
Synth_2hr    0:50  1.380
Synth_2hr    0:55  1.199
Synth_2hr    1:00  1.131
Synth_2hr    1:05  1.074
Synth_2hr    1:10  1.029
Synth_2hr    1:15  0.984
Synth_2hr    1:20  0.950
Synth_2hr    1:25  0.916
Synth_2hr    1:30  0.882
Synth_2hr    1:35  0.848
Synth_2hr    1:40  0.826
Synth_2hr    1:45  0.803
Synth_2hr    1:50  0.780
Synth_2hr    1:55  0.758
;
;Description
Synth_24hr    0:00  0.042
Synth_24hr    0:05  0.042
Synth_24hr    0:10  0.042
Synth_24hr    0:15  0.042
Synth_24hr    0:20  0.042
Synth_24hr    0:25  0.042
Synth_24hr    0:30  0.042
Synth_24hr    0:35  0.042
Synth_24hr    0:40  0.042
Synth_24hr    0:45  0.041
Synth_24hr    0:50  0.042
Synth_24hr    0:55  0.042
Synth_24hr    1:00  0.042
Synth_24hr    1:05  0.042
Synth_24hr    1:10  0.042
Synth_24hr    1:15  0.042
Synth_24hr    1:20  0.042
Synth_24hr    1:25  0.041
Synth_24hr    1:30  0.041
Synth_24hr    1:35  0.041
Synth_24hr    1:40  0.041
Synth_24hr    1:45  0.041
Synth_24hr    1:50  0.041
Synth_24hr    1:55  0.042
Synth_24hr    2:00  0.043
Synth_24hr    2:05  0.043
Synth_24hr    2:10  0.043
Synth_24hr    2:15  0.043
Synth_24hr    2:20  0.043
Synth_24hr    2:25  0.043
Synth_24hr    2:30  0.046
Synth_24hr    2:35  0.046
Synth_24hr    2:40  0.046
Synth_24hr    2:45  0.046
Synth_24hr    2:50  0.046
Synth_24hr    2:55  0.046
Synth_24hr    3:00  0.049
Synth_24hr    3:05  0.049
Synth_24hr    3:10  0.049
Synth_24hr    3:15  0.049
Synth_24hr    3:20  0.049
Synth_24hr    3:25  0.049
Synth_24hr    3:30  0.052
Synth_24hr    3:35  0.052
Synth_24hr    3:40  0.052
Synth_24hr    3:45  0.052
Synth_24hr    3:50  0.052
Synth_24hr    3:55  0.052
Synth_24hr    4:00  0.056
Synth_24hr    4:05  0.056
Synth_24hr    4:10  0.056
Synth_24hr    4:15  0.056
```

Synth_24hr	4:20	0.056
Synth_24hr	4:25	0.056
Synth_24hr	4:30	0.059
Synth_24hr	4:35	0.059
Synth_24hr	4:40	0.059
Synth_24hr	4:45	0.059
Synth_24hr	4:50	0.059
Synth_24hr	4:55	0.059
Synth_24hr	5:00	0.062
Synth_24hr	5:05	0.062
Synth_24hr	5:10	0.062
Synth_24hr	5:15	0.062
Synth_24hr	5:20	0.062
Synth_24hr	5:25	0.062
Synth_24hr	5:30	0.065
Synth_24hr	5:35	0.065
Synth_24hr	5:40	0.065
Synth_24hr	5:45	0.065
Synth_24hr	5:50	0.065
Synth_24hr	5:55	0.065
Synth_24hr	6:00	0.072
Synth_24hr	6:05	0.072
Synth_24hr	6:10	0.072
Synth_24hr	6:15	0.072
Synth_24hr	6:20	0.072
Synth_24hr	6:25	0.072
Synth_24hr	6:30	0.082
Synth_24hr	6:35	0.082
Synth_24hr	6:40	0.082
Synth_24hr	6:45	0.082
Synth_24hr	6:50	0.082
Synth_24hr	6:55	0.082
Synth_24hr	7:00	0.092
Synth_24hr	7:05	0.092
Synth_24hr	7:10	0.092
Synth_24hr	7:15	0.092
Synth_24hr	7:20	0.092
Synth_24hr	7:25	0.092
Synth_24hr	7:30	0.103
Synth_24hr	7:35	0.103
Synth_24hr	7:40	0.103
Synth_24hr	7:45	0.103
Synth_24hr	7:50	0.103
Synth_24hr	7:55	0.103
Synth_24hr	8:00	0.120
Synth_24hr	8:05	0.120
Synth_24hr	8:10	0.120
Synth_24hr	8:15	0.120
Synth_24hr	8:20	0.120
Synth_24hr	8:25	0.120
Synth_24hr	8:30	0.144
Synth_24hr	8:35	0.144
Synth_24hr	8:40	0.144
Synth_24hr	8:45	0.144
Synth_24hr	8:50	0.144
Synth_24hr	8:55	0.144
Synth_24hr	9:00	0.168
Synth_24hr	9:05	0.168
Synth_24hr	9:10	0.168
Synth_24hr	9:15	0.168
Synth_24hr	9:20	0.168
Synth_24hr	9:25	0.168
Synth_24hr	9:30	0.191
Synth_24hr	9:35	0.191
Synth_24hr	9:40	0.191
Synth_24hr	9:45	0.191
Synth_24hr	9:50	0.191
Synth_24hr	9:55	0.191
Synth_24hr	10:00	0.228

Synth_24hr	10:05	0.228
Synth_24hr	10:10	0.228
Synth_24hr	10:15	0.228
Synth_24hr	10:20	0.228
Synth_24hr	10:25	0.228
Synth_24hr	10:30	0.278
Synth_24hr	10:35	0.278
Synth_24hr	10:40	0.278
Synth_24hr	10:45	0.278
Synth_24hr	10:50	0.278
Synth_24hr	10:55	0.278
Synth_24hr	11:00	0.398
Synth_24hr	11:05	0.398
Synth_24hr	11:10	0.398
Synth_24hr	11:15	0.398
Synth_24hr	11:20	0.398
Synth_24hr	11:25	0.398
Synth_24hr	11:30	1.677
Synth_24hr	11:35	1.677
Synth_24hr	11:40	1.677
Synth_24hr	11:45	1.677
Synth_24hr	11:50	1.677
Synth_24hr	11:55	1.677
Synth_24hr	12:00	1.677
Synth_24hr	12:05	1.677
Synth_24hr	12:10	1.677
Synth_24hr	12:15	1.677
Synth_24hr	12:20	1.677
Synth_24hr	12:25	1.677
Synth_24hr	12:30	0.398
Synth_24hr	12:35	0.398
Synth_24hr	12:40	0.398
Synth_24hr	12:45	0.398
Synth_24hr	12:50	0.398
Synth_24hr	12:55	0.398
Synth_24hr	13:00	0.278
Synth_24hr	13:05	0.278
Synth_24hr	13:10	0.278
Synth_24hr	13:15	0.278
Synth_24hr	13:20	0.278
Synth_24hr	13:25	0.278
Synth_24hr	13:30	0.228
Synth_24hr	13:35	0.228
Synth_24hr	13:40	0.228
Synth_24hr	13:45	0.228
Synth_24hr	13:50	0.228
Synth_24hr	13:55	0.228
Synth_24hr	14:00	0.191
Synth_24hr	14:05	0.191
Synth_24hr	14:10	0.191
Synth_24hr	14:15	0.191
Synth_24hr	14:20	0.191
Synth_24hr	14:25	0.191
Synth_24hr	14:30	0.168
Synth_24hr	14:35	0.168
Synth_24hr	14:40	0.168
Synth_24hr	14:45	0.168
Synth_24hr	14:50	0.168
Synth_24hr	14:55	0.168
Synth_24hr	15:00	0.144
Synth_24hr	15:05	0.144
Synth_24hr	15:10	0.144
Synth_24hr	15:15	0.144
Synth_24hr	15:20	0.144
Synth_24hr	15:25	0.144
Synth_24hr	15:30	0.120
Synth_24hr	15:35	0.120
Synth_24hr	15:40	0.120
Synth_24hr	15:45	0.120

Synth_24hr	15:50	0.120
Synth_24hr	15:55	0.120
Synth_24hr	16:00	0.103
Synth_24hr	16:05	0.103
Synth_24hr	16:10	0.103
Synth_24hr	16:15	0.103
Synth_24hr	16:20	0.103
Synth_24hr	16:25	0.103
Synth_24hr	16:30	0.092
Synth_24hr	16:35	0.092
Synth_24hr	16:40	0.092
Synth_24hr	16:45	0.092
Synth_24hr	16:50	0.092
Synth_24hr	16:55	0.092
Synth_24hr	17:00	0.082
Synth_24hr	17:05	0.082
Synth_24hr	17:10	0.082
Synth_24hr	17:15	0.082
Synth_24hr	17:20	0.082
Synth_24hr	17:25	0.082
Synth_24hr	17:30	0.072
Synth_24hr	17:35	0.072
Synth_24hr	17:40	0.072
Synth_24hr	17:45	0.072
Synth_24hr	17:50	0.072
Synth_24hr	17:55	0.072
Synth_24hr	18:00	0.065
Synth_24hr	18:05	0.065
Synth_24hr	18:10	0.065
Synth_24hr	18:15	0.065
Synth_24hr	18:20	0.065
Synth_24hr	18:25	0.065
Synth_24hr	18:30	0.062
Synth_24hr	18:35	0.062
Synth_24hr	18:40	0.062
Synth_24hr	18:45	0.062
Synth_24hr	18:50	0.062
Synth_24hr	18:55	0.062
Synth_24hr	19:00	0.059
Synth_24hr	19:05	0.059
Synth_24hr	19:10	0.059
Synth_24hr	19:15	0.059
Synth_24hr	19:20	0.059
Synth_24hr	19:25	0.059
Synth_24hr	19:30	0.056
Synth_24hr	19:35	0.056
Synth_24hr	19:40	0.056
Synth_24hr	19:45	0.056
Synth_24hr	19:50	0.056
Synth_24hr	19:55	0.056
Synth_24hr	20:00	0.053
Synth_24hr	20:05	0.053
Synth_24hr	20:10	0.053
Synth_24hr	20:15	0.053
Synth_24hr	20:20	0.053
Synth_24hr	20:25	0.053
Synth_24hr	20:30	0.050
Synth_24hr	20:35	0.050
Synth_24hr	20:40	0.050
Synth_24hr	20:45	0.050
Synth_24hr	20:50	0.050
Synth_24hr	20:55	0.050
Synth_24hr	21:00	0.048
Synth_24hr	21:05	0.048
Synth_24hr	21:10	0.048
Synth_24hr	21:15	0.048
Synth_24hr	21:20	0.048
Synth_24hr	21:25	0.048
Synth_24hr	21:30	0.046

```
Synth_24hr    21:35  0.046
Synth_24hr    21:40  0.046
Synth_24hr    21:45  0.046
Synth_24hr    21:50  0.046
Synth_24hr    21:55  0.046
Synth_24hr    22:00  0.043
Synth_24hr    22:05  0.043
Synth_24hr    22:10  0.043
Synth_24hr    22:15  0.043
Synth_24hr    22:20  0.043
Synth_24hr    22:25  0.043
Synth_24hr    22:30  0.041
Synth_24hr    22:35  0.041
Synth_24hr    22:40  0.041
Synth_24hr    22:45  0.041
Synth_24hr    22:50  0.041
Synth_24hr    22:55  0.041
Synth_24hr    23:00  0.039
Synth_24hr    23:05  0.039
Synth_24hr    23:10  0.039
Synth_24hr    23:15  0.039
Synth_24hr    23:20  0.039
Synth_24hr    23:25  0.039
Synth_24hr    23:30  0.036
Synth_24hr    23:35  0.036
Synth_24hr    23:40  0.036
Synth_24hr    23:45  0.036
Synth_24hr    23:50  0.036
Synth_24hr    23:55  0.036

[REPORT]
;;Reporting Options
SUBCATCHMENTS  ALL
NODES  ALL
LINKS  ALL

[TAGS]

[MAP]
DIMENSIONS  -100  -100  600  500
Units  Feet

[COORDINATES]
;;Node  X-Coord  Y-Coord
;;--------------  ------------------  ------------------
J1  500  300
O1  500  400

[VERTICES]
;;Link  X-Coord  Y-Coord
;;--------------  ------------------  ------------------

[Polygons]
;;Subcatchment  X-Coord  Y-Coord
;;--------------  ------------------  ------------------
S1  0  0
S1  100  0
S1  100  100
S1  0  100
S2  100  0
S2  200  0
S2  200  100
S2  100  100
S3  200  0
S3  300  0
S3  300  100
S3  200  100
S4  300  0
S4  400  0
```

```
S4  400  100
S4  300  100
S5  400  0
S5  500  0
S5  500  100
S5  400  100
S6  0  100
S6  100  100
S6  100  200
S6  0  200
S7  100  100
S7  200  100
S7  200  200
S7  100  200
S8  200  100
S8  300  100
S8  300  200
S8  200  200
S9  300  100
S9  400  100
S9  400  200
S9  300  200
S10  400  100
S10  500  100
S10  500  200
S10  400  200


[SYMBOLS]
;;Gage  X-Coord  Y-Coord
;;--------------  ------------------  ------------------
RainGage  0  300
```

Appendix D: Case Study Post-development SWMM Input File

```
[TITLE]
;;Project Title/Notes
GI Tool
Post-Development Runoff

[OPTIONS]
;;Option  Value
FLOW_UNITS  CFS
INFILTRATION  CURVE_NUMBER
FLOW_ROUTING  KINWAVE
LINK_OFFSETS  DEPTH
MIN_SLOPE  0
ALLOW_PONDING  NO
SKIP_STEADY_STATE  NO

START_DATE  1/1/2007
START_TIME  0:00:00
REPORT_START_DATE  1/1/2007
REPORT_START_TIME  0:00:00
END_DATE  1/3/2007
END_TIME  0:00:00
SWEEP_START  1/1
SWEEP_END  12/31
DRY_DAYS  0
REPORT_STEP  0:05:00
WET_STEP  0:05:00
DRY_STEP  1:00:00
ROUTING_STEP  0:05:00
RULE_STEP  0:00:00

INERTIAL_DAMPING  PARTIAL
NORMAL_FLOW_LIMITED  SLOPE
FORCE_MAIN_EQUATION  H-W
VARIABLE_STEP  0.75
LENGTHENING_STEP  0
MIN_SURFAREA  12.566
MAX_TRIALS  8
HEAD_TOLERANCE  0.005
SYS_FLOW_TOL  5
LAT_FLOW_TOL  5
MINIMUM_STEP  0.5
THREADS  1

[EVAPORATION]
;;Data Source  Parameters
;;--------------  ----------------
CONSTANT  0
DRY_ONLY  NO

[RAINGAGES]
;;Name  Format  Interval  SCF  Source
;;--------------  ---------  ------  ------  ----------
RainGage  INTENSITY  0:05  1  TIMESERIES  Synth_24hr

[SUBCATCHMENTS]
;;Name  Rain Gage  Outlet  Area  %Imperv  Width  %Slope  CurbLen  SnowPack
;;--------------  ----------------  ----------------  --------  --------  --------  --------  --------  --
--------------
S1  RainGage  J1  0.8  0  132  0.5  0
S2  RainGage  J1  1.4  0  174.62  3  0
S3  RainGage  J1  1.1  0  154.784  1.5  0
S4  RainGage  J1  0  0  0  0  0
S5  RainGage  J1  0  0  0  0  0
S6  RainGage  J1  0  0  0  0  0
S7  RainGage  J1  0  0  0  0  0
S8  RainGage  J1  0  0  0  0  0
S9  RainGage  J1  0  0  0  0  0
```

```
S10  RainGage  J1  0  0  0  0  0
S11  RainGage  J1  1.16  0  158.949  2  0
S12  RainGage  J1  1.16  0  158.949  3  0
S13  RainGage  J1  0  0  0  0  0
S14  RainGage  J1  0  0  0  0  0
S15  RainGage  J1  0  0  0  0  0
S16  RainGage  J1  0  0  0  0  0
S17  RainGage  J1  0  0  0  0  0
S18  RainGage  J1  0  0  0  0  0
S19  RainGage  J1  0  0  0  0  0
S20  RainGage  J1  0  0  0  0  0


[SUBAREAS]
;;Subcatchment  N-Imperv  N-Perv  S-Imperv  S-Perv  PctZero  RouteTo  PctRouted
;;--------------  ----------  ----------  ----------  ----------  ----------  ----------  ----------
S1  0.015  0.24  0.06  0.3  25  OUTLET
S2  0.015  0.24  0.06  0.3  25  OUTLET
S3  0.015  0.24  0.06  0.3  25  OUTLET
S4  0.015  0.24  0.06  0.3  25  OUTLET
S5  0.015  0.24  0.06  0.3  25  OUTLET
S6  0.015  0.24  0.06  0.3  25  OUTLET
S7  0.015  0.24  0.06  0.3  25  OUTLET
S8  0.015  0.24  0.06  0.3  25  OUTLET
S9  0.015  0.24  0.06  0.3  25  OUTLET
S10  0.015  0.24  0.06  0.3  25  OUTLET
S11  0.015  0.24  0.06  0.3  25  OUTLET
S12  0.015  0.24  0.06  0.3  25  OUTLET
S13  0.015  0.24  0.06  0.3  25  OUTLET
S14  0.015  0.24  0.06  0.3  25  OUTLET
S15  0.015  0.24  0.06  0.3  25  OUTLET
S16  0.015  0.24  0.06  0.3  25  OUTLET
S17  0.015  0.24  0.06  0.3  25  OUTLET
S18  0.015  0.24  0.06  0.3  25  OUTLET
S19  0.015  0.24  0.06  0.3  25  OUTLET
S20  0.015  0.24  0.06  0.3  25  OUTLET


[INFILTRATION]
;;Subcatchment  CurveNum    DryTime
;;--------------  ----------  ----------  ----------
S1  98  0.2  6.5
S2  98  0.2  6.5
S3  87  0.2  6.5
S4  0  0.2  6.5
S5  0  0.2  6.5
S6  0  0.2  6.5
S7  0  0.2  6.5
S8  0  0.2  6.5
S9  0  0.2  6.5
S10  0  0.2  6.5
S11  79  0.2  6.5
S12  69  0.2  6.5
S13  0  0.2  6.5
S14  0  0.2  6.5
S15  0  0.2  6.5
S16  0  0.2  6.5
S17  0  0.2  6.5
S18  0  0.2  6.5
S19  0  0.2  6.5
S20  0  0.2  6.5


[JUNCTIONS]
;;Name  Elevation  MaxDepth  InitDepth  SurDepth  Aponded
;;--------------  ----------  ----------  ----------  ----------  ----------
J1  0  0  0  0  0

[OUTFALLS]
```

```
;;Name  Elevation  Type  Stage Data  Gated  Route To
;;--------------  ----------  ----------  ----------------  --------  ----------------
O1  0  FREE    NO

[CONDUITS]
;;Name  From Node  To Node  Length  Roughness  InOffset  OutOffset  InitFlow  MaxFlow
;;--------------  ----------------  ----------------  ----------  ----------  ----------  ----------  ----
------  ----------
C1  J1  O1  66.28  0.001  0  0  0  0

[XSECTIONS]
;;Link  Shape  Geom1  Geom2  Geom3  Geom4  Barrels  Culvert
;;--------------  ------------  ----------------  ----------  ----------  ----------  ----------  --------
--
C1  DUMMY  0  0  0  0  1

[TIMESERIES]
;;Name  Date  Time  Value
;;--------------  ----------  ----------  ----------
;Adapted from SWMM Applciations Manual
Synth_2hr    0:00  1.131
Synth_2hr    0:05  1.289
Synth_2hr    0:10  1.504
Synth_2hr    0:15  2.522
Synth_2hr    0:20  3.212
Synth_2hr    0:25  6.209
Synth_2hr    0:30  11.254
Synth_2hr    0:35  4.660
Synth_2hr    0:40  2.805
Synth_2hr    0:45  1.651
Synth_2hr    0:50  1.380
Synth_2hr    0:55  1.199
Synth_2hr    1:00  1.131
Synth_2hr    1:05  1.074
Synth_2hr    1:10  1.029
Synth_2hr    1:15  0.984
Synth_2hr    1:20  0.950
Synth_2hr    1:25  0.916
Synth_2hr    1:30  0.882
Synth_2hr    1:35  0.848
Synth_2hr    1:40  0.826
Synth_2hr    1:45  0.803
Synth_2hr    1:50  0.780
Synth_2hr    1:55  0.758
;
;Description
Synth_24hr    0:00  0.042
Synth_24hr    0:05  0.042
Synth_24hr    0:10  0.042
Synth_24hr    0:15  0.042
Synth_24hr    0:20  0.042
Synth_24hr    0:25  0.042
Synth_24hr    0:30  0.042
Synth_24hr    0:35  0.042
Synth_24hr    0:40  0.042
Synth_24hr    0:45  0.041
Synth_24hr    0:50  0.042
Synth_24hr    0:55  0.042
Synth_24hr    1:00  0.042
Synth_24hr    1:05  0.042
Synth_24hr    1:10  0.042
Synth_24hr    1:15  0.042
Synth_24hr    1:20  0.042
Synth_24hr    1:25  0.041
Synth_24hr    1:30  0.041
Synth_24hr    1:35  0.041
Synth_24hr    1:40  0.041
Synth_24hr    1:45  0.041
Synth_24hr    1:50  0.041
```

Synth_24hr	1:55	0.042
Synth_24hr	2:00	0.043
Synth_24hr	2:05	0.043
Synth_24hr	2:10	0.043
Synth_24hr	2:15	0.043
Synth_24hr	2:20	0.043
Synth_24hr	2:25	0.043
Synth_24hr	2:30	0.046
Synth_24hr	2:35	0.046
Synth_24hr	2:40	0.046
Synth_24hr	2:45	0.046
Synth_24hr	2:50	0.046
Synth_24hr	2:55	0.046
Synth_24hr	3:00	0.049
Synth_24hr	3:05	0.049
Synth_24hr	3:10	0.049
Synth_24hr	3:15	0.049
Synth_24hr	3:20	0.049
Synth_24hr	3:25	0.049
Synth_24hr	3:30	0.052
Synth_24hr	3:35	0.052
Synth_24hr	3:40	0.052
Synth_24hr	3:45	0.052
Synth_24hr	3:50	0.052
Synth_24hr	3:55	0.052
Synth_24hr	4:00	0.056
Synth_24hr	4:05	0.056
Synth_24hr	4:10	0.056
Synth_24hr	4:15	0.056
Synth_24hr	4:20	0.056
Synth_24hr	4:25	0.056
Synth_24hr	4:30	0.059
Synth_24hr	4:35	0.059
Synth_24hr	4:40	0.059
Synth_24hr	4:45	0.059
Synth_24hr	4:50	0.059
Synth_24hr	4:55	0.059
Synth_24hr	5:00	0.062
Synth_24hr	5:05	0.062
Synth_24hr	5:10	0.062
Synth_24hr	5:15	0.062
Synth_24hr	5:20	0.062
Synth_24hr	5:25	0.062
Synth_24hr	5:30	0.065
Synth_24hr	5:35	0.065
Synth_24hr	5:40	0.065
Synth_24hr	5:45	0.065
Synth_24hr	5:50	0.065
Synth_24hr	5:55	0.065
Synth_24hr	6:00	0.072
Synth_24hr	6:05	0.072
Synth_24hr	6:10	0.072
Synth_24hr	6:15	0.072
Synth_24hr	6:20	0.072
Synth_24hr	6:25	0.072
Synth_24hr	6:30	0.082
Synth_24hr	6:35	0.082
Synth_24hr	6:40	0.082
Synth_24hr	6:45	0.082
Synth_24hr	6:50	0.082
Synth_24hr	6:55	0.082
Synth_24hr	7:00	0.092
Synth_24hr	7:05	0.092
Synth_24hr	7:10	0.092
Synth_24hr	7:15	0.092
Synth_24hr	7:20	0.092
Synth_24hr	7:25	0.092
Synth_24hr	7:30	0.103
Synth_24hr	7:35	0.103

Synth_24hr	7:40	0.103
Synth_24hr	7:45	0.103
Synth_24hr	7:50	0.103
Synth_24hr	7:55	0.103
Synth_24hr	8:00	0.120
Synth_24hr	8:05	0.120
Synth_24hr	8:10	0.120
Synth_24hr	8:15	0.120
Synth_24hr	8:20	0.120
Synth_24hr	8:25	0.120
Synth_24hr	8:30	0.144
Synth_24hr	8:35	0.144
Synth_24hr	8:40	0.144
Synth_24hr	8:45	0.144
Synth_24hr	8:50	0.144
Synth_24hr	8:55	0.144
Synth_24hr	9:00	0.168
Synth_24hr	9:05	0.168
Synth_24hr	9:10	0.168
Synth_24hr	9:15	0.168
Synth_24hr	9:20	0.168
Synth_24hr	9:25	0.168
Synth_24hr	9:30	0.191
Synth_24hr	9:35	0.191
Synth_24hr	9:40	0.191
Synth_24hr	9:45	0.191
Synth_24hr	9:50	0.191
Synth_24hr	9:55	0.191
Synth_24hr	10:00	0.228
Synth_24hr	10:05	0.228
Synth_24hr	10:10	0.228
Synth_24hr	10:15	0.228
Synth_24hr	10:20	0.228
Synth_24hr	10:25	0.228
Synth_24hr	10:30	0.278
Synth_24hr	10:35	0.278
Synth_24hr	10:40	0.278
Synth_24hr	10:45	0.278
Synth_24hr	10:50	0.278
Synth_24hr	10:55	0.278
Synth_24hr	11:00	0.398
Synth_24hr	11:05	0.398
Synth_24hr	11:10	0.398
Synth_24hr	11:15	0.398
Synth_24hr	11:20	0.398
Synth_24hr	11:25	0.398
Synth_24hr	11:30	1.677
Synth_24hr	11:35	1.677
Synth_24hr	11:40	1.677
Synth_24hr	11:45	1.677
Synth_24hr	11:50	1.677
Synth_24hr	11:55	1.677
Synth_24hr	12:00	1.677
Synth_24hr	12:05	1.677
Synth_24hr	12:10	1.677
Synth_24hr	12:15	1.677
Synth_24hr	12:20	1.677
Synth_24hr	12:25	1.677
Synth_24hr	12:30	0.398
Synth_24hr	12:35	0.398
Synth_24hr	12:40	0.398
Synth_24hr	12:45	0.398
Synth_24hr	12:50	0.398
Synth_24hr	12:55	0.398
Synth_24hr	13:00	0.278
Synth_24hr	13:05	0.278
Synth_24hr	13:10	0.278
Synth_24hr	13:15	0.278
Synth_24hr	13:20	0.278

Synth_24hr	13:25	0.278
Synth_24hr	13:30	0.228
Synth_24hr	13:35	0.228
Synth_24hr	13:40	0.228
Synth_24hr	13:45	0.228
Synth_24hr	13:50	0.228
Synth_24hr	13:55	0.228
Synth_24hr	14:00	0.191
Synth_24hr	14:05	0.191
Synth_24hr	14:10	0.191
Synth_24hr	14:15	0.191
Synth_24hr	14:20	0.191
Synth_24hr	14:25	0.191
Synth_24hr	14:30	0.168
Synth_24hr	14:35	0.168
Synth_24hr	14:40	0.168
Synth_24hr	14:45	0.168
Synth_24hr	14:50	0.168
Synth_24hr	14:55	0.168
Synth_24hr	15:00	0.144
Synth_24hr	15:05	0.144
Synth_24hr	15:10	0.144
Synth_24hr	15:15	0.144
Synth_24hr	15:20	0.144
Synth_24hr	15:25	0.144
Synth_24hr	15:30	0.120
Synth_24hr	15:35	0.120
Synth_24hr	15:40	0.120
Synth_24hr	15:45	0.120
Synth_24hr	15:50	0.120
Synth_24hr	15:55	0.120
Synth_24hr	16:00	0.103
Synth_24hr	16:05	0.103
Synth_24hr	16:10	0.103
Synth_24hr	16:15	0.103
Synth_24hr	16:20	0.103
Synth_24hr	16:25	0.103
Synth_24hr	16:30	0.092
Synth_24hr	16:35	0.092
Synth_24hr	16:40	0.092
Synth_24hr	16:45	0.092
Synth_24hr	16:50	0.092
Synth_24hr	16:55	0.092
Synth_24hr	17:00	0.082
Synth_24hr	17:05	0.082
Synth_24hr	17:10	0.082
Synth_24hr	17:15	0.082
Synth_24hr	17:20	0.082
Synth_24hr	17:25	0.082
Synth_24hr	17:30	0.072
Synth_24hr	17:35	0.072
Synth_24hr	17:40	0.072
Synth_24hr	17:45	0.072
Synth_24hr	17:50	0.072
Synth_24hr	17:55	0.072
Synth_24hr	18:00	0.065
Synth_24hr	18:05	0.065
Synth_24hr	18:10	0.065
Synth_24hr	18:15	0.065
Synth_24hr	18:20	0.065
Synth_24hr	18:25	0.065
Synth_24hr	18:30	0.062
Synth_24hr	18:35	0.062
Synth_24hr	18:40	0.062
Synth_24hr	18:45	0.062
Synth_24hr	18:50	0.062
Synth_24hr	18:55	0.062
Synth_24hr	19:00	0.059
Synth_24hr	19:05	0.059

```
Synth_24hr    19:10  0.059
Synth_24hr    19:15  0.059
Synth_24hr    19:20  0.059
Synth_24hr    19:25  0.059
Synth_24hr    19:30  0.056
Synth_24hr    19:35  0.056
Synth_24hr    19:40  0.056
Synth_24hr    19:45  0.056
Synth_24hr    19:50  0.056
Synth_24hr    19:55  0.056
Synth_24hr    20:00  0.053
Synth_24hr    20:05  0.053
Synth_24hr    20:10  0.053
Synth_24hr    20:15  0.053
Synth_24hr    20:20  0.053
Synth_24hr    20:25  0.053
Synth_24hr    20:30  0.050
Synth_24hr    20:35  0.050
Synth_24hr    20:40  0.050
Synth_24hr    20:45  0.050
Synth_24hr    20:50  0.050
Synth_24hr    20:55  0.050
Synth_24hr    21:00  0.048
Synth_24hr    21:05  0.048
Synth_24hr    21:10  0.048
Synth_24hr    21:15  0.048
Synth_24hr    21:20  0.048
Synth_24hr    21:25  0.048
Synth_24hr    21:30  0.046
Synth_24hr    21:35  0.046
Synth_24hr    21:40  0.046
Synth_24hr    21:45  0.046
Synth_24hr    21:50  0.046
Synth_24hr    21:55  0.046
Synth_24hr    22:00  0.043
Synth_24hr    22:05  0.043
Synth_24hr    22:10  0.043
Synth_24hr    22:15  0.043
Synth_24hr    22:20  0.043
Synth_24hr    22:25  0.043
Synth_24hr    22:30  0.041
Synth_24hr    22:35  0.041
Synth_24hr    22:40  0.041
Synth_24hr    22:45  0.041
Synth_24hr    22:50  0.041
Synth_24hr    22:55  0.041
Synth_24hr    23:00  0.039
Synth_24hr    23:05  0.039
Synth_24hr    23:10  0.039
Synth_24hr    23:15  0.039
Synth_24hr    23:20  0.039
Synth_24hr    23:25  0.039
Synth_24hr    23:30  0.036
Synth_24hr    23:35  0.036
Synth_24hr    23:40  0.036
Synth_24hr    23:45  0.036
Synth_24hr    23:50  0.036
Synth_24hr    23:55  0.036

[REPORT]
;;Reporting Options
SUBCATCHMENTS  ALL
NODES  ALL
LINKS  ALL

[TAGS]

[MAP]
DIMENSIONS  -100  -100  600  500
```

```
Units  Feet

[COORDINATES]
;;Node  X-Coord  Y-Coord
;;--------------  ------------------  ------------------
J1  500  300
O1  500  400

[VERTICES]
;;Link  X-Coord  Y-Coord
;;--------------  ------------------  ------------------

[Polygons]
;;Subcatchment  X-Coord  Y-Coord
;;--------------  ------------------  ------------------
S1  0  0
S1  100  0
S1  100  50
S1  0  50
S2  100  0
S2  200  0
S2  200  50
S2  100  50
S3  200  0
S3  300  0
S3  300  50
S3  200  50
S4  300  0
S4  400  0
S4  400  50
S4  300  50
S5  400  0
S5  500  0
S5  500  50
S5  400  50
S6  0  50
S6  100  50
S6  100  100
S6  0  100
S7  100  50
S7  200  50
S7  200  100
S7  100  100
S8  200  50
S8  300  50
S8  300  100
S8  200  100
S9  300  50
S9  400  50
S9  400  100
S9  300  100
S10  400  50
S10  500  50
S10  500  100
S10  400  100
S11  0  50
S11  100  50
S11  100  100
S11  0  100
S12  100  50
S12  200  50
S12  200  100
S12  100  100
S13  200  50
S13  300  50
S13  300  100
S13  200  100
S14  300  50
S14  400  50
```

```
S14  400  100
S14  300  100
S15  400  50
S15  500  50
S15  500  100
S15  400  100
S16  0  100
S16  100  100
S16  100  150
S16  0  150
S17  100  100
S17  200  100
S17  200  150
S17  100  150
S18  200  100
S18  300  100
S18  300  150
S18  200  150
S19  300  100
S19  400  100
S19  400  150
S19  300  150
S20  400  100
S20  500  100
S20  500  150
S20  400  150


[SYMBOLS]
;;Gage  X-Coord  Y-Coord
;;--------------  ------------------  ------------------
RainGage  0  300
```