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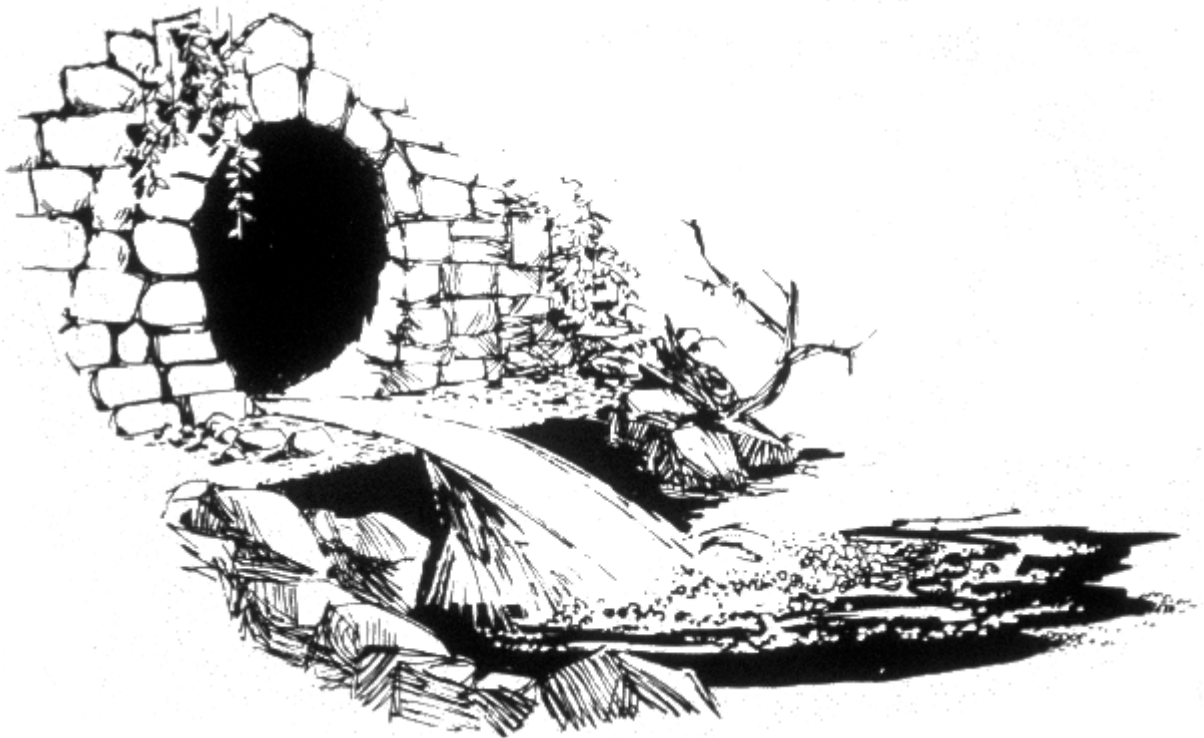# Methods for Optimizing Urban Wet-Weather Control System

Methods for Optimizing Urban Wet-Weather Control System

by

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Abstract

To minimize impacts of urban nonpoint source pollution and associated costs of control (storage and treatment) associated with wet-weather flows (WWFs), stormwater runoff volumes and pollutant loads must be reduced. A number of control strategies, so-called "best management practices" (BMPs) are being used to mitigate runoff volumes and associated nonpoint source (diffuse) pollution due to WWFs. They include ponds, bioretention facilities, infiltration trenches, grass swales, filter strips, dry wells, and cisterns. Another control option is popularly termed "low impact development" (LID) – or hydrologic source control – and strives to retain a site's pre-development hydrologic regime, reducing WWF and the associated nonpoint source pollution and treatment needs.

Methods are needed to evaluate these BMPs, their effectiveness in attenuating flow and pollutants, and to optimize their cost/performance since most models only partially simulate BMP processes. Enhanced simulation capabilities will help planners derive the least-cost combination for effectively treating WWFs. There is a confusing array of options for analyzing hydrologic regimes and planning for LID. Integrating available BMP and LID processes into one model is highly desirable.

Described in this report is a methodology that integrates simulation ("what–if" analysis) and optimization ("what's– best" analysis) for evaluating which of the myriad of alternative wet-weather controls deserves the title of "best." The optimization analysis integrates process simulation, cost-effectiveness analysis, performance specification, and optimization methods to find this "best" solution. All of these analyses are performed using a spreadsheet platform. Following a general review of optimization methods and previous applications to wet-weather control optimization, a series of spreadsheet based tools are described. Use of these spreadsheets allows for an improved method for spatial analysis and therefore, to a more accurate representation of land use. A spreadsheet-based method for analyzing precipitation records to partition them into storm events or to develop intensity-duration-frequency curves is presented, along with simple methods for estimating infiltration and performing flow routing. Influent pollutant loads may be described simply as event mean concentrations (EMCs). A spreadsheet version of the STORM model for continuous simulations is presented, followed by an update on the cost of wet-weather controls. A primer on optimization methods describes the ease of using these techniques in a spreadsheet environment and the application of these tools to optimize storm sewer design is discussed. At the conclusion, an integrated stormwater management optimization model that combines land use optimization and a storage-release system is outlined.

The effort documented in this report is linked to a parallel effort at Oregon State University titled: BMP Modeling Concepts and Simulation. This work analyzes several current modeling methods to evaluate BMP performance with the intention of facilitating integration of improved BMP modeling methods into the EPA's Storm Water Management Model (SWMM). Several other models are examined as part of this study. Options for enhancement of SWMM's LID simulation capabilities are also presented. Two extensive case studies in Portland, Oregon help to clarify current SWMM capabilities and needs for enhancement.

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List of Acronyms and Abbreviations

BMP   best management practice
CDF   cumulative distribution function
cfs   cubic feet per second
CFSTR   continuous-flow, stirred-tank reactor
CSO   combined sewer overflow
CU   University of Colorado
CV   coefficient of variation
DCIA   directly connected impervious area
DTM   digital terrain model
EPA   United States Environmental Protection Agency
EMC   event mean concentration
EP   evolutionary program
ES   evolutionary solver
ET   evapotranspiration
FPC   fundamental process category
GIS   geographic information system
GUI   graphical user interface
ICIA   indirectly connected impervious area
IETD   inter-event time definition
LID   low-impact development
LP   linear program
NLP   nonlinear program
NRMRL   National Risk Management Research Laboratory
NURP   Nationwide Urban Runoff Program
O&M   operation and maintenance
OSU   Oregon State University
P   phosphorus
PA   pervious area
PFR   plug flow reactor
RTC   real time control
RTD   residence time distribution
SS   spreadsheet
SSO   sanitary sewer overflow
S/T   storage/treatment
SWMM   EPA Storm Water Management Model
TIA   total impervious area
TMDL   total maximum daily load
TP   total phosphorus
TSS   total suspended solids
USDA   United States Department of Agriculture
USGS   United States Geological Survey
USLE   Universal Soil Loss Equation
UWRRC   Urban Water Resources Research Council
VMT   vehicle miles traveled
WEF   Water Environment Federation
WWC   wet-weather control
WWF   wet-weather flow

Acknowledgments

Project Publications

Publications resulting wholly or in part from Task A project activities are included in the References section and are listed by author below:

Lee, J. G, and Heaney, J. P. (2002). "Directly Connected Impervious Areas as Major Sources of Urban Stormwater Quality Problems-Evidence from South Florida." In *Proc. 7th Biennial Conference on Stormwater Research and Water Quality Management*, Tampa, Florida.

Heaney, J. P. (2002). "Methodology for Finding the Optimal Mix of On-site and Off-site Wet-weather Controls." *Proc. 9th Int. Conference on Urban Storm Drainage, Portland*, Oregon, September.

Lee, J. G., and Heaney, J. P. (2003). "Estimation of urban imperviousness and its impacts on stormwater systems." *Jour. of Water Resources Planning and Management*. Vol. 129, No. 5, pp. 419-426.

Lee, J. G. (2003). *Process Analysis and Optimization of Distributed Urban Stormwater Management Strategies*. PhD Dissertation, Univ. of Colorado, Boulder, CO.

Lee, J. G., Heaney, J. P., and Lai, F-H. (2005). "Optimization of Integrated Urban Wet-weather Control Strategies", *Journal of Water Resources Planning and* Managemen, Vol. 131, No. 4, pp. 307-315..

Chapter 1 Introduction

The Needs

Urban stormwater runoff causes many environmental problems, including flooding, soil erosion, and pollutant discharge to receiving waters. A wide variety of urban wet-weather control (WWC) alternatives have been applied to abate stormwater-induced water quantity and quality problems. However, the technical performance and/or economic optimization of these controls have not been studied because of their complexity. In recent years, more distributed approaches or on-site urban WWCs, e.g., low impact development (LID) and parcel-level best management practices (BMPs), have been promoted as urban stormwater management choices besides traditional off-site stormwater storage-release systems. The potential cost of these stormwater controls can be very large and every effort needs to be made to select the most cost-effective and sustainable long-term solutions. These approaches need to be analyzed using improved process simulation and optimization models to determine the best mix of traditional downstream controls and the more recently espoused upstream or source controls. The wide variety of upstream and downstream WWCs makes the optimization problem complex.

Statement of Task

This report is part of a larger project titled "Optimization of Urban Sewer Systems during Wet-weather Periods" (EPA Contract No. 68-C-01-020) to the University of Colorado (CU), with a subcontract to Oregon State University (OSU) that deals with the related issues of simulation and optimization of wet-weather controls. The subcontract to OSU focuses on simulation of the hydrology, hydraulics, and water quality aspects of urban wet-weather flows with emphasis on simulating the performance of wet-weather controls (WWCs). They review and compare existing simulators and specifically suggest how the EPA SWMM can be used for this purpose. Simulators allow the analyst to find out "what-if" and see the outcomes of inputting various scenarios into the simulator. The OSU effort presents a current summary of how simulations can be done using existing software on a variety of platforms. The results of the OSU effort are contained in Huber, Clannon, and Stouder (2004). The University of Colorado contract deals with the more general question of "what's best," i.e., what combination of control variables achieves a stated objective at the least life-cycle cost. To accomplish this objective, it is necessary to directly link the simulation model to optimization models with associated cost functions and a metric of performance. It is easiest to interface simulation and optimization if a common computational platform is used. In this project, spreadsheets have been selected since they are widely used by stormwater professionals and they offer the most flexibility.

Traditionally, wet-weather controls (WWCs) were applied at a centralized, downstream location. Accordingly, process modeling was set up to track the movement of water and associated pollutants to the point at which control was first exercised. For example, SWMM (as SWMM version 4.4h [Huber 2004]) is set up so that the water moves from the Rainfall-Runoff Block to the Transport Block (with or without Extran), and finally to the Storage/Treatment

Block where control may occur. Current thinking is that control can occur anywhere in the system, not just at a downstream, centralized, location. LID concepts embody this change in attitude. Therefore, control can be exercised in the Rainfall-Runoff Block of SWMM, as well as at downstream locations. From a more fundamental viewpoint, a wet-weather system is a linkage of control volumes in series and/or in parallel. Water and pollutants are routed through each control volume, and this process can be manipulated, at a cost, to achieve some desired objective. The new SWMM 5.0 released in 2004 takes this more fundamental approach to the modeling problem, and this approach is adopted in this report.

Three key factors are important to optimize wet-weather control (WWC) system performance: (1) knowing the costs of control alternatives, (2) knowing how well the control alternatives perform, and (3) being able to consider all viable alternatives. The cost should consider all life-cycle costs of capital and O&M investments, including costs of land for each of the viable control alternatives. Evaluation of performance involves the use of computer models/tools and/or empirical relationships derived from operational data for a quantitative estimate of the amount of pollution removed. The full analysis of all viable alternatives is essentially a cost-effectiveness analysis that integrates cost and performance data of all controls (source, in-system and off-system) and treatment alternatives. It ensures that the least-cost combination of alternatives is identified for a given level of pollution control (such as that needed for a specific level of receiving water quality improvement or a given reduction of pollutant loadings discharged to receiving waters), and that additional investment is not justified for the amount of environmental improvement gained by additional wet-weather controls (WWCs).

Descriptively, the wet-weather optimization problem can be expressed as follows:

>   Minimize: Life cycle costs for the controls

>   Subject to satisfying the following constraints:
>   A. Production Function: Process constraints that describe the relationships between inputs and outputs.
>   B. Performance Standard: Minimum required level of pollution control.

Methods for determining life cycle costs were presented in an earlier EPA report (Heaney et al. 2002). The performance standard is a pre-specified effluent quality for a single design event or the total over multiple events. In this report, the performance is specified in terms of a minimum percent removal of one or more pollutants measured in terms of concentration, flow volume, or load. This methodology is introduced in the following section.

Research Objectives
This research project has been performed for two primary objectives:

1. to develop methods to evaluate urban WWC systems to optimize their performance during wet-weather periods in terms of cost effectiveness; and
2. to develop modeling formulations for BMP/LID alternatives that can be incorporated into the optimization model.

The developed urban WWC optimization models are decision-aiding tools for analyzing cost-effective solutions to abate contamination from urban wet-weather flows. This includes developing modeling formulations for alternatives that can be used later by EPA when developing more refined optimization models.

Organization of the Report
The general research framework and hierarchical approach to optimizing urban WWCs are presented in Chapter 2. A case study of land use and spatial analysis is summarized in Chapter 3 and methods of long-term precipitation data analysis are presented in Chapter 4. Depression storage and infiltration modeling is described in Chapter 5. Flow routing techniques for WWC optimization are presented in Chapter 6. Pollutant characterization with site-specific data is described in Chapter 7, and then a spreadsheet-based continuous rainfall-runoff and storage-release model is

presented in Chapter 8. Cost analysis for urban WWCs is described in Chapter 9. A primer on optimization is presented in Chapter 10. A storm sewer design optimization model is presented in Chapter 11 and an optimization model for storage-based urban WWCs is presented in Chapter 12. An integrated approach for optimizing overall urban WWC strategies, including distributed on-site controls and centralized off-site systems, is presented in Chapter 13 using developed spreadsheets models. Finally, a summary and conclusions are presented in Chapters 14. Examples of functional spatial analysis are presented in the Appendix.

Chapter 2 Optimization Approach for Urban WWCs

Introduction

Historically, people have developed villages and towns to attain a safer and more convenient life. About seven percent of the land in the United States is classified as urban, but 74 percent of the people live in this area (Ferguson 1998). Urban development and its associated impact on stormwater management is one of the field's most important contemporary issues. After urbanization, stormwater runoff quantity and quality changed dramatically. The increased impervious areas associated with urbanization result in higher peak flow, increased total runoff volume, and more polluted discharge. A variety of technical approaches have been applied to reduce these impacts of urbanization and protect our environment. Modern technologies of systems analysis and optimization have been applied to water resources and stormwater management applications. In this chapter, existing efforts in the field of urban stormwater management are summarized, and then a hierarchical approach of this research project is presented.

Urban Stormwater Management

Urbanization trends and impacts

Urban land development means the conversion of natural land into the improved area that can support a desired urban activity (Dion 1993). It includes the development of transportation, water supply, wastewater, and stormwater drainage. The general approaches to urban development have changed in recent decades. People want a more isolated private life, and this is forcing the shift of new urban development approaches (Kunstler 1996). The most significant trends of contemporary urbanization are related to automobiles. In general, recent urban development needs more land and has a higher automobile dependence. Urban areas are using land four to eight times faster than the population growth rate in the United States (Heaney 2000). Goldstein (1997) estimates that about 25 percent or more of newly developed urban land is devoted to transportation-related activities. Southworth and Ben-Joseph (1995) argue that streets virtually dictate a dispersed, disconnected community pattern in order to provide automobile access. Automobile-related activities have profoundly affected urban development from the early 20th century. Between 1915 and 1994, while the population of the United States grew by a factor of 2.6 – from 100 million to 261 million, the number of automobiles grew by a factor of 80 – from 2.5 million to 200 million (Tetra Tech 1996; Heaney 2000). The number of vehicles and vehicle miles traveled (VMT) per capita have been increasing at a steady rate. Automobiles require a huge volume of transportation infrastructure – the network of streets, highways, connections or approaches, driveways and garages, and parking lots or garage buildings. Automobiles also generate hydrocarbons, oil, metals, and other pollutants in the air and on the urban surface. They are accumulated and released between rainfall events. Transportation-related infrastructure development also results in high energy consumption.

For stormwater management, the main shift due to urbanization might be the shift of land surface conditions due to urban development. Loose natural surface soil covered by vegetation has been changed to impermeable surface materials or very dense soil with much smaller vegetative cover. The impervious area increases and rainfall abstraction capability decreases. Initial depression storage, infiltration, and evapotranspiration decrease significantly after urban land development (WEF-ASCE 1998), causing the peak and volume of runoff to increase. Runoff travel time decreases because of smoother surfaces and sewer system development. Flow velocity during storms and the

stream's annual flow also increase; however, base flow and groundwater levels decrease. As a result, the frequency of runoff events and flooding increases due to urban development expanding the floodplain.

Imperviousness could be the most important indicator of urbanization (Schueler 1994; Arnold and Gibbons 1996; WEF-ASCE 1998). The impervious surfaces from transportation systems are the biggest portion of the entire impervious area (City of Olympia 1994; Schueler 1994), comprising 63 to 70 percent of total impervious surface in 11 residential, multifamily, and commercial areas (City of Olympia 1994). Urban impervious surfaces affect urban hydrologic processes significantly by preventing on-site rainwater from being absorbed and stored in the soil.

Urban rainfall washes off oils, litter, sediment, fertilizers, and chemicals from pervious and impervious surfaces. Ferguson (1998) states that about 70 percent of the water pollution in the United States comes from nonpoint sources (NPS); the excess sediment, oils, and chemicals that runoff carries from eroding soil, parking lots, and intensely maintained lawn areas. Lawns cover a larger area than any one agricultural crop in the United States, and many are maintained with large amounts of herbicides, pesticides, and fertilizers (Bormann et al. 1993). Most automobile-related infrastructures generate organic compounds, foreign chemicals, metals, and salts making it the biggest source of pollutants in urban areas, after soil and stream bank erosion (Ferguson 1998).

Water quality is affected by two phases of urbanization (Schueler 1992). During the construction phase, soil erosion and sediment transport are the main problems. After the development phase, accumulated and washed off deposits mainly from impervious surfaces become the main problem. The higher loadings can cause turbid water, nutrient enrichment, bacterial contamination, organic material loads, toxic compounds, temperature increases, and debris (WEF-ASCE 1998), resulting in harmful effects on stream ecology and habitats.

Urban stormwater quantity and quality controls
Wet-weather controls (WWCs) defined
Pollutant and hydraulic loadings from stormwater runoff are a significant component of the residual wet-weather problems in urban areas. During the past 30 years, a variety of BMPs (U.S. terminology) or Sustainable Urban Drainage Systems (SUDS) (U.K. terminology) have been deployed to reduce these impacts. The term LID,has also gained in popularity after implementation in Prince Georges County, Maryland (2000) and surrounding areas. None of these terms accurately describes the wet-weather flow processes in a more generic manner. The phrase, wet-weather controls (WWCs), will be used to describe individual control units. These units control the quantity and/or the quality of the wet-weather flows. In the most general case, control of wet-weather flows can be included anywhere in the system by manipulating, at a cost, the flow and storage processes. If the results of the optimization indicate that a WWC is the "best," then it will have merited the designation that it is indeed a BMP.

Urban WWC approaches
Historically, excess runoff drainage and flood control have been the main targets of stormwater management. Usually, 5- to 10-year return period minor storms are used for designing drainage systems and 50- to 100-year major storms are used for designing flood control facilities. However, the most frequent and highly contaminated urban runoff might be contributed by two-year or less return period micro storms (Guo and Urbonas 1996; Pitt 1999). Guo and Urbonas (1996) show that nearly 95 percent of runoff-producing events may be smaller than a two-year storm as demonstrated by the runoff event distribution for Denver, Colorado. Small but frequent rainfall-runoff events need to be evaluated for studying stormwater quality management practices.

One of the most effective quality controls is reducing the runoff peak and volume (ASCE-WEF 1992). Source controls may be the best way to prevent pollution, but can be difficult to implement. The basic concept of BMPs is to maximize on-site infiltration and to minimize runoff discharge. Drained stormwater from impervious to pervious areas can be filtered and infiltrated using swales and filter strips. Longer flow travel time on pervious surfaces results in higher pollutant removal by sedimentation and transformation. Porous pavement (Niemczynowicz 1990) and parking blocks (Pratt 1990) have excellent potential to support stormwater control practices. Infiltration devices, either above-ground infiltration basins or buried infiltration trenches, are effective stormwater control devices (ASCE-WEF 1992), but their effectiveness depends on the site-specific properties of soil and groundwater. Detention and

retention basins can be used for stormwater quantity and quality controls, even though their primary application has been for quantity-based drainage controls (ASCE-WEF 1992). The best way to solve urban runoff problems is to control them as close to the source as possible (Dunne and Leopold 1978; ASCE-WEF 1992). General design strategies for distributed WWCs are summarized as follows:

- Maximize on-site depression storage and infiltration and minimize runoff discharge.
- Minimize directly connected impervious area (DCIA).
- Maximize flow paths and time of concentration.
- Control micro storms while maintaining traditional drainage and flood control functions.
- Reduce the quantity of runoff to control water quality problems.
- Increase visibility of the hydrologic cycle in urban environs for aesthetic benefits and an identifiable urban ecology, e.g., water fountains, urban streams, and water falls.
- A maintainable and sustainable system without sacrificing large amounts of usable space.

Systems Analysis and Optimization
Operations research in general
The complexity of many real problems has led to a long history of applying systems analysis and economic optimization techniques to aid in decision making. Popular contemporary texts on operations research (OR) include the classic text by Hillier and Lieberman (2000) and a book by Rardin (2000). These texts emphasize understanding optimization concepts and algorithms. During the past several years, texts have emerged that emphasize problem formulation using spreadsheet-based software to do optimization (classical and metaheuristic), risk analysis using @Risk or Crystal Ball, and risk optimization using Risk Optimizer or OptQuest linked to Crystal Ball. Ragsdale (2000) and Winston and Albright (2001) are examples of this kind of text. State-of-the-art software comes with these books. Edgar et al. (2001) describes a variety of classical and metaheuristic optimization methods with applications to chemical processes. Optimization problems range from operation of existing systems to design of new systems. Classical optimization techniques have been applied to these problems, with varying degrees of success. A common problem of classical optimization techniques is the inability to incorporate realistic representations of the physical system. Hazelrigg (1996) defines systems engineering as "the treatment of engineering design as a decision-making process." The conclusion from these works is that the methods of systems analysis are powerful tools to aid in the design, generation, and selection of engineering alternatives to complex problems.

Modern heuristic search techniques such as genetic algorithms are not as restrictive in problem formulation as the classical approaches. Heuristic methods may be applied to optimization problems that exhibit characteristics that are incompatible with traditional optimization methods. A meta-heuristic (MH) is a "master strategy" used to guide other heuristics and generate solutions beyond local optima (Glover and Laguna 1997). Meta-heuristic search techniques are commonly divided into population-based methods and methods that utilize adaptive memory, though this distinction may be blurred under close scrutiny (Glover and Laguna 1997). Genetic algorithms and scatter search are examples of population-based methods, and tabu search is an example of a method that uses adaptive memory (Glover and Laguna 1997).

Four common forms of meta-heuristic optimization techniques are genetic algorithms (GA), tabu search (TS), simulated annealing (SA) and scatter search (SS). Of these, GA is the most common. GAs are based on an evolutionary metaphor, where the goal is to carry positive traits of solution sets (populations) forward, mixing portions of solutions in a semi-random fashion. TS methods are derived from techniques based in artificial intelligence, using adaptive and short-term memory to move from one solution to another. SA algorithms use a physical metaphor, that of metal crystallizing as it cools. Crystals form by forming low-energy molecular structure, and SA techniques exploit mathematical representations of this process. SS is an early technique, predating other meta-heuristics, and a constrained random search is used to improve solution performance as measured by an objective function.

Conceptually, all of these techniques may be linked to process simulators, and it is likely that they will perform

reasonably well. MH performance is dependent on the process simulator as well as the optimization algorithm. Therefore, performance of the optimization algorithm is tied to the performance of the simulator.

Operations research in urban WWCs

The general gravity storm sewer design problem weighs the costs of excavation against pipe diameter costs (Mays and Tung 1992, Miles and Heaney 1988). Smaller diameter pipes are cheaper per linear foot, but require greater energy to transmit a given flow rate than larger and more expensive pipes. The energy used in storm sewers is gravitational; therefore the increased gravitational energy needed for smaller pipes is incorporated in the system by using steeper slopes than those needed for larger pipes. Steeper slopes require greater excavation volumes. Thus, the tradeoff between pipe size and slope is fundamental to developing a least cost sewer design (Mays and Tung 1992).

Researchers using dynamic programming (DP) techniques have had some success in optimizing storm sewer design problems (Robinson and Labadie 1981; Mays and Tung 1992 and 1996). Dynamic programming is an optimization method that breaks down a complex problem with many decision variables into a series of separate, but interrelated, single-decision subproblems (Mays and Tung 1992 and 1996). Dynamic programming requires significant computational resources, and suffers from the "curse of dimensionality", the rapid growth of computational requirements with small growth in problem size (Mays and Tung 1992). Discrete differential dynamic programming (DDDP) was developed to relax some of the computational requirements of DP. However, DDDP techniques suffer in that they are dependent on an initial trial solution, and may converge to a local optimum instead of a global optimum (Mays and Tung 1992).

Mays and Tung (1992) developed a DDDP solution for a small (three pipes and four manholes) storm sewer. The solution for this small problem requires significant computational development. Expanding this to a larger system with a finer solution grid would require significant computational resources, as well as a complex problem formulation. Another common criticism of DP techniques is that the formulations tend to be problem specific, and are therefore difficult to program in a general code. Specific problems tend to require specific computer codes. Robinson and Labadie (1981) describe CSUDP-SEWER, a generalized DP code for storm sewer design. This approach requires more computational resources than problem-specific DP codes, and suffers from the other limiting aspects of DP.

Miles and Heaney (1988) demonstrated the weaknesses of DDDP approaches to storm sewer optimization. A case study originally formulated by Yen et al. (1976) using a DDDP approach, and later revisited by Robinson and Labadie (1981) using the general DDDP code CSUDP-SEWER, was used to show how simplifications used in the DDDP led to infeasible and sub-optimal solutions. The two DDDP solutions used a simplified velocity calculation to verify that a solution did not exceed a maximum allowable pipe velocity. However, Miles and Heaney (1988) showed that a more accurate velocity estimation revealed that both the Yen et al. (1976) and the Robinson and Labadie (1981) solutions violated the maximum velocity constraint. The Miles and Heaney (1988) approach was simply a spreadsheet trial and error solution method modeled after a design template used for hand calculations. However, the spreadsheet used a more accurate calculation of velocity. This trial and error formulation is perfectly suited for adaptation to an intelligent search technique like genetic algorithms. For these reasons, there has been interest in using heuristic search techniques for complex water resources problems such as the storm sewer design problem. A heuristic algorithm is defined by Reeves (1993) as "a technique which seeks good (i.e., near-optimal) solutions of an optimization problem at a reasonable computational cost without being able to guarantee optimality."

Jacobs et al. (1997) define a reliability and cost trade-off curve for a stormwater drainage system. A multi-objective mixed integer optimization model is used. A Monte Carlo (MC) analysis based on fitted rainfall distributions is used to generate the statistical parameters of a hyetograph. These storm events are in turn used to determine the hydraulic performance of the system using the kinematic wave formulations of overland and channel flow. The optimization model is only solved relatively few times (compared to a meta-heuristic). In the example solution, the reliability and cost trade-off curve is defined by only eight least-cost solutions at various levels of probability of failure (Jacobs et al. 1997). However large the population of "elite" solutions, it is evident that the use of a set of solutions is valuable in evaluating various measures of fitness. Stinson et al. (2000) summarized efforts to optimize real-time control of

combined sewerage systems in Quebec City, Canada and near Paris, France. Nicklow and Hellman (2000) show how the hydraulic design of highway drainage inlet systems can be optimized by linking a hydraulic simulator to a genetic algorithm. Heaney et al. (2000 and 2002) show how the storm sewer design problem can be optimized using a MH. Wright et al. (2001) showed how risk optimization techniques can be used to address a sanitary sewer overflow problem in Vallejo, California.

Optimization techniques are also useful in calibrating a simulation model to find the "best fit" between the simulated and measured values. This problem is often called the parameter estimation problem. Much work has been done on the parameter estimation problem for calibrating surface and groundwater models in hydrology. The parameter estimation problem can be represented as one of minimizing the total error by varying model parameters over reasonable ranges. The typical output of interest in urban wet-weather controls is a hydrograph or pollutograph. For example, Hamid (1995) used the SCS method and SWMM parameters to calibrate models for the Kings Creek dataset for South Florida. For the SCS method, the curve number can be used for calibration. Using SWMM, the catchment width is a popular choice for calibration. Mohan (1997) shows how a GA can be used to find the parameters of the Muskingum Method. One difficulty with parameter estimation is that the parameters of "best fit" may not be plausible from a physical point of view. For example, the calibrated catchment width is much different from the actual width. Another problem is that a large, if not infinite, number of solutions exists when many parameters can be used for calibration.

Early Work on Multiple Levels of SWMM and STORM
The initial motivation for developing the EPA SWMM was to evaluate combined sewer overflow (CSO) problems. The original SWMM was set up to evaluate a single storm event. However, it soon became evident that simpler evaluation models were needed that could evaluate the integrated performance of the wet-weather system over multiple events. The STORM model was developed to address this need. Also, simple versions of SWMM (Levels I and II) were developed to support wet-weather evaluations including areawide wastewater planning, popularly called the "208" planning. These models relied on graphical solution techniques. The database for these simpler models was created by running more complex computer simulation models, typically SWMM and STORM. Huber and Dickinson (1988) mention four simulation modeling levels associated with SWMM:

- Level I - Screening Models, e.g., Heaney, et al. (1976), and Heaney and Nix (1977)
- Level II - Planning Models (e.g., Medina, et al. 1981a, b, c)
- Level III - Design Models (SWMM)
- Level IV - Operational Models (SWMM with Extran; calibration against measured data).

The approach presented in this report draws on this earlier taxonomy. The major technological change since the 1970's when these ideas were originally developed is the switch from mainframe computers to PCs. A major domain shift in recent years is from total reliance on large, centralized control systems to better integration of decentralized controls into the overall wet-weather management system. Computationally, it is now possible to directly link process simulators and optimizers within a spreadsheet (SS) platform. Also, many of the numerical computations that couldn't be solved without simulators such as SWMM can now be solved on a spreadsheet.

Wet-weather Optimization Framework
Progress has been made in successfully applying optimization methods to urban WWC problems. In previous efforts, the optimization process was divided into two main steps. First, a process simulator such as STORM or SWMM was run numerous times to define the production function, i.e., the locus of technologically efficient combinations of n inputs, $x = (x_1, x_2, ..., x_n)$, and m outputs, $y = (y_1, y_2, ..., y_m)$. Because of the limitations of optimization techniques and the complexity of the process simulations, the most complete functional relationships that could be analyzed consisted of problems with a single output, y, and two inputs, or $y = f(x_1, x_2)$. An illustrative production function is shown in Figure 2-1.

(a) response surface   (b) contour plot
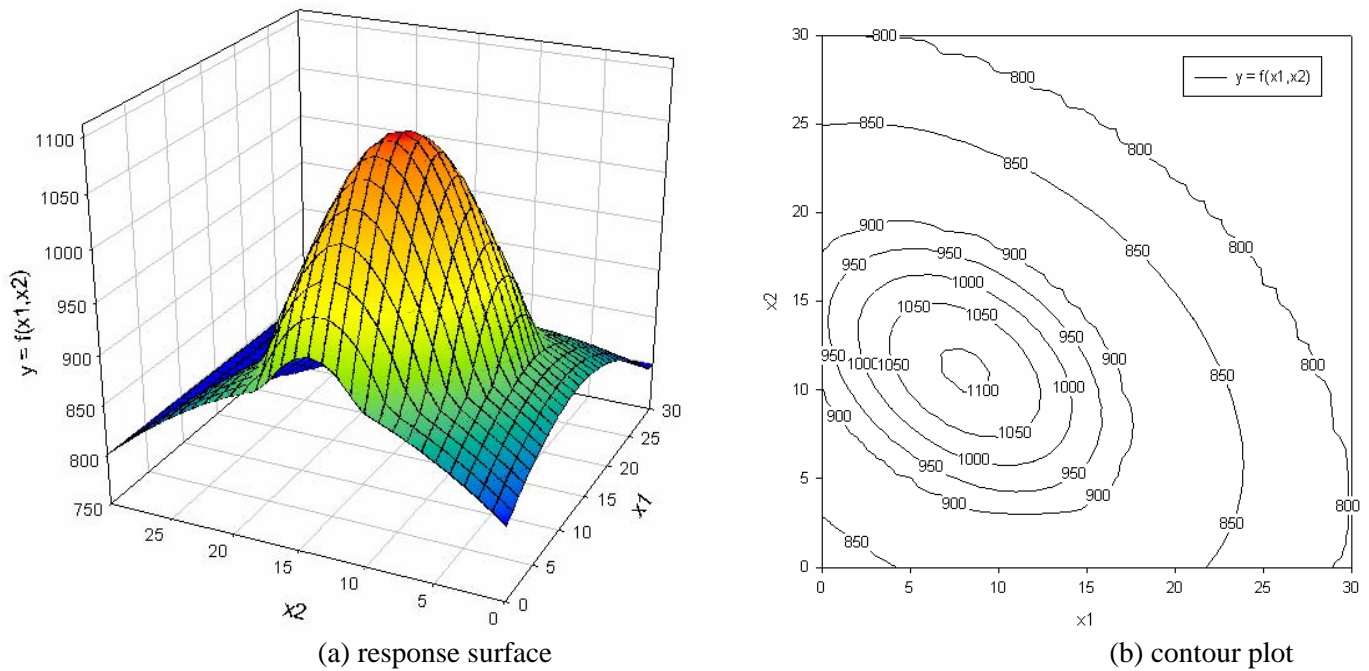
Figure 2-1 Production function of output, y, as a function of two inputs, x_1 and x_2.

This optimization was then done for each individual control, and then the controls were linked in series and parallel in order to find the best overall solution. In summary, the steps in the traditional optimization process are as follows:

Step 1. Set up and run a process simulator such as STORM or SWMM numerous times to generate a database of observations, [y, x_1, x_2]. Ideally, the process simulation model should be calibrated using local data. In the earlier work, it was relatively difficult to run these simulations. Thus, only a limited number of m simulations were run. As computers have become much faster, it is now possible to run many more simulations and reduce, or even eliminate, the work in Step 2.

Step 2. Using a variety of data mining techniques, find a functional relationship for $y = f(x_1, x_2)$. The result may be one of the following:

- A single analytical equation.
- A numerical solution using splines that allows the user to approximate y for any assumed values of x_1 and x_2 within the range of the database. Using this approach, the original database from m simulations is now approximated by a much larger number of n realizations.
- The original database, [y, x_1, x_2], may be used if sufficient simulations have been done to define the response surface. This total enumeration approach may be preferable if the simulation run times are short.

Step 3. Estimate the cost of this control as a function of x_1 and x_2 using historical data on comparable systems and/or engineering estimates based on process costing approaches. The result of the cost analysis is another database, [C, x_1, x_2].

Step 4. Using a variety of data mining techniques, find a functional relationship for $C = g(x_1, x_2)$. The result may be one of the following:

- A single analytical equation.

- A numerical solution using splines that allows the user to approximate C for any assumed values of x_1 and x_2 within the range of the database.
- The original database, $[C, x_1, x_2]$, may be used if sufficient cost simulations have been done to define the cost response surface. This total enumeration approach may be preferable if process costing simulation run times are short. Automated costing software is rarely available so this option is seldom used.

Step 5. Find the least cost solution for various values of the output, y, using optimization procedures and derive a performance database of $[C, y]$ pairs. Traditional choices for optimization are as follows:

- If single analytical expressions are available for the production function, $y = f(x_1, x_2)$ and the cost function, $C = g(x_1, x_2)$, then the optimal solution for $C = h(y)$ can be derived analytically in the form of a single equation. This option is the most elegant but is seldom accurate due to the difficulty in finding a single analytical expression for the production function.
- If a numerical expression for the production function is available, enumerate sufficient realizations of the production function and then calculate the associated cost to generate a $[C, y]$ database. Fit a function to this database to find the final answer.

Much of the research in optimizing WWC problems was done in the 1970's and early 1980's. These methods proved to be very helpful in doing national assessments and in finding more cost-effective solutions to wet-weather problems.

Computational Considerations

During the past 20 years, the personal computer (PC) revolution has significantly changed the way professionals evaluate problems. Spreadsheets (SSs) have become ubiquitous and are the standard computing tool. These SSs are excellent for doing "what-if" analysis wherein many scenarios can be run by changing one or two variables at a time. Optimizers are also built into SSs allowing users to determine "what's best." Recent advances in optimization methods have added very powerful metaheuristic (MH) search procedures so that "what's best" can be determined for virtually any problem that is set up on a SS for doing "what-if" analysis. These advanced optimization techniques can be added to the standard SS Solvers. This report describes these newer optimization techniques and how they can be used to solve WWC problems.

Recall that the initial step in contemporary WWC analysis is to run a process simulator such as STORM or SWMM in order to generate the $[y, x_1, x_2]$ database. STORM and SWMM are legacy Fortran programs that were developed in the 1970s. STORM has not been updated and is not widely available. SWMM is still widely used and has been totally rewritten. The primary input for STORM was long time series of hourly precipitation data. The actual calculations of performance are relatively straightforward. SWMM includes extensive use of numerical methods to solve the more complex hydraulic problems such as estimating backwater curves. The other major thrust of this research is to see which components of STORM and SWMM can be programmed on a SS. If the simulation can be done directly on a SS, then it is simple to invoke the built-in optimizer and solve the WWC problem directly. If the simulation cannot be done on a SS, then it is necessary to write an interface between the SS and process simulator. This can be done if the simulation software is of recent vintage, e.g., EPANET for evaluating water distribution systems (Lippai et al. 1999). However, it is difficult to do for legacy software such as STORM and SWMM. Thus, a major effort in this research was devoted to incorporating the functionality of SWMM and STORM into a SS environment so that it would be directly available to the dominant class of users who routinely work in this environment. The major paradigm shift of this approach is that, instead of arbitrarily selecting a uniform grid of input variables to test, the optimizer is used to direct the selection of the simulation runs in order to find the optimal solution in the most direct manner. Also, the decision space of the problem is no longer restricted to simple three dimensional representations, i.e., $y = f(x_1, x_2)$, but is now able to find optimal solutions for many decision variables.

SS Simulation Options and Hierarchical Approach for Optimization
Optimization-simulation methodology
The traditional formulation for a constrained optimization problem is as follows:

Max. or min. $\quad Z = f(x)$

Subject to $\quad g(x) \leq, =, or \geq b$ **Equation 2-1**

$\quad\quad\quad\quad x \geq 0$

The constraint set, $g(x) \leq = \geq b$, includes the process characterization relationships and performance criteria. If classical programming techniques are used, then the objective function and the constraint set must be well behaved in the mathematical sense, meaning that the relationships are linear, nonlinear but convex, etc. If the problem can be formulated as a coupled system, the optimization methods work quite well and the optimal solution can be found quickly. However, if the objective function and/or the constraint set violate the conditions for solving a classical optimization problem, then the more flexible evolutionary solvers (ESs) need to be used. Some expertise is needed to properly match the simulator and the optimizer. For example, logical statements in the simulator may disallow the use of classical optimization methods.

The newer paradigm is to decouple the optimization model and the process simulator as shown in Figure 2-2. The fundamental paradigm shift from the classical to newer approaches is to view the problem as one of linking an optimizer to a simulator so that the optimizer drives the simulator. A key computational question is how to write an interface program that links the optimizer and the simulator. If the simulator is in a spreadsheet, it is very simple to link the optimizer and no interface program is needed.
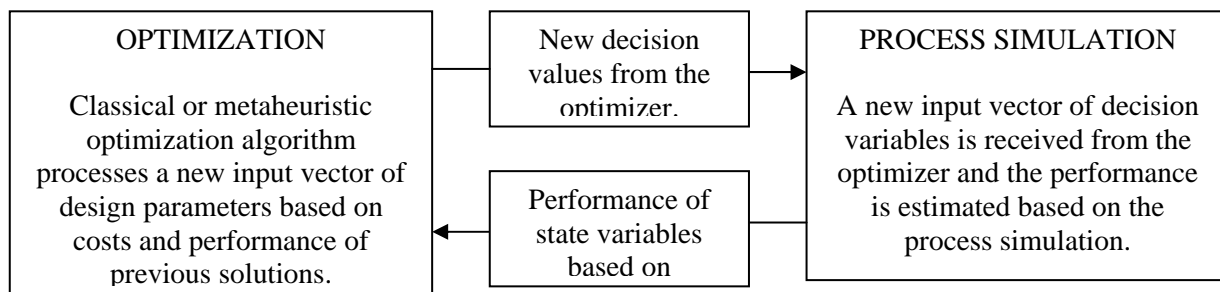
Figure 2-2 Decoupled optimization-simulation model (after Wright et al. 2001).

Hierarchical process simulators and SS optimization options
A hierarchy of spreadsheet-based process simulators has been developed as shown in Table 2-1. The user can select from this menu and combine the components as they see fit to do a simulation. Details of these methods are described in subsequent sections.

The standard Solver in Excel has the following two choices for doing optimization:

- Default is Generalized Reduced Gradient (GRG) nonlinear programming.
- Linear model option can be chosen if linearity assumptions are satisfied.

The Excel Premium Solver (http://www.solver.com) offers three choices for doing the optimization:

- Linear and quadratic programming
- GRG nonlinear programming
- Evolutionary programming

The evolutionary programming option is a metaheuristic that employs sophisticated search techniques to find a good, and perhaps, an optimal solution. Strategic considerations for selecting among these options will be presented in Chapter 10.

Table 2-1 Options in urban wet weather simulator.

Land Use	Weighted average of land uses from references
	GIS-based functional spatial analysis
Rainfall Input	Single intensity-duration-frequency (IDF) design storm
	Storm event time series
	Measured time series
	Disaggregated time seriese
Depression Storage (DS) and Infiltration	Fixed DS and infiltration rate
	Fixed DS with Horton's infiltration method
Evapotranspiration (ET)	Averaged monthly ET data
	Measured daily ET data
	Measured hourly ET data
Flow Routing	Rational Method (Single event, Peak only)
	Indirect using Rational Method
	Continuous rainfall-runoff & storage-release simulation
Pollutant Inputs	Annual total pollutant load
	Fixed event mean concentration (EMC)
	Measured or assumed pollutographs
Pollutant Treatment	Plug-flow model with assumed reaction rate
	Plug-flow model with site-specific reaction rate

Summary and Conclusions

This chapter summarized urban development trends, urbanization impacts on stormwater management, and systems analysis and optimization in water resources engineering. Automobile-dependent modern urbanism has become the most significant trend of urbanization during the past decades. A hierarchical approach of process simulation and optimization for analyzing urban WWCs was schematically presented in this chapter. Detailed descriptions of each option will be followed in the rest of this report.

Chapter 3 Land Use and Spatial Analysis

Introduction
Prior to the availability of geographic information systems (GIS) and associated databases, spatial analysis typically consisted of highly aggregated characterizations of land uses into major categories such as residential, commercial, and industrial. In addition, study areas were divided into relatively few catchments to make the computations easier. This approach was easier to justify when the analysis focused on large, downstream controls. However, now much of the interest is in decentralized controls such as low impact development (LID). In such an approach, a larger number of smaller wet-weather controls can be used instead of a smaller number of centralized controls.

This chapter evaluates the potential improvements in land use characterization that can result from improved spatial resolution, which in turn can lead to much improved estimates of peak runoff rates and runoff volumes. This procedure, published in Lee and Heaney (2003), is described below. Also, a high quality GIS and associated database for Happy Hectares has been developed along with an accompanying tutorial. This information is presented in Appendix that also includes ArcExplorer and MS Access files.

Literature Review on Urban Imperviousness
Imperviousness may be the most critical indicator for analyzing urbanization impacts on the water environment (Schueler, 1994; Arnold and Gibbons, 1996; WEF-ASCE, 1998). The oldest, and still most widely used, method for storm drainage design is the Rational Method that was first introduced by the Irish engineer Mulvaney (1850), the American Kuichling (1889), and the British Lloyd-Davies (1906). The runoff coefficient of the Rational Method is directly proportional to the total imperviousness (EPA, 1983; Schueler, 1994; WEF-ASCE, 1998). While the American Rational Method uses the runoff coefficient according to rainfall characteristics and total land area, the British Lloyd-Davies method only considers 100 percent runoff from the DCIA (Lloyd-Davies, 1906; Mays, 2001). Thus, the original version of the British Lloyd-Davis formula is $Q = iA_{DCIA}$, instead of $Q = CiA$, where i is the rainfall intensity, A_{DCIA} is the drainage area as DCIA, C is the runoff coefficient, and A is the entire drainage area. However, urban imperviousness is site specific, and complicated to measure. Imperviousness has been estimated as a function of developed population density (Heaney et al. 1977). Novotny and Olem (1994) showed a strong correlation between the total imperviousness of residential areas and the curb length per unit area. Debo and Reese (1995) show a way to adjust the SCS curve number based on the proportion of directly connected impervious area. Schueler (1994) summarized the importance of imperviousness for some components of the urban water environment, such as runoff, stream shapes, water quality, stream warming, and stream biodiversity. He pointed out that transportation-related imperviousness often exerts a greater hydrological impact than the rooftop-related imperviousness. He also presented a relationship between urban land use and imperviousness (Schueler 1995). The City of Olympia (1994) analyzed eleven residential, multi-family and commercial sites to understand urban imperviousness. They found that about 63 to 70 % of total impervious area consists of transportation-related impervious surfaces which are mainly roads, driveways, and parking lots. Booth and Jackson (1997) explained the limitations of using the total impervious area (TIA) in urban hydrology. They suggested using DCIA to characterize urban development. However, they only

mentioned that the direct measurement of it is complicated. Alley and Veenhuis (1983) developed the following empirical relationship between TIA and DCIA from a highly urbanized portion of Denver, Colorado.

$$DCIA = 0.15(TIA)^{1.41}$$ **Equation 3-1**

Dinicola (1989) estimated the percentages of TIA and DCIA for five land use categories. However, the DCIA was not measured directly but estimated based on Equation 3-1, and the right-of-way area was not treated as a separate land use category. Boyd et al. (1993 and 1994) analyzed many rainfall-runoff data sets to predict pervious and impervious runoff. They considered two stages of rainfall-runoff phenomena: only impervious runoff from small storms and total runoff from both impervious and pervious areas associated with larger storms. They assumed that the DCIA could be identified by plotting rainfall depth against runoff depth. The R^2 in their results are mostly around 0.85, but they did not measure the impervious area directly. Instead, they used available reference data or measured it based on paper map basin by basin. Hoffman and Crawford (2001) developed a detailed GIS database tools to predict flooding of individual parcels using SWMM in the combined sewer system of Portland, Oregon. They developed very accurate coverages of impervious surfaces that consist of single-family residential (SFR) and commercial building rooftops (C), streets (S), and parking lots (P). In general, 80% of SFR and 100% of C and P are directly connected to the combined sewer system through laterals in their model. About 6% of SFR and 100% of S is considered as DCIA, and runoff from those areas flows to storm inlets. On the whole, they treated 86% of SFR and 100% of the other impervious surfaces as DCIA. That means almost all TIA works as DCIA in their model except 14% of SFR. In larger storm and runoff situations, like flooding, this model worked well. However, they did not measure the actual connectivity of the entire impervious surface, and did not mention driveways and sidewalks as impervious area. Using accurate and large-scale planimetric data, Prisloe et al. (2000) created a very accurate GIS database of impervious surface features that included buildings, roads, driveways, sidewalks, and other constructed impervious landscape features in four towns of Connecticut. They compared the results with traditional land use and/or zoning based imperviousness data, which were originally developed by analysis of satellite imagery for the same area in 1999 (Civco and Hurd). They found two different relationships between the actual imperviousness based on planimetric data and the predicted imperviousness based on satellite imagery. In residential areas, the predicted imperviousness of rural/suburban areas is about 2 to 3% larger than the actual imperviousness, but the predicted impervious of the urban area is about 6% smaller than the actual one. However, it is not a small difference based on the actual value of imperviousness because the actual imperviousness of the residential area is about 2 to 5% in rural/suburban and about 18% in urban. They separated the right-of-way from the private parcels. While their results show accurate determination of imperviousness, those are TIA, not DCIA, because they did not check the connectivity of impervious components.

Minimizing DCIA is a very important component to achieve the intent of a BMP/LID control (Wright and Heaney 2001). To understand the real effects of this practice, very accurate analysis of rainfall-runoff phenomena is essential. Goyen (2000) shows a scale-independent and storm-independent numerical approach to estimating urban runoff. He used high-resolution spatial and temporal information. Roofs, back and front yards, and paved roads are all separated in his parcel-level detailed spatial database. He also used time steps of 30 seconds or less for rainfall and flow data. He set up three drainage models, which separately estimate runoff from roofs, yards, and roads. His modeled runoff matches the gauged runoff very well. Therefore, if accurate micro-scale spatial data are available, it might be possible to analyze the impacts of different levels of the DCIA in the same area more accurately. Runoff from one impervious surface may flow through another impervious or pervious area to the drainage system. SWMM has recently been updated to model this kind of more complicated overland flow routing (Huber 2001).

The results of detailed measurements of TIA and DCIA for a small urban catchment in Boulder, Colorado and the associated impact on stormwater runoff estimates are presented next.

Detailed Spatial Analysis of Urban Imperviousness
Urban development can have a major impact on the local hydrology and water environment. It changes the rainfall-runoff relationship and affects the receiving water quality. Higher levels of impervious surfaces result in a higher volume of runoff with higher peak discharge, shorter travel time, and more severe pollutant loadings. Urban

imperviousness consists of transportation-related surfaces and rooftops of buildings. Direct connectivity to the drainage system is another critical attribute of urban imperviousness, especially for average storms. While DCIA is one of the most critical parameters for many models, the direct measurement of DCIA is complicated. In spite of the importance of urban imperviousness, few accurate analyses have been done. Remote sensing techniques have been applied to analyze urban imperviousness in many studies, but the spatial resolution and tree canopy of the imagery limit its accuracy. Most available data about urban imperviousness is based on land use or zoning, using image-processing techniques with satellite or airborne imagery. But this spatial resolution and accuracy may be inappropriate for micro-scale urban stormwater analysis. In this study, spatial analysis of urban imperviousness was performed for a three-block residential neighborhood in Boulder, Colorado using GIS and field investigations. The analysis is performed at five levels of accuracy to show the impact of improved accuracy on the estimated downstream runoff hydrograph for a 1-year storm. Runoff is estimated using SWMM 4.4h (Huber 2001).

Description of the study area

Figure 3-1 shows the 5.81 hectare (14.36 acres) three-block study area in Boulder, Colorado. It consists of 59 residential lots, about 210 meters of four-lane streets and about 1,130 meters of two-lane streets. Wonderland Creek, a small ephemeral creek, flows from west to east through this site. There are more residential areas in the upstream portion of the Wonderland Creek watershed, and it connects to the Rocky Mountain foothills. About 22.5% of the study area is in the right-of-way as public land, and the other 77.5% is private land: 60.9% as private lots and 16.6% as private shared land (see Table 3-1). The private shared land is owned by the home owners association, and consists of grassland, small playgrounds, the creek, and creek-side narrow wetlands. The private lots consist of buildings, driveways, and pervious landscaping areas. The surface of the public land is covered by street pavement, some driveways, sidewalks, and pervious vegetated strips.
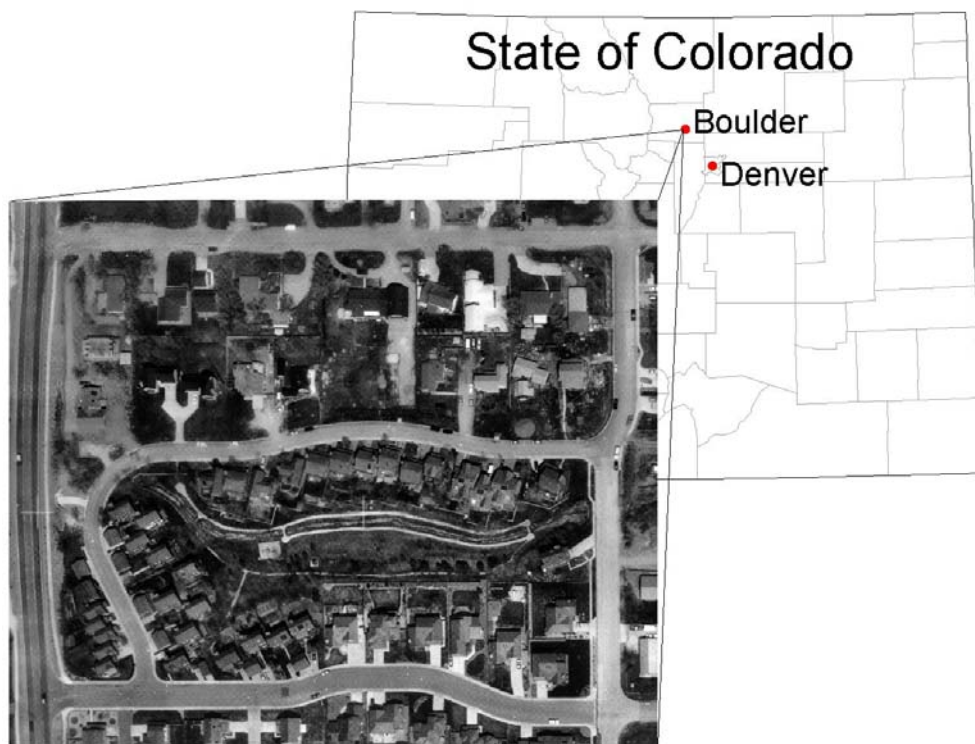
Figure 3-1 Aerial orthophoto of the Wonderland Creek study area.

Table 3-1 General description of Wonderland Creek sub-blocks.

| Sub-block | Area (m^2) | | | | | Lots | | Land use |
| | Private | | Public ROW | | | | Size | |
	Lot	Shared	(m^2)	(%)$^{(1)}$	Sum	Count	(m^2/lot)	
North	19,242	1,996	6,347	23.0%	27,585	18	1,069	Low density residential
South	14,897	7,365	4,620	17.2%	26,881	35	426	Medium density residential
Southwest	1,243	289	2,120	58.0%	3,652	6	207	High density residential
Total	35,382	9,650	13,087	22.5%	58,119	59	600	Single family residential

$^{(1)}$ % of each sub-block area

Flow in Wonderland Creek is highly regulated by a reservoir located a few blocks upstream of the study area. Flow rates were not monitored in this study. The study area consists of low to high density residential areas, with an average lot size of about 600 square meters. For purposes of this study, the area was divided into three sub-blocks described in Table 3.1 and shown in Figure 3-2.

The following spatial data sets for this area were collected mostly from the Public Works Department, City of Boulder: aerial orthophoto (image), street centerline (polyline), parcel boundary (polygon), waterbody (polyline), soil property (polygon), 2-foot contours, and planimetric data (CAD drawing).
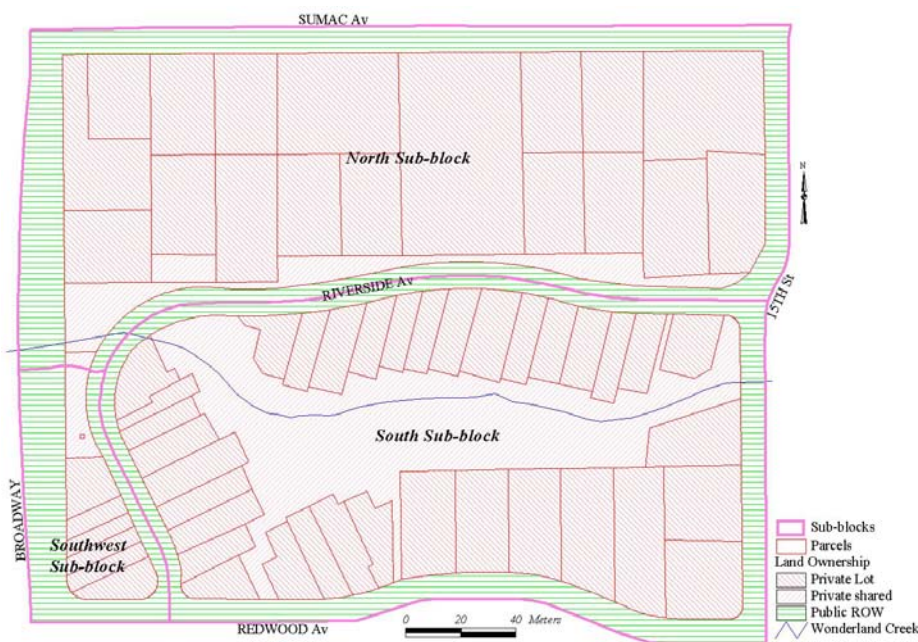
Figure 3-2 Three sub-blocks in Wonderland Creek study area.

Field investigations
A variety of field investigations were performed. First, the accuracy of existing spatial data sets was evaluated and updated as necessary. Next, every individual impervious surface was checked to estimate its hydraulic properties, including connectivity. The pavement material of every street was noted. Streets were classified into those with or without curb and gutter. The curbed streets are directly connected to the storm drainage system, but the streets without curbs drain to adjacent permeable swales. Pavement material and physical connectivity of every sidewalk

and driveway were also investigated carefully. Finally, the locations of every downspout from individual building rooftops were observed to get the actual hydraulic connection rate for each building.

Spatial database analysis
Urban impervious area consists of building rooftops and transportation infrastructure. For this study, it was necessary to update every impervious surface as polygon-type coverage. Polygon coverage for building rooftops is available from the City directly. While most boundary lines of transportation infrastructure are available from the planimetric CAD drawings, they were not ready to use as is. As polygons, each individual impervious surface has more than two different boundary components. For example, most of the driveways share three boundary components with buildings, streets, and pervious landscaping area. Furthermore, there are many errors in the planimetric CAD drawings because they were not usually drawn for engineering applications, but for general management purposes. Many of the edge areas were not connected or overlapped. Using AutoCAD and ArcView, significant revisions and corrections were made to these drawings. The updated and/or developed spatial databases are described below:

- Block boundary (based on the street centerline): Broadway (west), Redwood (south), 15th (east), and Sumac (north)
- Sub-blocks (divided by building density): North, South, and Southwest sub-block
- Paved streets: Street centerline, pavement edge, and street divide line
- Sidewalks: Sidewalk edge and connected street boundary
- Driveways: Driveway edge and connected street and building boundary
- Buildings: No requirement of redrawing

The streets are on public land and the buildings are on private land. However, driveways and sidewalks are partly on public land and partly on private land. Each portion of the driveways and sidewalks was divided by ownership and imperviousness through geo-processing. The attribute data of all coverages, which include ownership, impervious/pervious, hydraulic connectivity, connection rate of each building, etc., were also updated.

Levels of Detail in Estimating Imperviousness
To compare the efforts and the results, five levels of detail were applied in estimating imperviousness. The levels were classified by the detail of applying GIS and field investigations. Approaches for each level of analysis are summarized below.

- Level 1 (no GIS application): Literature reference data (Urban Drainage and Flood Control District 2001) was applied.
- Level 2 (GIS application): All paved streets, sidewalks, driveways, and buildings are classified as impervious surfaces.
- Level 3 (GIS application): Every indirectly connected impervious area (ICIA) is subtracted from the result of the Level 2 analysis. DCIA was initially derived by just using GIS resources.
- Level 4 (GIS application and field investigation for the right-of-way): Paved streets with curb are classified as DCIA and others as ICIA. All the other directly connected impervious surfaces in the Level 3 analysis are re-classified according to whether or not they are directly connected to the paved streets with curbs (DCIA).
- Level 5 (GIS application and field investigation for the entire area): The ground locations of building roof gutter downspouts are investigated to determine which rooftops are directly connected. If a downspout is located on DCIA, it is counted as directly connected. If there are four downspouts from one roof and one outlet is located on the directly connected surface, then 25 % of the rooftop is considered as DCIA.

The required effort for each level of detail is summarized in Table 3-2. The required time depends on the condition of available data sets and the analyst's skill level. The more time consuming tasks include CAD file redrawing, field investigation, and spatial and attribute data manipulation. In most engineering applications, the reference imperviousness data is usually used, like the Level 1. While it requires the least effort, it can cause significant errors. GIS analysis and field investigation were added to improve the accuracy of imperviousness estimation. Based on the

summary of effort, GIS related in-house work with only currently available digital data sets took about 50% of the effort. Field investigation and related data update took about another 50% of the whole effort: about 30% for public land and about 20% for private land. The required effort could be reduced significantly if the applied GIS database has suitable street pavement coverages with high accuracy in this particular study.

Table 3-2 Level of effort for the five levels of detail (hrs).

Item	Level 1	Level 2	Level 3	Level 4	Level 5
Raw Data Acquisition	1	20	20	20	20
AutoCAD Redrawing	0	40	40	50	50
Field Investigation	0	0	0	25	40
Attribute DB Development	0	4	12	20	30
ArcView Analysis	0	15	25	40	55
Sum	1	79	97	155	195

Results of Detailed Spatial Analysis

The results of the spatial analysis were arranged and discussed for each sub-block and surface component for the five levels of detail. The summary of surface components of the study area by their ownership is presented in Table 3-3. The actual TIA of the site is 20,858 square meters (35.9% of total area), and consists of streets (35.9%), driveways (21.6%), sidewalks (6.6%), and buildings (36.0%). That means about 64.0% of the TIA belongs to transportation-related imperviousness. In land ownership, 44.9% of the TIA is located on public land and the other 55.1% is located on private land. About 25.5% of private land is covered by impervious surfaces, and 71.5% of public land is covered by impervious surfaces.

Table 3-3 Summary of surface components of the study area.

Category	Component	Private land		Public land		Sum	
		(m^2)	$(\%)^{(1)}$	(m^2)	$(\%)^{(1)}$	(m^2)	$(\%)^{(2)}$
Pervious	Soil/Grass	33,530	74.5%	3,731	28.5%	37,261	64.1%
Impervious	Streets	0	0.0%	7,478	57.1%	7,478	12.9%
	Driveways	3,915	8.7%	595	4.5%	4,510	7.8%
	Sidewalks	86	0.2%	1,282	9.8%	1,368	2.4%
	Buildings	7,501	16.7%	0	0.0%	7,501	12.9%
	Sub-total	11,502	25.5%	9,356	71.5%	20,858	35.9%
Total		45,032	100%	13,087	100%	58,119	100%

[1] % of each ownership   [2] % of the total area

The final result of detailed spatial analysis is presented in Table 3-4 and shown in Figure 3-3. Every individual surface component was analyzed and summarized spatially and hydrologically.

Table 3-4 Results of detailed spatial analysis of imperviousness as DCIA (m^2).

Component	North Sub-block				South Sub-block			
	Level 2	Level 3	Level 4	Level 5	Level 2	Level 3	Level 4	Level 5
Streets	3,190	3,190	1,443	1,443	3,060	3,060	3,060	3,060
Driveways	3,248	3,248	16	16	1,186	1,186	1,186	1,186
Sidewalks	199	199	0	0	876	876	792	792
Buildings	3,306	1,895	0	0	3,860	2,938	2,938	215
Sub-total	9,943	8,531	1,459	1,459	8,982	8,059	7,976	5,253
Component	Southwest Sub-block				Total Area			
	Level 2	Level 3	Level 4	Level 5	Level 2	Level 3	Level 4	Level 5
Streets	1,229	1,229	649	649	7,478	7,478	5,152	5,152
Driveways	75	75	75	75	4,510	4,510	1,277	1,277
Sidewalks	293	293	141	141	1,368	1,368	933	933
Buildings	335	190	190	0	7,501	5,022	3,127	215
Sub-total	1,933	1,787	1,055	865	20,858	18,378	10,489	7,576

Figure 3-3 Detailed spatial analyses of Wonderland Creek study area.

The hydraulic connectivity of the impervious surfaces was reduced significantly based on the detailed spatial analysis. In the Level 5 analysis, 68.9% of streets, 28.3% of driveways, 68.2% of sidewalks, and only 2.9% of the entire

building rooftops are classified as DCIA. On the whole, the percentage of the actual DCIA has changed dramatically, from 35.9% in the Level 2 analysis to 13.0% in the Level 5 analysis. The rooftop-related DCIA is only 2.8% of the entire DCIA, and that means about 97.2% of the DCIA belongs to transportation-related imperviousness in the most accurate spatial analysis. The street pavement surface is the largest portion of the DCIA, i.e., 68.0% of the whole DCIA in the Level 5 analysis.

The imperviousness estimates from the Level 1 to Level 5 are shown in Table 3-5. The result of the Level 2 analysis is the TIA of the study area, and that of the Level 5 is the most accurate DCIA. Imperviousness from the literature and the actual TIA and DCIA were compared in Figure 3-4. The literature data in the Level 1 analysis show close numbers to the TIA in the Level 2. But, large differences exist between the literature data and the DCIA in the Level 5.

Table 3-5 Directly connected imperviousness for each level of analysis.

Sub-block	Level 1	Level 2	Level 3	Level 4	Level 5
North	30.0%	36.0%	30.9%	5.3%	5.3%
South	40.0%	33.4%	30.0%	29.7%	19.5%
Southwest	52.5%	52.9%	48.9%	28.9%	23.7%
Total area	36.0%	35.9%	31.6%	18.0%	13.0%
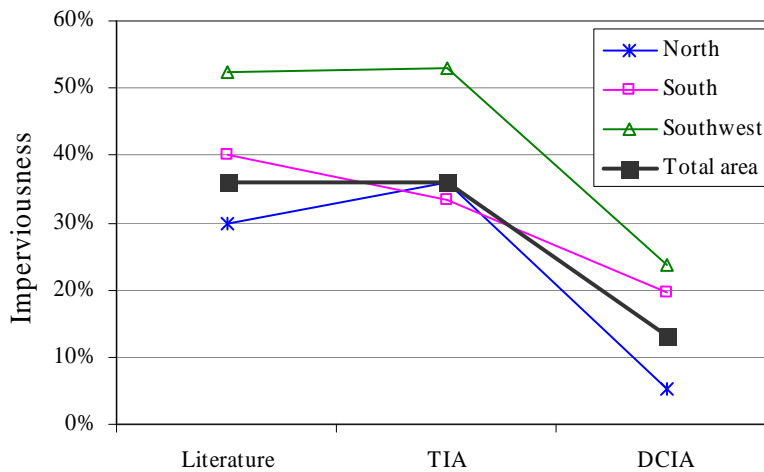
Figure 3-4 Comparison of Imperviousness: Literature, TIA, and DCIA.

Impervious area is re-arranged by surface component for private vs. public land. It is presented in Table 3-6 and Table 3-7. About 36.3% of the TIA is directly connected to the drainage system in the entire study area: about 66.6% of the TIA is public land and only about 11.7% of the TIA is private land.

Table 3-6 Impervious area by surface component for private vs. public land.

	TIA (m^2)			DCIA (m^2)		
	Private	Public	Sum	Private	Public	Sum
Streets	0	7,478	7,478	0	5,152	5,152
Driveways	3,915	595	4,510	1,071	207	1,277
Sidewalks	86	1,282	1,368	58	875	933
Buildings	7,501	0	7,501	215	0	215
Sum	11,502	9,356	20,858	1,343	6,233	7,576

Table 3-7 Proportion of impervious area by connectivity for private vs. public land.

	Private land		Public land		Sum	
	(m^2)	(%)	(m^2)	(%)	(m^2)	(%)
DCIA	1,343	11.7%	6,233	66.6%	7,576	36.3%
ICIA	10,159	88.3%	3,123	33.4%	13,281	63.7%
TIA	11,502	100%	9,356	100%	20,858	100%

The proportions of imperviousness by impervious surface components and land ownership are shown in Figure 3-5. Based on the TIA, streets and buildings show very similar proportions, and also private and public TIA estimates are similar. Based on the DCIA, however, buildings contribute only 2.8% of the DCIA, and streets contribute 68.0% of that amount. Transportation-related imperviousness is about 64.0% of the TIA and 97.2% of the DCIA. In a micro-storm situation, almost all runoff might be contributed by the transportation-related imperviousness. From the perspective of land ownership, 82.3% of the DCIA belongs to public land. Public land inside the right-of-way may be the most important area to manage urban imperviousness, and street pavement surface is the most significant one.
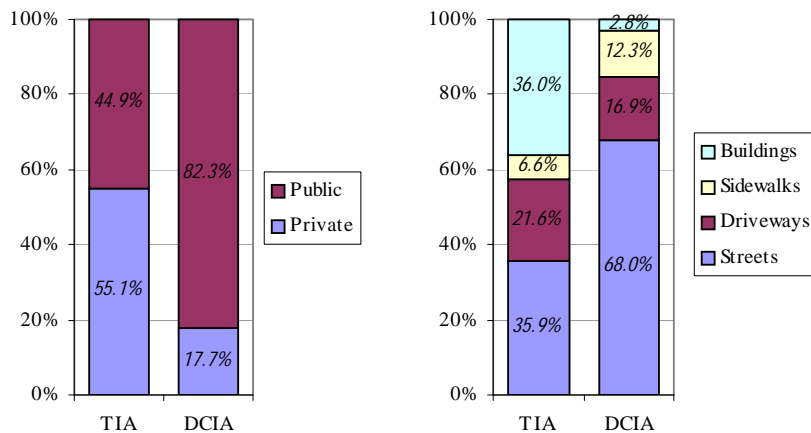
Figure 3-5 Proportion of imperviousness by ownership and surface component.

Example Showing the Importance of Accurate GIS Data

This example, taken from Lee and Heaney (2003), funded by this EPA contract, demonstrates the importance of accurate GIS information in evaluating peak discharges for storm sewer design. Hydrologic simulation was performed using SWMM 4.4h (Huber 2001) to compare the effect of the five different DCIA levels on the predicted runoff hydrograph from a single precipitation event. The time of concentration of the area is about one hour based on travel time calculations for the entire study area. From the newly developed IDF curve using software described in Chapter 4, a small design storm of 18 mm total rainfall depth with 1 hour of duration is applied to the model. It is about a 1-year return period storm. Other modeling parameters, such as depression storage and infiltration parameters, are adapted from the revised Urban Storm Drainage Criteria Manual based on the physical conditions of the study site (Urban Drainage and Flood Control District 2001). Each sub-block was divided into three sub-catchments: the DCIA, the PA, and the ICIA. Thus, there are nine sub-catchments in the study site for the SWMM modeling. The predicted runoff hydrographs are shown in Figure 3-6.
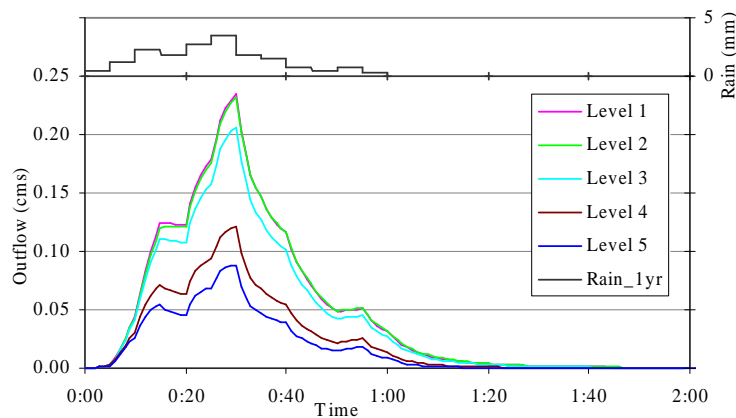
Figure 3-6 Predicted runoff hydrographs for the five levels of accuracy in estimating imperviousness.

The modeling results show quite large differences for the five levels of accuracy. The relationship between the DCIA and the runoff hydrographs is shown in Table 3-8 and Figure 3-7. It shows a linear relationship because the entire runoff is contributed by DCIA. For the same reason, the proportional significance of the Level 1 and the Level 5 analysis indicate very similar results: about 276% of DCIA, about 265% of peak discharge, and about 275% of the total runoff volume. The result of the Level 5 analysis provides the most accurate DCIA. Thus, the drainage system would be over-designed using the default reference data. Of course, the significance of peak flow and total volume between the Level 1 and 5 would change if a larger storm is analyzed.

Table 3-8 Imperviousness (DCIA) and runoff for the five levels.

	Level 1	Level 2	Level 3	Level 4	Level 5
DCIA (%)	36.0%	35.9%	31.6%	18.0%	13.0%
Peak (m^3/s)	0.235	0.232	0.207	0.121	0.089
Total Vol (m^3)	361.0	359.4	316.9	181.3	131.3
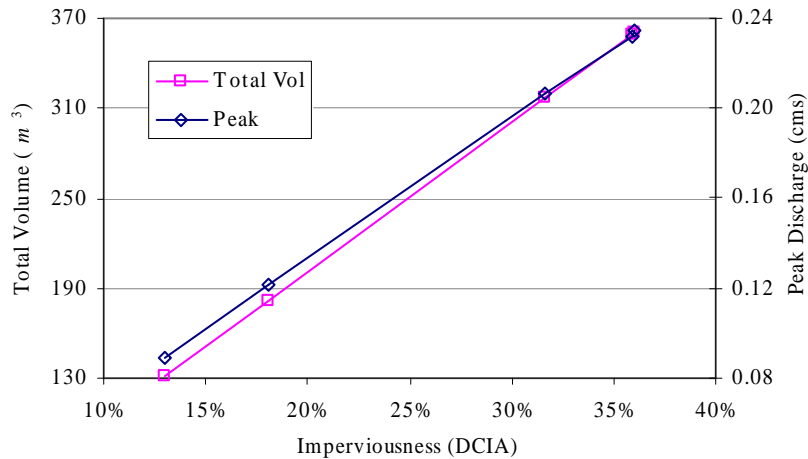
Figure 3-7 Imperviousness (DCIA) and runoff for the five levels.

Both the conventional and Lloyd-Davies Rational methods were applied to compare to the SWMM simulation results. The SWMM results are well matched to both methods with less than five percent error. However, the Lloyd-Davies method needs a simple multiplication between the rainfall intensity and the DCIA while the conventional method needs a very careful selection of suitable runoff coefficients. In this example, the peak design flow using conventional land use analysis is about 2.5 times larger than the more accurate estimate based on detailed GIS evaluations. Thus, a significant savings in storm sewer infrastructure costs could be realized if these improved evaluation methods are deployed.

Conclusions
The results of a careful determination of the nature of imperviousness in urban areas indicate that existing estimates can be very inaccurate. The primary findings of this study are listed below:

- The conventional Rational Method for estimating peak discharges (Q) for sewer design may provide inaccurate estimates. In the formula, $Q = CiA$, C is the runoff coefficient, i is the average rainfall intensity associated with the time of concentration, and A is the drainage area. Much of the debate in the literature is about how to select the average rainfall intensity, i. However, the analysis shown above indicates the large potential error in the estimate of CA if conventional design methods are used. Furthermore, the Lloyd-Davies formula using accurate DCIA could be better than the conventional Rational Method formula for evaluating urban storm drainage systems.
- This analysis shows the critical importance of the contributions from transportation-related activities in the right-of-way.
- Evaluation of decentralized control systems such as LID needs to incorporate explicit recognition of DCIA as a separate source of imperviousness and the role of the right-of-way as a major source.
- Curb and gutter drainage is the major source of DCIA. Alternatives such as swale drainage that minimize the concentration of stormwater can only be accurately evaluated if the nature and importance of DCIA is quantified for proposed developments.
- Modern tools such as GIS make it much easier to accurately describe the true nature of the imperviousness associated with urbanization.

Chapter 4 Long-term Precipitation Data Analysis

Introduction

Traditionally, a design event approach has been used for designing stormwater infrastructure. Continuous simulation, using one or more years of data, has been recommended for better stormwater management and pollution control. Long-term precipitation data is one of the most important data sets in urban stormwater analysis. Precipitation intensity, duration, and frequency analysis can be performed using these data sets. While precipitation data are very critical, it is hard to obtain suitable data sets that have a long period of record and reasonable resolution in time and monitoring depth. Hydrologic response time, e.g., the time of concentration, is much faster in urban areas. Thus, small time-step precipitation data is essential for developing intensity-duration-frequency (IDF) curves to analyze single design events and to perform continuous simulation. Easy access to precipitation data in electronic form is vitally important for doing continuous simulations. This chapter presents spreadsheet-based methods to extract precipitation data from national data sources and perform a variety of analyses that are essential parts of evaluating wet-weather controls options. Three precipitation data analysis methods were developed using spreadsheets and Visual Basic Application (VBA). They include a method for event-based synoptic analyses, a continuous disaggregation procedure, and a method for developing IDF curves and are described in the following sections.

NCDC Precipitation Data

Local precipitation data can be obtained from the National Climatic Data Center (NCDC) climate data as CDs or downloaded directly from their website (http://www.ncdc.noaa.gov). Hourly and 15-minute data sets are available for most areas of the United States. There are a couple of different formats in NCDC data. Time series format of event data is appropriate for applying them to the Precipitation Data Analyzer that is described below. In this case, an event refers not a storm event but a non-zero precipitation record. How to download or export data is explained in their CDs and website. Much of the NCDC's hourly data has been monitored since 1940s and 15-minute data since 1970s. These data are reported in 0.01 or 0.10 inch increments. Better local data may be available in some areas but the rest of this chapter is based on NCDC data.

As an example, two precipitation data sets are available for Boulder, Colorado: 52 years of hourly data and 30 years of 15-minute data (NCDC 2003). This rainfall data was first analyzed by the SWMM Synop program. Based on a minimum inter-event time (MIT) of six hours, the average annual precipitation is about 486 mm with a range from 300 mm to 719 mm. An average of 69 precipitation events occurs annually. The objectives of the revised precipitation analyzer are to:

1. Develop a spreadsheet version of Synop for 15 minute and hourly data.
2. Extend the precipitation analysis to provide a time series of precipitation for intervals as short as one minute.
3. Develop spreadsheet software to generate IDF curves using this data.

The methods for doing this analysis are described in the next sections.

Data status and pre-treatment

Each record of the NCDC precipitation data includes station number, start time, depth, and a flag based on data status. There are three kinds of flags for a record: A-accumulating, M-missing, and D-deleting. If there is no flag, the record is a well-monitored data point. Several steps of data pre-treatment, which are required for further analysis, are summarized below:

Step 1. Select a period of good data
If there are many flags within a certain period, it should not be selected.

Step 2. Clean flagged data
The flagged data need to be cleaned to make a good data set for further analysis. A reasonable way to do it may be to delete all flagged data. Data with "A" are accumulating values during several time steps and data with "M" or "D" are all zero values.

Step 3. Convert start time
Text formatted start time for each record needs to be converted into a real number format. An example of converting a start time is presented below:

$$Start\ time = Date + Hour / 24 + Minute / (24 \times 60) \qquad \textbf{Equation 4-1}$$

After pre-treatment, the selected data need to be copied to the main worksheet of the data analyzer like Figure 4-1. An example of pre-treated input data is also shown in Figure 4-1. The dates shown in column A are stored as numbers, not text, so arithmetic operations can be performed on them.

Figure 4-1 Analyze worksheet of precipitation data analyzer (Prcp Analyzer.xls).

Precipitation Data Analyzer

The main worksheet of the precipitation data analyzer (sheet name: "Analyze") is shown in Figure 4-1. To perform each analysis, simply click each button to perform SS Synop, time series disaggregation, or IDF curves. Each of these methods is described below.

Event-based synoptic analysis

SS synoptic analysis is an event-based statistical analysis of long-term precipitation data using a spreadsheet as the computing platform. It is similar to the SWMM Rain module and the EPA's Synop software. Rainfall events are separated by dry time, which means it has not rained for that period. This dry period for defining a storm event is called a minimum inter-event time (MIT) (Huber and Dickinson 1988) or inter-event time definition (IETD) (Adams

and Papa 2000.)

Definitions

Given a precipitation record reported in equal intervals, Δt, as shown in Figure 4-2, the following definitions will be used. To calculate the exact time spans of duration and inter-event time, Δt is added and subtracted in Equations 4-3 and 4-5 because every record has been reported using a start time of each pulse.

- Volume (*V*): the sum of the rainfall depths of each sampling interval for an event

$$V = \sum v \qquad\qquad \textbf{Equation 4-2}$$

- Duration (*T*): the period from the beginning of the first rainfall pulse to the end of the last rainfall pulse in a storm event

$$T = (te_i - tb_i) + \Delta t \qquad\qquad \textbf{Equation 4-3}$$

- Average intensity (*i*)

$$i = V / T \qquad\qquad \textbf{Equation 4-4}$$

- Inter-event time (*IT*): the period from the end of the previous event to the beginning of the current event

$$IT = (tb_i - te_{i-1}) - \Delta t \qquad\qquad \textbf{Equation 4-5}$$

- *MIT*: a minimum period between consecutive pulses of rainfall to separate individual storm events where *MIT* $= n\Delta t$ and *n* is the number of time steps used to define *MIT*. For example, if $\Delta t = 5$ minutes and $n = 60$, then *MIT* = 300 minutes or 5 hours.
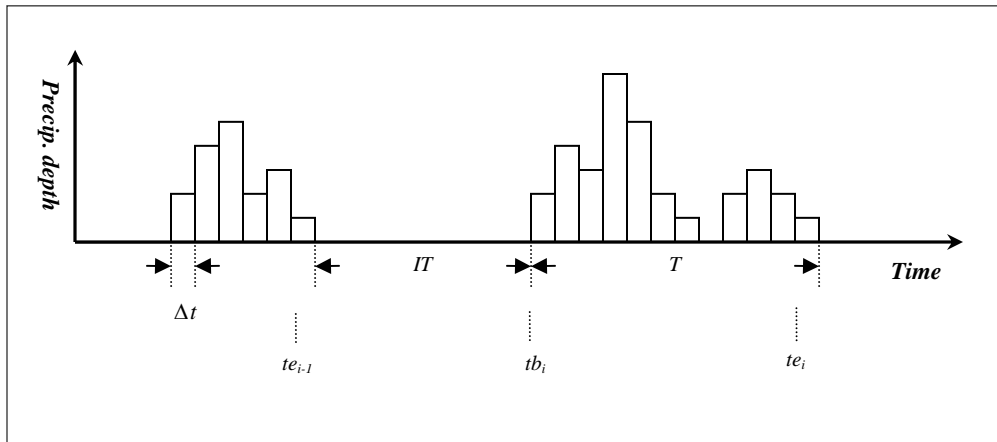
Figure 4-2 Defining storm events.

SS synoptic procedure

When the <Synop> button is clicked (see Figure 4-1), it automatically pops-up the window shown in Figure 4-3. In this window, a user needs to define the input data time step and IETD, and then click the <Synop> button in the window to perform synoptic analysis. Some suggestions for selecting hourly or 15-minute data, and the IETD are presented below.
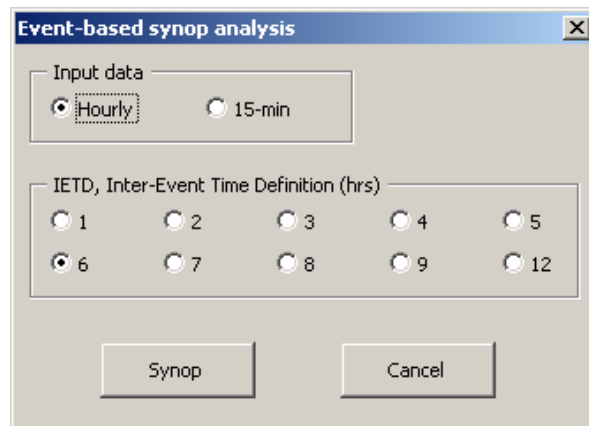
Figure 4-3 Pop-up window for SS synoptic analysis.

The selected time steps for the analysis should be based on the expected residence time of a parcel of water moving through the system of interest. The total residence time, t, is the sum of the overland flow time, t_{of}, the time of flow through transport systems such as sewers, t_t, and the time spent in a wet-weather control, t_{wwc}, i.e.,

$$t = t_{of} + t_t + t_{wwc} \hspace{4cm} \textbf{Equation 4-6}$$

Much of the literature on the statistics of urban stormwater, e.g., Adams and Papa (2000), is based on hourly precipitation data and inter-event times of several hours. These selected times reflect the historical interest in centralized controls wherein travel times were of the order of hours. However, these intervals are too long for smaller catchments with high-rate controls. The time interval for the precipitation data should be only about 20-50% of t, as calculated above. For example, a high-rate treatment device has a residence time of about 30 minutes. Thus, input data with a 10-15 minute frequency are needed to properly analyze the performance of this WWC.

SS Synop analysis rearranges the precipitation data for each event. Each event will be listed with its duration, depth, intensity, and inter-event time in a separate worksheet named "Synop." An example SS Synop result is shown in Table 4-1.

Table 4-1 Example of event-based SS Synop result.

Storm	Start	Dur (hr)	Vol (in)	Int (in/hr)	IT (hr)
338	2001/05/02 02:15	5.25	0.4	0.0762	227.5
339	2001/05/02 20:15	27	0.7	0.0259	12.75
340	2001/05/04 10:45	26.5	1.2	0.0453	11.5
341	2001/05/17 12:30	6.25	0.2	0.032	287.3
342	2001/05/20 13:00	11	0.5	0.0455	66.25

Dur: duration; Vol: volume; Int: intensity; and IT: inter-event time

Disaggregation procedure

Several approaches to disaggregate hourly precipitation data to shorter time steps have been developed (Ormsbee 1989; Durrans et al. 1999; Burian et al. 2000; Burian et al. 2001). Ormsbee (1989) proposed continuous deterministic and stochastic disaggregation models and the others described a polynomial-based approach and artificial neural network based models. Among those methods, Ormsbee's continuous deterministic disaggregation procedure was

applied in this project because most of the other methods need training or sample data sets with smaller time steps for developing the models and these smaller interval data may not be available. The basic assumption of the Ormsbee's method is that the distribution of precipitation within a time-step is proportional to the distribution of precipitation over the three time-step sequence with adjacent before and after time steps (Ormsbee 1989). Based on this linear assumption, precipitation data can be disaggregated into smaller time steps, i.e., 1, 2, 3, 5, 10, 15, 20, or 30 minutes. This feature is particularly important for the small catchments that are associated with studies of LIDs.

When the <DisAgg> button is clicked (see Figure 4-1), it automatically pops-up the window shown in Figure 4-4. In this window, a user needs to define the input and output data time steps, and output pulse depth, and then click <Disaggregate> button in the window to perform disaggregation.
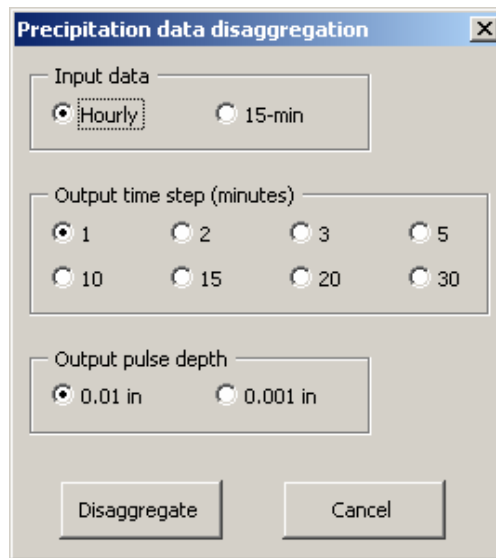
Figure 4-4 Pop-up window for disaggregation.

Disaggregated precipitation data will be listed in the same worksheet. An illustrative disaggregation result is shown in Table 4-2. This disaggregation data can be applied to perform shorter time-step continuous simulations and to develop smaller time step IDF curves for design event approaches.

Table 4-2 Example of precipitation data disaggregation with one minute intervals.

Start	Prcp
2001/08/08 11:17	0.001
2001/08/08 11:18	0.003
2001/08/08 11:19	0.002
2001/08/08 11:20	0.002
2001/08/08 11:21	0.002
2001/08/08 11:22	0.003

Prcp: precipitation depth (in)

IDF analysis

In design event approaches, a design storm can be selected from the local IDF curves with a certain time of concentration (or duration) and return period. A local IDF curve may be available, but it may exclude either shorter durations (e.g. less than 60 minutes) or more frequent events (e.g. less than 2 year return period). However, frequent small storms with short durations and return periods are very important for urban stormwater management because of the fast hydrologic response time in urban areas. For this reason, an IDF analysis procedure was developed using disaggregated data. The 1-hr or 15-minute NCDC precipitation data must be disaggregated before doing the IDF analysis.

The empirical return period is calculated using the following general equation, (Gringorten 1963; Cunnane 1978), which is used in the SWMM Rain block (Huber and Dickinson 1988):

$$Ret = (Yrs + 1 - 2A) / (Rnk - A) \qquad \textbf{Equation 4-7}$$

where Ret = return period in years; Yrs = number of years of data; Rnk = rank of event (ranked in descending order); and A = parameter for plotting position.

A value of 0.4 for parameter A was suggested as a good compromise for US customary units by Cunnane (1978), and this value is adopted for this IDF procedure.

When the <IDF> button is clicked (see Figure 4-1), it runs automatically with disaggregated precipitation data. Times of concentration (or durations) for IDF analysis are automatically assigned based on the time step of the applied disaggregation data. The results are shown in a separate worksheet named "IDF." In this worksheet, up to 1,500 ranked precipitation depths will be listed in descending order based on significance of precipitation intensity for each time of concentration. Based on these lists, an IDF table will be created to develop IDF curves. Examples of the IDF lists and IDF table are shown in Tables 4-3 and 4-4.

Table 4-3 Example of IDF lists.

Rank	Return Period	Start	Precip.	Intensity
1	51.45	1997/07/26 08:20	0.555	6.66
2	19.29	1999/08/17 09:29	0.538	6.456
3	11.87	1993/06/26 22:41	0.537	6.444
4	8.58	1995/07/26 08:12	0.488	5.856
5	6.71	1997/09/06 18:52	0.485	5.82

Precip.: precipitation depth (in.); Intensity: (in./hr)

Table 4-4 Example of a developed IDF table (intensity in in/hr).

Duration	Return Period				
(min)	2 yrs	1 yr	6 mth	3 mth	1 mth
5	4.125	3.333	2.916	2.4	1.706
10	3.62	2.935	2.543	2.124	1.53
20	2.357	2.06	1.873	1.611	1.2
30	1.799	1.615	1.496	1.338	1.016
60	1.168	1.13	1.063	0.939	0.696

Because of the linear assumption in Ormsbee's continuous deterministic disaggregation model, the IDF analysis results with durations for about less than one third of the initial input data time step may not be reliable. Thus, long-term 15-minute data need to be applied for developing IDF curves with shorter durations, i.e., from 5 to 60 minutes. Figure 4-5 shows IDF curves that were developed using 31 years of 15-minute data from Boulder, Colorado.

SS synoptic analysis does not take a long computational time, but the disaggregation procedure with smaller output time steps and pulse depths may take a relatively long time. The IDF analysis may need around ten times more computational time than the disaggregation procedure. If several decades of input data are applied, it may take a couple of hours. In this situation, it is better to perform disaggregation and IDF analysis as one procedure. To do this, one more button named <DisAgg-IDF> is included in the lower part of the main worksheet. Once it is clicked and the disaggregation properties are set up (output time step and pulse depth), the disaggregation procedure and IDF analysis are performed sequentially. In this case, the actual computational time should be the same, but two procedures can be done with one button.
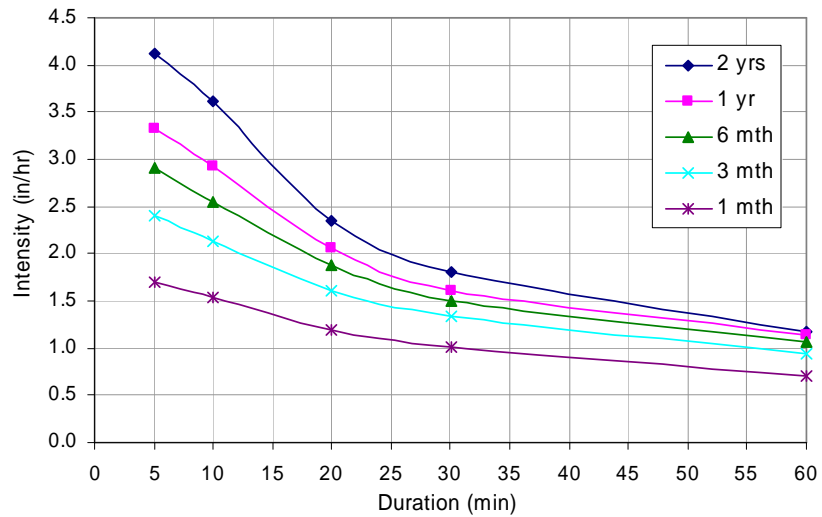
Figure 4-5 IDF curves for Boulder, Colorado based on 31 years of 15-min data.

Finding optimal parameter estimates for IDF curves
It is convenient to replace IDF curves with an equation that accurately represents them. Two questions need to be addressed:

1. What form of equation should be used?
2. What are the best parameter estimates for this equation?

Equation 4-8 shows how IDF data for a variety of return periods can be fit to a single equation. Based on rainfall intensities and recurrence intervals from IDF analysis (see Table 4-4), parameters of *a*, *b*, and *c* can be decided using the Solver optimization procedure in Excel.

$$i = \frac{a + b\ln(r)}{(t + c)}$$

Equation 4-8

where i = average intensity (in./hr); r = recurrence interval (yrs); and a, b, c = parameters.

SS Synop with the disaggregation procedure and the ability to generate IDF curves has many applications in analysis of urban wet-weather flows. The next section describes an example that demonstrates some of the uses of this software.

Illustrative Application of Long-term Precipitation Data

This section presents an illustration of how the precipitation data analysis software can be used for evaluating the effect of better GIS data on the estimated peak flow. These results are taken from Lee and Heaney (2003) supported in part by this contract. A high quality wet-weather quantity and quality database at a high-density residential area in Miami, Florida was analyzed to evaluate rainfall-runoff relationships, and how they are affected by directly connected impervious area as discussed in Chapter 3. This data was collected by the U.S. Geological Survey (USGS) in the 1970's (Miller 1979), and remains one of the best databases in the world for evaluating rainfall-runoff relationships (Lee and Heaney 2002). The Kings Creek site is a 5.95 hectare drainage basin that is part of an apartment complex in South Florida. The DCIA is 2.62 hectares (44.1%). Rainfall and runoff data for 16 storms were reported at 5 minute intervals.

The rainfall database at the Miami WSCMO Airport, Miami, Florida covers the period from August 1948 to December 2000 with a 1-hour frequency. A rainfall event is assumed to end if it hasn't rained for six consecutive hours. The cumulative density function for the rainfall events is shown in Figure 4-6. This cumulative distribution is easy to obtain using SS Synop to separate the hourly data into precipitation events. Event based rainfall depths are plotted against the percent of the rainfall events that are less than or equal to the indicated value. For example, about 91% of the rainfall events are less than or equal to 30 mm in total depth. Typical drainage designs use the 2 to 10 year recurrence interval for their evaluation. However, as is shown in Figure 4-6, events with a recurrence interval of less than or equal to one month comprise over 90% of the total rainfall that occurs in Miami. Thus, control of these frequent events is the most critical component of urban stormwater quality management.
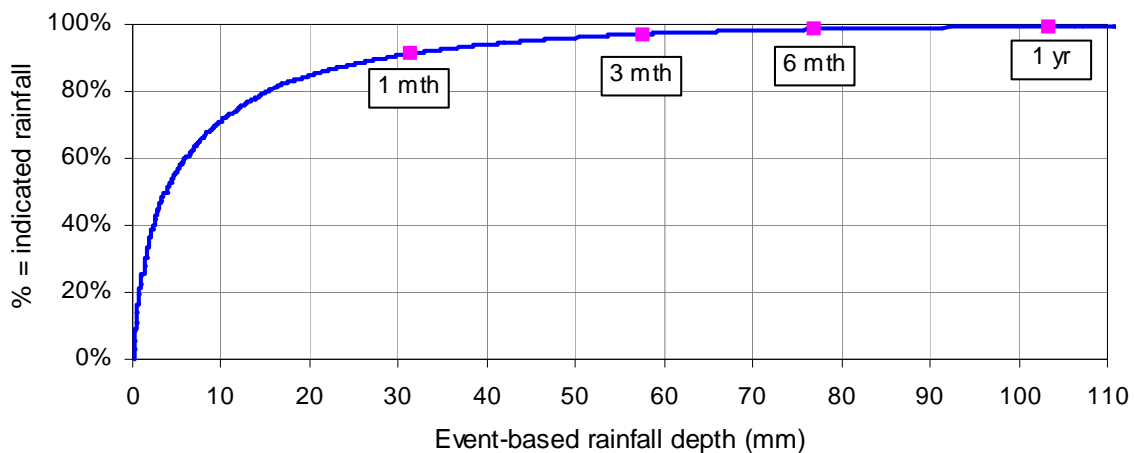
Figure 4-6 Event based rainfall depth in Miami, Florida: 1948-2000.

For developing a runoff depth estimation model, runoff was analyzed as a function of rainfall using the USGS rainfall-runoff database. To calculate excess rainfall, 2.54 mm of initial abstraction (or depression storage) is assumed for both impervious and pervious areas. Total runoff can be estimated by combining runoff from DCIA and other areas. The runoff from the Kings Creek site was separated into two components as shown in Equation 4-10 and 4-11. The developed rainfall-runoff estimation model is shown in Figure 4-7.

$$Total\ Runoff = DCIA\ Runoff + Other\ Runoff \qquad\qquad \textbf{Equation 4-9}$$

$$DCIA\ Runoff = DCIA(Excess\ Rainfall) = 0.441(Excess\ Rainfall) \qquad \textbf{Equation 4-10}$$

$$Other\ Runoff = 0.3636\,(Excess\,Rainfall) - 5.542 \qquad\qquad \textbf{Equation 4-11}$$
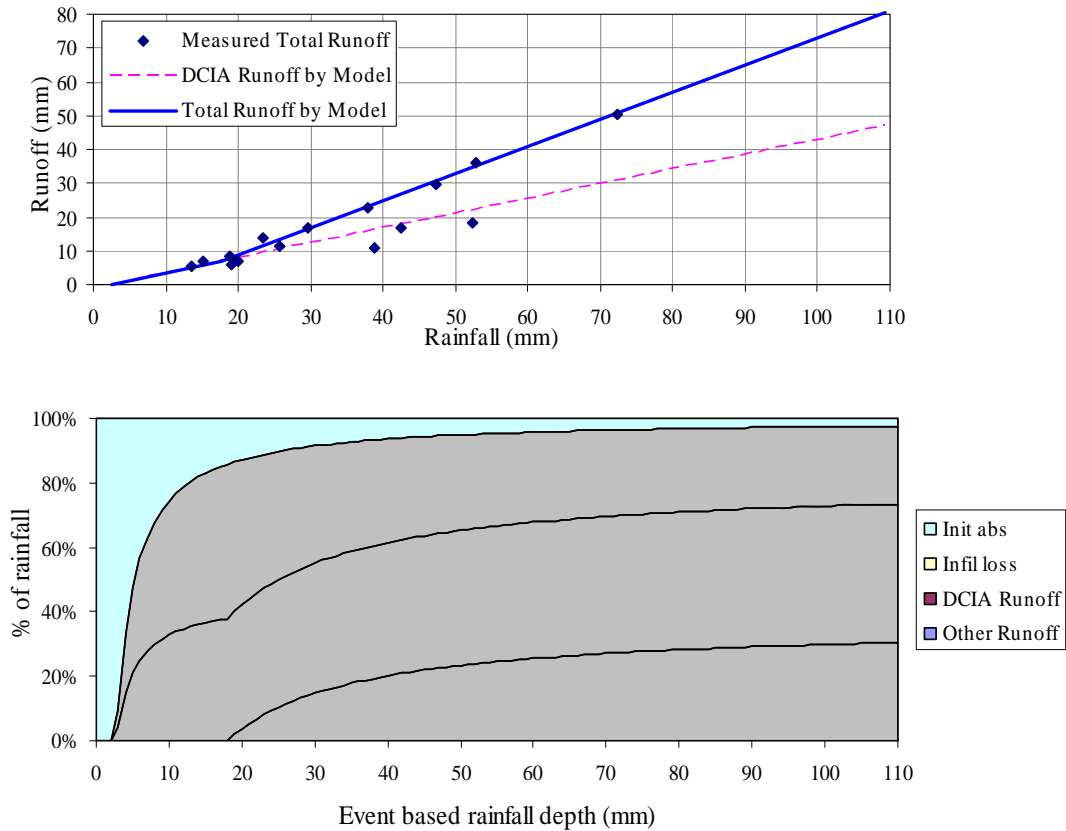
Figure 4-7 Rainfall runoff relationship and long-term analysis in Miami, Florida.

About 50 years of long-term rainfall data are applied to the runoff depth estimation models. The one-hour rainfall data was collected from August 1948 to December 2000 at the Miami WSCMO Airport, Miami, Florida (NCDC 2003). The data are reported to the nearest 0.254 mm. DCIA runoff and other runoff are estimated by developing rainfall-runoff models. The infiltration loss is calculated by subtracting the initial abstraction, DCIA runoff and other runoff from the total rainfall. The total rainfall is partitioned into the four components shown in Equation 4-12.

$$Rainfall = Initial\ Abstract + Infiltration\ Loss + DCIA\ Runoff + Other\ Runoff \qquad \textbf{Equation 4-12}$$

Table 4-5 Results of long-term hydrologic analysis in Miami, Florida.

	Events	Depth (mm)
Rainfall	7,204	77,809
Runoff	4,098	39,162
DCIA Runoff only	2,869	28,256
Non-DCIA Runoff	1,229	10,905
Initial Abstraction		13,709
Infiltration Loss		24,938

The results of the long-term hydrologic analysis are shown in Figure 4-7 and Table 4-5. Using the one-month rainfall of 31 mm, about 73% of the runoff is contributed by 44% of DCIA. A total of 7,204 precipitation events occurred during this 52.4 year period. About 43.1% of these events are less than or equal to 2.54 mm. That means 56.9% of the rainfall events contribute runoff from the site based on the applied rainfall-runoff model. Runoff from non-DCIA areas occurs only for the larger storms. Based on runoff created events, about 70.0% of the runoff events are caused by DCIA runoff only. While DCIA represents about 44% for the site, it contributes about 72 % of the total runoff during those 52 years based on runoff depth.

Summary and Conclusions

Three fundamental precipitation data analysis tools are presented in this chapter. Developed spreadsheet-based precipitation data analyzer (Prcp Analyzer.xls) can be an excellent tool for manipulating long-term precipitation records in many WWC applications. The event-based synoptic analysis can be readily done with a number of IETDs. The disaggregation procedure provides an important tool by generating better resolution of rainfall time series data for a variety of applications, e.g., numerical hydrologic simulation tools such as kinematic wave and diffusion wave simulators. This disaggregation procedure can also be applied to develop site-specific IDF curves with smaller time-step and less than 2-year recurrent frequencies, which are not readily available at present.

Chapter 5 Depression Storage and Infiltration

Introduction

Excess precipitation (or effective precipitation) can be obtained by subtracting initial depression storage and infiltration loss from the actual precipitation depth. Initial depression storage depends on the condition of the land surface and infiltration depends on sub-surface soil properties. During dry periods, initial depression storage and infiltration capacity will be recovered by evaporation and extended deep infiltration.

A continuous model (Prcp Analyzer.xls) for computing depression storage and infiltration was developed using MS-Excel. This model for computing excess precipitation (sheet name: "ExPrcp") is shown in Figure 5-1. To calculate excess precipitation, click the <Excess Prcp> button after checking all required parameters (see Figure 5-1). Two methods for computing infiltration loss, the Φ-Index method and Horton's equation, were programmed and can be selected as shown in Figure 5-2. More detailed descriptions about how to compute depression storage and infiltration are summarized in the following sections.

	A	B	C	D	E	F
1	**Initial depression**			**Input data time step**		
2	Area	(in)		del-T	60	(min)
3	Perv.	0.1				
4	Imperv.	0.02		**Phi-Index method**		
5				Phi	0.1	(in/hr)
6	**Evaporation rate**					
7	Month	(in/d)		**Horton's equation**		
8	Jan	0.015		f_max	3	(in/hr)
9	Feb	0.018		f_min	0.1	(in/hr)
10	Mar	0.025		k	0.00115	(/sec)
11	Apr	0.039		R	0.2	
12	May	0.065				
13	Jun	0.099				
14	Jul	0.127		**Excess Prcp**		
15	Aug	0.124				
16	Sep	0.063				
17	Oct	0.043		Prcp_T	0	(in)
18	Nov	0.03		Rff_imp	0	(in)
19	Dec	0.015		Rff_prv	0	(in)

Figure 5-1 Main worksheet for computing excess precipitation.

Initial Depression Storage

Depression storage (or initial abstraction) must be filled prior to the occurrence of runoff caused by interception, surface wetting and ponding, and instant evaporation (Huber and Dickinson 1988). Some of the precipitation will be stored in depression storage during wet periods, and then the stored water will be depleted by evaporation during subsequent dry periods. In the developed SS model, it is necessary to input two values of initial depression storage for pervious and impervious areas (see Figure 5-1). For computing replenishment of depression storage by evaporation, local monthly evaporation rates are required (see Figure 5-1). Inputted monthly rates are automatically assigned for each month within the computing procedure. Available depression storage for each time step is calculated continuously as a function of precipitation and evaporation. If depression storage is available, it will be subtracted from the actual precipitation depth prior to considering infiltration loss.

Infiltration

Φ-Index infiltration method

The Φ-Index method computes infiltration loss as a constant for any precipitation period. Thus, the single value of Φ will be enough for computing excess precipitation as follows:

$$ExPrcp = ActPrcp - DS - (\Delta t \times \Phi) \qquad \textbf{Equation 5-1}$$

where $ExPrcp$ = excess precipitation; $ActPrcp$ = actual precipitation; DS = available depression storage; Δt = time step of precipitation data; and Φ = constant infiltration rate.

After clicking the <Phi-Index method> button (see Figure 5-2), the required parameters can be confirmed, including the input data time step, depression storage for pervious and impervious areas, and the value of Φ as shown in Figure 5-3. If all parameters are correct and the <OK> button is clicked, continuous excess precipitation for pervious and impervious areas will be computed automatically. An example of computed excess precipitation is shown in Table 5-1.
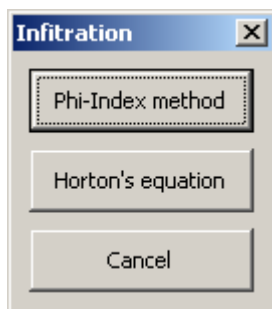
Figure 5-2 Selection of infiltration methods.
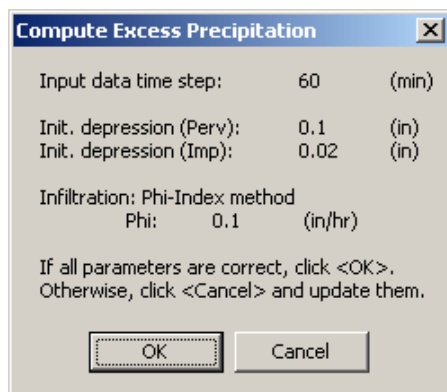
Figure 5-3 Confirmation of input parameters for Φ-Index method.

5-35

Table 5-1 Example of computing excess precipitation using the Φ-Index method.

Start	Precip.	ExP_imprev.	ExP_perv.
2000/06/23 11:15	0.1	0.08	0
2000/06/23 14:15	0.2	0.189	0.164
2000/06/23 14:30	1.5	1.5	1.475
2000/06/23 14:45	1.1	1.1	1.075
2000/06/23 15:00	1.6	1.6	1.575
2000/06/23 15:15	0.4	0.4	0.375

Horton's infiltration equation

Using Horton's equation, the infiltration rate for each time step varies between maximum (or initial) and minimum (or ultimate) rates as a function of time (Horton 1933). The following equation represents the Horton's infiltration equation and the required parameters depend on soil properties.

$$f_p = f_c + (f_o - f_c) e^{-kt}$$

Equation 5-2

where f_p = infiltration rate in time t [L/T]; f_c = minimum (or ultimate) infiltration rate [L/T]; f_o = maximum (or initial) infiltration rate [L/T]; t = time from beginning of consecutive infiltration [T]; and k = decay coefficient [/T].

Infiltration capacity will be recovered during subsequent dry periods. For continuous computing, it can be calculated according to the following equations, which are rearranged based on the SWMM manual (Huber and Dickinson 1988).

$$t_w = t_i - \frac{1}{k_d} \ln \frac{(f_o - f_c)}{(f_o - f)}$$

Equation 5-3

$$f_d = f_o - (f_o - f_c) e^{-k_d (t - t_w)}$$

Equation 5-4

$$t_{new} = t + \frac{1}{k} \ln \frac{(f_d - f_c)}{(f_o - f_c)}$$

Equation 5-5

$$k_d = R k$$

Equation 5-6

where t_w = hypothetical projected time at which infiltration rate is f_c on the recovery curve [T]; f = infiltration rate at the beginning of dry period [L/T]; t_i = time at the beginning of dry period [T]; k_d = decay coefficient for the recovery curve [/T]; f_d = recovered infiltration rate after dry period [L/T]; t = time at the beginning of the following wet period [T]; t_{new} = hypothetical projected infiltration start time at which infiltration rate is f_o for the new consecutive infiltration period [T]; and R = constant rate for recovery.

Finally, excess precipitation depth is computed continuously using Equation 5-7 for each time step.

$$ExPrcp = ActPrcp - DS - (\Delta t \times (f_{p1} + f_{p2})/2)$$

Equation 5-7

where *ExPrcp* = excess precipitation; *ActPrcp* = actual precipitation; *DS* = available depression storage; Δt = time step of precipitation data; f_{p1} = infiltration rate at the start of current time step; and f_{p2} = infiltration rate at the end of current time step.

After clicking the <Horton's equation> button (see Figure 5-2), the required parameters can be confirmed, including input data time step, depression storage for pervious and impervious area, maximum and minimum infiltration rate,

decay coefficient, and recovery rate constant as shown in Figure 5-4. If all of the parameters are correct and the <OK> button is clicked, continuous excess precipitation for pervious and impervious area will be computed automatically. An example of computed excess precipitation is shown in Table 5-2.

When the computing procedure is finished, the total precipitation depth with runoff depths from both pervious and impervious area are shown below the <Excess Prcp> button in the same worksheet (see Figure 5-1).

Table 5-2 Example of computing excess precipitation using Horton's equation.

Start	Precip.	ExP_imp.	ExP_perv.
2000/06/23 11:15	0.1	0.08	0
2000/06/23 14:15	0.2	0.189	0
2000/06/23 14:30	1.5	1.5	1.322
2000/06/23 14:45	1.1	1.1	1.02
2000/06/23 15:00	1.6	1.6	1.556
2000/06/23 15:15	0.4	0.4	0.368
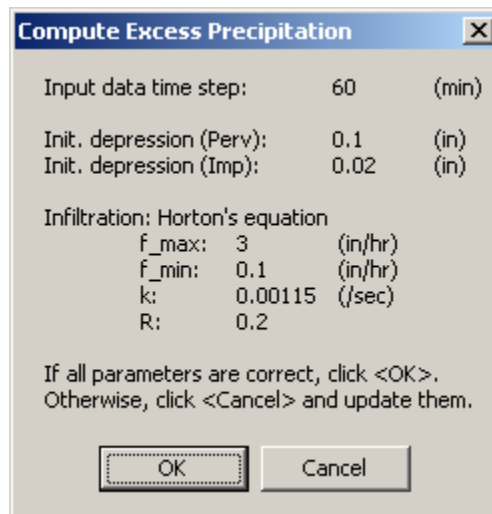
Figure 5-4 Confirmation of input parameters for Horton's equation.

The classical Horton method, which is used in this SS model, can be inaccurate, especially when light precipitation falls at the beginning of a rainfall event. The model predicts an artificially greater drop in infiltration rate that consequently results in a higher excess precipitation than it should be. Because of this reason, the integrated Horton method is used for modeling cumulative infiltration effect but an iterative solution is required. The Newton-Raphson iteration is used in SWMM for performing the integrated Horton method. An iterative approach can be modeled in spreadsheets using VBA (Visual Basic Application); however, it requires a longer computing time and may not be well suited for doing other modeling features. Hence, the iteration method is not incorporated into this SS model.

Site-specific Infiltration Data
Accurate information regarding infiltration rates is critical for estimating the performance of WWCs that rely on infiltration such as many of the LID options. Infiltration rates for soils ranging from sands to clays vary over several orders of magnitude from more than 10 inches per hour for sandy soils to essentially zero infiltration for tight clay soils. In urban systems, infiltration rates change during development due to:

1. Stripping the original topsoil and vegetation during land clearing.
2. Compaction due to heavy construction.
3. Importing soils for the construction such as raising the elevation for the slab.
4. Landscaping changes.
5. Adding irrigation systems that reduce infiltration rates because the soil moisture is increased.

Ferguson (1994) presents a detailed description of urban stormwater infiltration processes including alternative methods for measuring infiltration rates. Given local infiltration data, it is simple to estimate the parameters of the infiltration equation using standard regression analysis techniques or the Excel Solver.

Summary and Conclusions
This chapter presented continuous modeling procedures of depression storage and infiltration. Two infiltration simulation methods are included in the SS model: fixed infiltration rate and time-varied Horton's infiltration. If site-specific infiltration data are available, the modeling parameters can be calibrated using spreadsheets. This depression storage and infiltration model can be used in any stormwater simulation-optimization procedures.

Chapter 6 Flow Routing in WWC Optimization

Introduction
In traditional evaluation of WWCs, runoff routing was done to estimate the input to the WWC. Contemporary WWC evaluation methods employ the principle that control can occur anywhere in the urban system. Thus, runoff routing is not a passive aspect and may be controlled by expending resources to modify runoff coefficients, Manning's n, and other parameters associated with urban runoff calculations. This chapter describes runoff routing techniques that are incorporated into spreadsheets and can link with optimization procedures.

Rational Method for Catchment Runoff Routing
The Rational Method is the dominant approach used in the design of urban storm drainage system for small catchments. The recommended upper limits of drainage area sizes for using the Rational Method are listed in Table 6-1. The upper range of catchment sizes is from 100 to 240 acres with times of concentration of 20 minutes or less.

Table 6-1 Recommended upper limit on the size of the catchment for using the Rational Method.

Source	Upper limit (ac)	Time of concentration (min)
ASCE (1982)	200	20
Debo and Reese (1995)	100	
Denver UDFCD (2001)	160	
Singh (1992)	100-240	
Wanielista et al. (1997)		20

In applying the Rational Method ($Q = CiA$), rainfall intensity, i, is determined from the established IDF relationship. If the runoff concentration time (duration) is increased, the applied rainfall intensity in the Rational Method is decreased. The site-specific regional IDF analysis, which was described in Chapter 4 and Figure 4-5, can be used in this application. More detailed explanation on rainfall intensity and runoff concentration is presented in the following sections.

Runoff concentration time of a catchment
Runoff concentration time is an important property of small urban catchments as illustrated in Figure 6-1 (Ponce 1989). Runoff concentration means that the outflow from the catchment will increase until the entire area is contributing runoff.

If average rainfall intensity is used, then the runoff response can fall into three categories:

1. Concentrated ($t_r = t_c$). The duration of the excess precipitation (t_r) equals the time of concentration (t_c). This is the worst case condition as far as maximizing the peak flow at this outlet.
2. Superconcentrated ($t_r > t_c$). The excess rainfall lasts longer than the time of concentration. In this case, a lower average intensity could be used in the Rational Method leading to a lower peak flow estimate.
3. Subconcentrated ($t_r < t_c$). In this case the duration of the excess rainfall is less than the time of concentration. Thus, a lower maximum discharge results. This case is more typical of larger catchments.
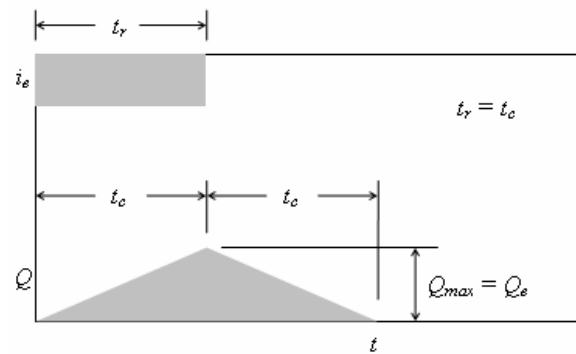
(a) Concentrated
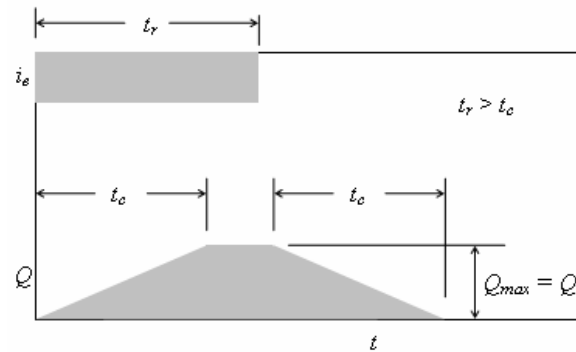
(b) Superconcentrated
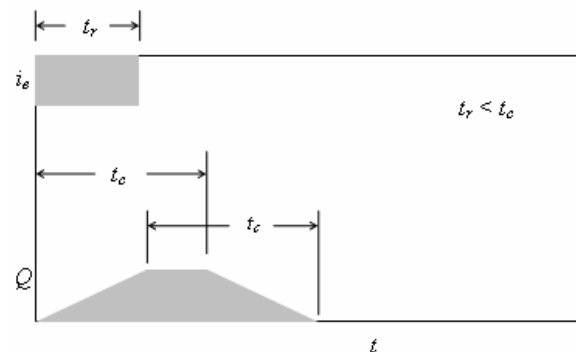
(c) Subconcentrated

Figure 6-1 Three cases: Uniform rain equal to, greater than, or less than the time of concentration (Ponce 1989).

Time of concentration and rainfall intensity

Calculating time of concentration

Numerous formulas exist for calculating the time of concentration in urban areas (ASCE 1996). Three formulas that are more suited to urban areas are described next (ASCE 1996).

A. The Izzard (1946) formula is:

$$t_c = \frac{41.025(0.0007i + c)L^{0.33}}{S^{0.333}i^{0.667}}$$

Equation 6-1

where t_c = time of concentration (min); i = average rainfall intensity (in./hr); c = retardance coefficient; L = length of flow path (ft); and S = slope of flow path (ft/ft).

Izzard's equation is based on laboratory experiments by the Bureau of Public Roads for overland flow on roadways and turf surfaces. An iterative solution is required since rainfall intensity, i, depends on the time of concentration, t_c. Values of the retardance coefficients range from 0.007 for very smooth pavement, 0.012 for concrete pavement, to 0.06 for dense turf.

B. The Federal Aviation Agency (FAA) (1970) formula is:

$$t_c = \frac{1.8(1.1 - C)L^{0.5}}{S^{0.333}}$$

Equation 6-2

where t_c = time of concentration (min); C = Rational Method runoff coefficient; L = length of overland flow (ft); and S = average overland slope (%).

The FAA equation was developed from air field drainage data and would represent runoff from these types of areas. It has an explicit solution since the time of concentration is not expressed as a function of the rainfall intensity.

C. The kinematic wave formula (Morgali and Linsley 1965) is:

$$t_c = \frac{0.94(Ln)^{0.6}}{S^{0.3}i^{0.4}}$$

Equation 6-3

where t_c = time of concentration (min); i = average rainfall intensity (in./hr); n = Manning roughness coefficient; L = length of flow path (ft); and S = slope of flow path (ft/ft).

Equation 6-3 was developed from the kinematic wave analysis of surface runoff from developed surfaces. Like Izzard's equation, iteration is required since rainfall intensity depends on the time of concentration.

Interactive relationship between rainfall intensity and time of concentration

For the iterative solution required for Izzard's and the kinematic wave equations, the other needed equation is that approximates the IDF relationship. A popular form to use is:

$$i = \frac{a}{(t^b + c)}$$

Equation 6-4

where i = average intensity [L/T]; t = time of concentration or duration [T]; and a, b, c = parameters.

Equation 6-4 can be solved for t yielding:

$$t_{idf} = (\frac{a}{i} - c)^{1/b}$$ **Equation 6-5**

where t_{idf} = duration from the IDF curve [T].

Goal Seek under Tools in Excel can be used to find the value of i such that

$$i_e = i_{idf}$$ **Equation 6-6**

where i_e = excess rainfall intensity [L/T] and i_{idf} = intensity from the IDF curve [L/T].

Velocity method for estimating travel times
McCuen (1998) describes how the velocity of travel, V, can be estimated using Manning's equation, i.e.,

$$V = \frac{1.486}{n} R^{2/3} S^{1/2}$$ **Equation 6-7**

where V = velocity (ft/sec); n = Manning's resistance coefficient; R = hydraulic radius; and S = slope (dimensionless).

For a given land use and flow condition, n and R are constant. Thus,

$$V = kS^{1/2}$$ **Equation 6-8**

where $k = (1.486/n)R^{2/3}$ is the coefficient of velocity. Values for n, R, and k for a variety of land uses are shown in Table 6-2. Table 6-3 shows the velocities and travel times for flow over grass, sheet flow on pavements, and gutter flows for a variety of slopes. The differences in velocities and travel times between the pervious areas and DCIAs are dramatic. For a slope of 1%, the velocity for grass flow is 0.15 ft/sec, whereas paved area flow is 2.0 ft/sec and paved gutter flow 4.6 ft/sec. Both paved area and gutter flows are more than an order of magnitude greater. Correspondingly, the time to travel 200 feet across these same surfaces is only a minute or two for the impervious surfaces but is over 20 minutes for the pervious surfaces. These response times for small urban catchments are very important since the average intensities on the IDF curves change significantly for small concentration times, particularly in the range from 5 to 20 minutes. For example, the IDF curves for Boulder Colorado, generated on the basis of 15-minute data, are shown in Figure 4-5. Unlike the typical IDF curves, which only include recurrence intervals of two years or longer, these curves include recurrence intervals of 1, 3, 6, and 12 months. Figure 4-5 shows that the average rainfall intensity increases significantly when the duration is less than 30 minutes. This implies that higher design rainfall intensities must be used for small DCIA catchments that have 15 minutes or less time of concentration.

Table 6-2 Values of N, R, and k for various land uses (McCuen 1998).

Land use/flow regime	n	R (ft)	k
Forest			
Dense underbrush	0.8	0.25	0.7
Light underbrush	0.4	0.22	1.4
Heavy ground litter	0.2	0.2	2.5
Grass			
Bermuda grass	0.41	0.15	1
Dense	0.24	0.12	1.5
Short	0.15	0.1	2.1
Short grass pasture	0.025	0.04	7
Conventional tillage			
With residue	0.19	0.06	1.2
No residue	0.09	0.05	2.2
Agricultural			
Cultivated straight row	0.04	0.12	9.1
Contour or strip cropped	0.05	0.06	4.6
Trash fallow	0.045	0.05	4.5
Rangeland	0.13	0.04	1.3
Alluvial fans	0.017	0.04	10.3
Grassed waterway	0.095	1	15.7
Small upland gullies	0.04	0.5	23.5
Paved area (shallower sheet flow)	0.011	0.06	20.8
Paved area (deeper sheet flow)	0.025	0.2	20.4
Paved gutter	0.011	0.2	46.3

Table 6-3 Velocities and travel times for grass, paved area, and paved gutters based on Equation 6-8.

Land use	k
Grass	1.5
Paved area, sheet flow	20.6
Paved gutter	46.3

	Velocity (ft/sec)			Time to travel 200 ft. (min)		
Slope	Grass	Paved area	Paved gutter	Grass	Paved area	Paved gutter
0.001	0.05	0.65	1.46	70.27	5.12	2.28
0.002	0.07	0.92	2.07	49.69	3.62	1.61
0.003	0.08	1.13	2.54	40.57	2.95	1.31
0.004	0.09	1.3	2.93	35.14	2.56	1.14
0.005	0.11	1.46	3.27	31.43	2.29	1.02
0.006	0.12	1.6	3.59	28.69	2.09	0.93
0.007	0.13	1.72	3.87	26.56	1.93	0.86
0.008	0.13	1.84	4.14	24.85	1.81	0.8
0.009	0.14	1.95	4.39	23.42	1.71	0.76
0.01	0.15	2.06	4.63	22.22	1.62	0.72
0.015	0.18	2.52	5.67	18.14	1.32	0.59
0.02	0.21	2.91	6.55	15.71	1.14	0.51
0.025	0.24	3.26	7.32	14.05	1.02	0.46
0.03	0.26	3.57	8.02	12.83	0.93	0.42
0.035	0.28	3.85	8.66	11.88	0.86	0.38
0.04	0.3	4.12	9.26	11.11	0.81	0.36

Channel and Reservoir Flow Routing

The two main changes associated with channel and reservoir flow routing are:

1. Pure translation of the parcel of water.
2. Attenuation of the peak flow due to the storage effect associated with channel routing.

For channels, the translation effect dominates whereas for reservoirs, the storage effect dominates. Thus, one can speak in general of the integrated effect of storage and flow. From a control point of view, it is possible to achieve preferred outcomes by manipulating the parameters. Simpler methods for flow routing are described below.

Indirect using Rational Method
The Rational Method does implicit flow routing by varying the peak intensity as the time of concentration changes. The IDF curves in Figure 4-5 imply that an effective way to reduce the peak flow is to increase the time of concentration. The time of concentration can be calculated using the kinematic wave equation (Morgali and Linsley 1965) as shown in Equation 6-3. This equation indicates that the time of concentration can be increased by increasing the flow length, L and/or Manning's roughness coefficient, n. It can also be increased by decreasing the slope, S.

Summary and Conclusions
This chapter described how to apply the Rational Method for analyzing stormwater flow. Time of concentration, t_c, is a function of rainfall intensity and catchment's spatial characteristics. Rainfall intensity, i, in the Rational Method should be considered with an appropriate site-specific time of concentration. A spreadsheet-based method for finding reasonable matches of t_c and i was presented. The Rational Method can also be used indirectly in many WWC optimization procedures. Real applications will be presented later in this report.

Chapter 7 Pollutant Characterization

Introduction

Pitt (2000) described the sources and characteristics of urban runoff pollutants, such as chemicals, dust and dirt, particulates, atmospheric sources, and toxicants. He summarized available data on pollutant loadings for various land uses. EMC data are also available from other sources. The ASCE/EPA BMP database has extensive EMC data available for specific study areas (http://www.bmpdatabase.org/docs.html). This database can be queried by BMP type, water quality parameter, location and size of watershed, and storm volume. However, the data are for storm events only, which makes it impossible to do intra-storm process analysis. The interested reader is referred to these sources for additional information.

Pollutant Characterization

Treatability studies

The treatability of wet-weather flows, like all other waste sources, depends on its site-specific characteristics. Some wastewaters, like domestic wastewater, have similar characteristics from area to area. However, similar attempts to depict stormwater characteristics as a function of land use, have been less successful due to its highly variable nature, both in space and time. As with infiltration estimates, site-specific information is essential. Rapp et al. (2004) presents some sample wet-weather treatability data and a simple use of optimization methods to find the parameter estimates as part of evaluating detention systems. A summary of this effort is presented below.

The most important, albeit most difficult, information for evaluating the water quality performance of detention basins is the rate of removal of pollutants. Many detention basin models rely on first order reaction kinetics to determine the removal of pollutants. Therefore, these models are heavily dependent on the reaction rate constant, k. The value of k is often estimated based on previous studies, but caution should be exercised because the value can vary significantly from site to site. When considering suspended solids as a measure of pollutant concentration, the range of concentrations can vary significantly for different storms as well as different regions. The removal rate of suspended solids is dependent on many factors including initial concentration, particle size distribution, and settling velocity. Site specific characterization data including particle size distribution and settling velocities are very important in determining an accurate removal rate for a site. However, very few studies have collected this high quality characterization data.

The U.S. Geological Survey has conducted a major study for the Federal Highway Administration (FHWA) on stormwater runoff quality and its impacts (Bent et al. 2001). A key technical finding of this paper is that suspended solids are being measured incorrectly in many of the highway runoff studies. The USGS recommends measuring suspended sediment concentration (SSC), and not total suspended solids (TSS). The popular TSS method excludes many of the coarser sand size materials that may absorb significant amounts of pollutants. The suspended solids sampling method can have a major impact on the reported removal efficiencies of WWCs.

Very few studies have measured particle size distribution and sedimentation rates in stormwater runoff. A study by

Randall et al. (1982) provides data from seven urban runoff sedimentation tests that are useful to develop numerous settling rate curves. An analysis of this urban runoff sedimentation data to develop first order reaction rate constants using linear regression techniques is presented in this section. The reaction rate constants and associated initial concentrations are then used to evaluate the dependency of detention basin designs on these assumed values.

Determination of reaction rate constants
Pollutant removal in a detention basin is often modeled using plug flow reaction kinetics. A first order plug flow model is typically used because the solution for concentration as a function of time can be easily represented as:

$$c = c_o e^{-kt}$$ **Equation 7-1**

where c is the final concentration (mg/L), c_o is the initial concentration (mg/L), k is the reaction rate constant (hr^{-1}), and t is the detention time (hr) of each pulse.

Randall et al. (1982) present one of the few studies done regarding pollutant removal by sedimentation in urban runoff. Another study on sedimentation was conducted by Whipple and Hunter (1981). The results from Randall et al. (1982) are used in this section to develop values of k for use in a simulation-optimization model. The settleability data from Randall et al. (1982) are shown in Table 7-1.

Table 7-1 Settleability of suspended solids in urban runoff (Randall et al. 1982).

Test No.	Total Suspended Solids Concentration, mg/L						Maximum % Removal
		Sedimentation Time, hours					
	Initial	2	6	12	24	48	
1	15	14	14	13	11	2	87
2	35	20	18.5	18	14.5	7	80
3	38	24	16		6		84
4	100	45	34	30	19	7	93
5	155	21	17	12	9	7	95
6	215	67	40	26	17	9	96
7	721	103	34	30	18	18	98

One method to determine k from the settleability data is using nonlinear least squares regression. First, each of the seven settleability tests is considered individually. The measured concentration data are compared with the calculated concentration data using Excel Solver to minimize the sum of squared errors.

$$SSE = \sum (c_{meas} - c_{calc})^2$$ **Equation 7-2**

where SSE = sum of squared errors; c_{meas} = measured concentration; and c_{calc} = calculated concentration by Equation 7-1.

The SSE is minimized by changing the decision variable k until an optimal solution is achieved, i.e., the minimum value of SSE is found. In addition to considering each settleability test separately, average values for c_o and k are determined by minimizing the SSE for all the settleability data simultaneously. The values of c_o and k from this analysis are found in Table 7-2.

Table 7-2 Optimal fitted values for c_o and k using the Excel Solver.

Test Number	1	2	3	4	5	6	7	Grouped
c_o (mg/L)	15	35	38	100	155	215	721	181.97
k (/min)	0.0226	0.0555	0.1851	0.1881	0.9761	0.4935	0.9632	0.6837

Summary and Conclusions

Pollutant treatability varies site by site. This chapter described how to develop site-specific pollutant characterization data, e.g., initial concentration (c_o) and removal reaction rate (k), using measured stormwater quality data. Inflow concentration and reaction rate is very important in analyzing the performance of WWCs. The results presented in this chapter are applied to a continuous simulation-optimization model, which will be presented in Chapter 11.

Chapter 8 Development of SS STORM for WWC Simulation/Optimization

Introduction

All wet-weather controls (WWCs) are combinations of storage (S) and treatment (T). The amount of treatment depends on the detention time within the control, t. The nominal detention time, t_{nom}, is:

$$t_{nom} = S/T \qquad \qquad \textbf{Equation 8-1}$$

A combination of storage and treatment occurs in every element of a wet-weather system, not just the units that have been designated as "control" units. The dilemma, from a performance point of view, is that pollutant removal increases as residence time increases. However, if water is left in the WWC too long then it won't be emptied before the next storm event arrives. In order to find the best mix of storage and release over an extended period of time, it is necessary to use continuous simulation. A model called STORM (Storage, Treatment, Overflow, Runoff Model) (Hydrologic Engineering Center 1977) was developed for this purpose. It has been widely used for doing these analyses. A spreadsheet version of STORM (SS STORM) has been developed as part of this project.

Continuous Rainfall-Runoff and Storage-Treatment Simulation Model

Urban stormwater quality control systems can be evaluated using a conceptual storage-release system shown in Figure 8-1. Stormwater flow may be captured and treated by a WWC or bypassed after its storage capacity is reached. Nix and Heaney (1988) presented a graphical two-variable production function approach using storage volume and release rate to optimize WWC strategies. They performed a number of continuous simulations to develop the two-variable production function, and then this production function was applied for optimization. WWC performance was evaluated based on captured runoff volume and long-term percent pollutant removal. However, this approach was tedious owing to the requirement of many continuous simulations and the difficulties in developing a graphical production function and an analytical representation of it.

Developing a physically sound process simulator is critically important for analyzing WWC alternatives. Based on the concept in the STORM model, a continuous rainfall-runoff and storage-treatment simulation model was developed using spreadsheets (Lee 2003). This SS STORM can be easily integrated with optimization tools in spreadsheets for finding the best WWC option.
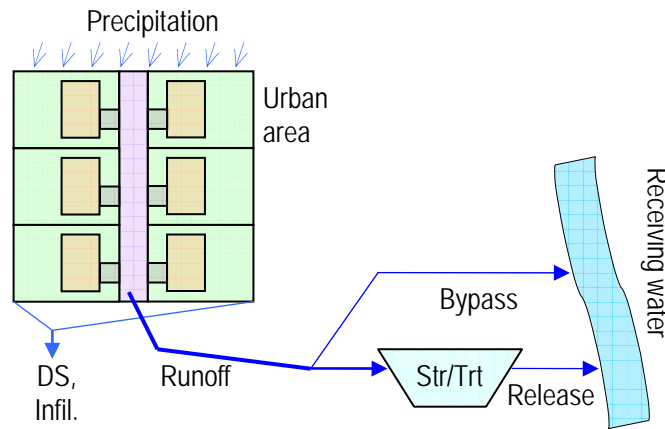
Figure 8-1 Stormwater rainfall-runoff and storage-release system.

Generic flow routing model

A mass-balance based simple area-normalized flow routing model can be applied. Some of the precipitation will fill depression storage or infiltrate. Excess rainfall beyond depression storage and infiltration capacity will be released from the catchment. Released surface runoff may flow to the WWC facility before reaching its maximum capacity, and then is bypassed to the receiving water. The following equations show how to model rainfall-runoff and storage-treatment:

$$Rff = Prcp - DS - Inf \qquad \text{\textbf{Equation 8-2}}$$

$$Trt = Rff - Bpsd \qquad \text{\textbf{Equation 8-3}}$$

where Rff = catchment runoff; $Prcp$ = precipitation depth; DS = depression storage; Inf = infiltration loss; Trt = treated stormwater; and $Bpsd$ = bypassed stormwater.

Development of SS STORM

An example of the developed SS STORM is shown in Figure 8-2. The model consists of three main parts: input data, continuous water quantity and quality simulators, and overall modeling results. A detailed description of the model is presented next.

Model input data

A number of data are required for performing a continuous rainfall-runoff and storage-treatment simulation. There are basically two different situations: a dry period and a wet period. Rainfall-runoff occurs during a wet period. Available depression storage (DS) will decrease during a wet period, but it will recover during a dry period by evapotranspiration (ET). Stormwater pollutants can be removed as a function of storage volume and release rate. Required input data for a continuous simulation are summarized below:

- DS: average depression storage (or initial abstraction) throughout the catchment, [mm]
- f: average infiltration rate throughout the catchment, [mm/hr]
- del-T: applied precipitation data time step (Δt), [hr]
- C_{in}: stormwater runoff pollutant concentration, [mg/L]
- k: first-order reaction constant, [/hr]
- S: maximum storage capacity of the storage/treatment system, [mm]
- Q_{rls}: fixed release rate of the storage/treatment system, [mm/hr]
- Empty: calculated release time from full to empty for the storage/treatment system (Empty = S / Q_{rls}), [hr]
- ET: regional monthly average ET rate, [mm/d]
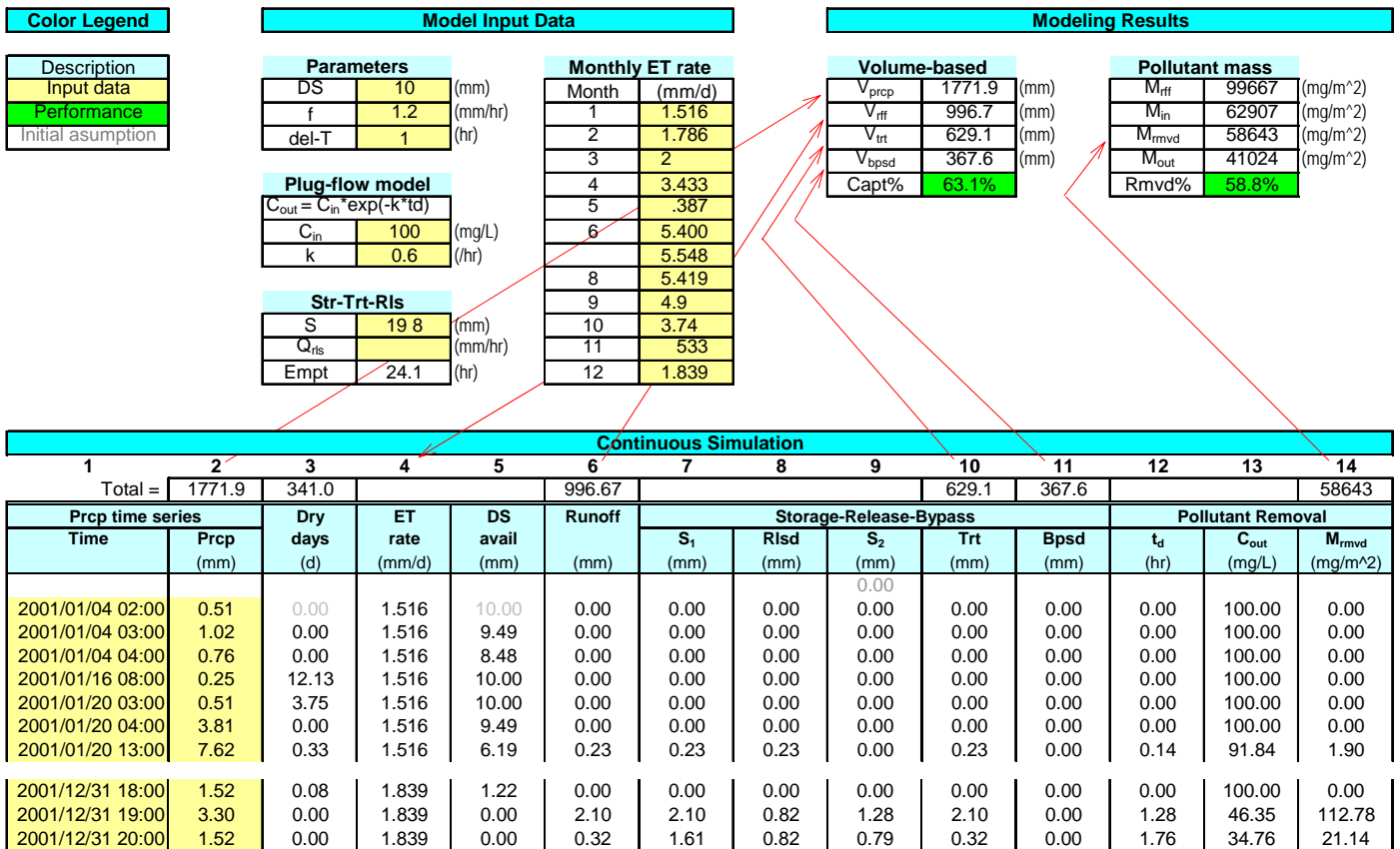
Figure 8-2 Developed SS STORM.

Model Input Data / Modeling Results panel:

Color Legend

Description
Input data
Performance
Initial asumption

Parameters

DS	10	(mm)
f	1.2	(mm/hr)
del-T	1	(hr)

Plug-flow model

$C_{out} = C_{in} \cdot \exp(-k \cdot t_d)$		
C_{in}	100	(mg/L)
k	0.6	(/hr)

Str-Trt-Rls

S	19 8	(mm)
Q_{rls}	533	(mm/hr)
Empt	24.1	(hr)

Monthly ET rate

Month	(mm/d)
1	1.516
2	1.786
3	2
4	3.433
5	.387
6	5.400
7	5.548
8	5.419
9	4.9
10	3.74
11	533
12	1.839

Volume-based

V_{prcp}	1771.9	(mm)
V_{rff}	996.7	(mm)
V_{trt}	629.1	(mm)
V_{bpsd}	367.6	(mm)
Capt%	63.1%	

Pollutant mass

M_{rff}	99667	(mg/m^2)
M_{in}	62907	(mg/m^2)
M_{rmvd}	58643	(mg/m^2)
M_{out}	41024	(mg/m^2)
Rmvd%	58.8%	

Continuous Simulation

1	2	3	4	5	6	7	8	9	10	11	12	13	14
Total =	1771.9	341.0			996.67				629.1	367.6			58643
Prcp time series		Dry	ET	DS	Runoff	Storage-Release-Bypass					Pollutant Removal		
Time	Prcp	days	rate	avail		S_1	Rlsd	S_2	Trt	Bpsd	t_d	C_{out}	M_{rmvd}
	(mm)	(d)	(mm/d)	(mm)	(mm)	(mm)	(mm)	(mm)	(mm)	(mm)	(hr)	(mg/L)	(mg/m^2)
								0.00					
2001/01/04 02:00	0.51	0.00	1.516	10.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	100.00	0.00
2001/01/04 03:00	1.02	0.00	1.516	9.49	0.00	0.00	0.00	0.00	0.00	0.00	0.00	100.00	0.00
2001/01/04 04:00	0.76	0.00	1.516	8.48	0.00	0.00	0.00	0.00	0.00	0.00	0.00	100.00	0.00
2001/01/16 08:00	0.25	12.13	1.516	10.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	100.00	0.00
2001/01/20 03:00	0.51	3.75	1.516	10.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	100.00	0.00
2001/01/20 04:00	3.81	0.00	1.516	9.49	0.00	0.00	0.00	0.00	0.00	0.00	0.00	100.00	0.00
2001/01/20 13:00	7.62	0.33	1.516	6.19	0.23	0.23	0.23	0.00	0.23	0.00	0.14	91.84	1.90
2001/12/31 18:00	1.52	0.08	1.839	1.22	0.00	0.00	0.00	0.00	0.00	0.00	0.00	100.00	0.00
2001/12/31 19:00	3.30	0.00	1.839	0.00	2.10	2.10	0.82	1.28	2.10	0.00	1.28	46.35	112.78
2001/12/31 20:00	1.52	0.00	1.839	0.00	0.32	1.61	0.82	0.79	0.32	0.00	1.76	34.76	21.14

Continuous simulation

Rainfall-runoff and storage-release routing

Runoff occurs after satisfying available land surface depression storage and deducting infiltration loss. Released surface runoff may flow to the control facility before reaching its maximum capacity, and then is bypassed to the receiving water. Water in the control facility will be released at a certain rate. The following column by column description shows how to simulate continuous rainfall-runoff and storage-treatment on a volumetric basis:

1. Time: beginning of the time for each precipitation pulse, [yyyy/mm/dd hh:mm]

2. Prcp: precipitation depth, [mm]

3. Dry days: number of dry days from the end of the previous precipitation pulse (t_{dry}), [day]

$$t_{dry}^{j} = \text{ROUND}[Time^{j} - \{Time^{j-1} + (\Delta t / 24)\}, 5]$$ **Equation 8-4**

4. ET rate: monthly average ET rate for recovering DS during dry period, [mm/hr]

$$ET^{j} = \text{LOOKUP}\{(\text{MONTH}(Time^{j}), Cell\_range\_months, Cell\_range\_ETs\}$$ **Equation 8-5**

5. DS avail: available depression storage at the beginning of the time step, [mm]

$$DS_{avail}^{j} = \text{MIN}(DS_{max}, \text{MAX}(0, DS_{avail}^{j-1} - Prcp^{j-1}) + ET^{j} \times t_{dry}^{j})$$ **Equation 8-6**

6. Runoff: area normalized runoff volume, [mm]

$$Rff^{j} = \text{MAX}(0, Prcp^{j} - DS_{avail}^{j} - f \times \Delta t)$$ **Equation 8-7**

7. S_1: upper bound of storage volume during the time step, before releasing, [mm]

$$S_1^j = \text{MIN}(S_{max}, \text{MAX}(0, S_2^{j-1} - Q_{rls} \times t_{dry}^j \times 24) + Rff^j)$$ **Equation 8-8**

8   Rlsd: released volume from the storage/treatment system during the time step, [mm]

$$Rlsd^j = \text{MIN}(S_1^j, Q_{rls} \times \Delta t)$$ **Equation 8-9**

9   S_2: lower bound of storage volume during the time step, after releasing, [mm]

$$S_2^j = S_1^j - Rlsd^j$$ **Equation 8-10**

10 Trt: inflow volume to the storage/treatment system for treatment during the time step, [mm]

$$Trt^j = \text{MIN}\{Rff^j, S_{max} - \text{MAX}(0, S_2^{j-1} - Q_{rls} \times t_{dry}^j \times 24)\}$$ **Equation 8-11**

11 Bpsd: bypassed stormwater volume during the time step, [mm]

$$Bpsd^j = Rff^j - Trt^j$$ **Equation 8-12**

where $Prcp$ = precipitation pulse [mm]; DS_{max} = maximum depression storage [mm]; DS_{avail} = available depression storage at the beginning of each time step [mm]; ET = monthly average ET rate [mm/hr]; t_{dry} = number of days after the previous precipitation pulse [day]; Rff = runoff depth during the time step [mm]; f = infiltration rate [mm/hr]; Δt = precipitation data time step size [hr]; S_{max} = maximum storage [mm]; S_1 = storage volume at the beginning of the time step [mm]; S_2 = storage volume at the end of the time step [mm]; Q_{rls} = release rate [mm/hr]; $Rlsd$ = release volume during the time step [mm]; Trt = treated stormwater during the time step [mm]; $Bpsd$ = bypassed stormwater during the time step [mm]; and j = temporal steps in simulation.

Long-term precipitation time-series data can be obtained from the National Climatic Data Center (NCDC) as described in Chapter 4. Time in column 1 represents the beginning time of each precipitation pulse. Number of dry days in column 3 is the time difference from the end of the previous precipitation pulse, i.e., $Time^{j-1} + (\Delta t / 24)$, to the beginning of the current precipitation pulse, i.e., $Time^j$. The $ROUND$ function is applied for calculating the number of dry days to avoid possible numerical errors, which might be introduced from using long decimal digits. The number of dry days at the first precipitation pulse is assumed as zero. The monthly ET rate in column 4 is obtained from the model input data using a spreadsheet $LOOKUP$ function.

Continuous rainfall-runoff is simulated in columns 5 and 6. Available DS in column 5 is decreased (filled) by precipitation during a wet period and increased (recovered) by ET during a dry period. However, the calculated value of available DS can be neither negative nor greater than the maximum DS. To avoid unrealistic simulations, i.e., negative value or beyond the maximum capacity, MAX and MIN functions are used in the SS STORM. Initial DS at the beginning of the first precipitation pulse is assumed as the maximum DS. Runoff volume in column 6 is calculated using precipitation depth, available DS, and infiltration loss. Runoff volume is decreased by filling available DS and deducting infiltration loss.

Continuous flow routing for a storage-based WWC system is simulated in columns 7 through 11. The developed flow routing model is based on the plug-flow (PF) model concept. Every pulse is simulated separately and no mixing is assumed to occur within or among pulses. Storage volume filled by stormwater runoff is calculated before (S_1 in column 7) and after (S_2 in column 9) releasing for every time step. Storage is increased by any new entering runoff to the system and decreased by releasing stormwater to the receiving water. Releases may happen continuously during dry and wet periods if stormwater is stored in the system. Released volume during the time step, i.e., releases during a wet period, is modeled in column 8. Released volume between two consecutive pulses, i.e., releases during a dry period ($Q_{rls} \times t_{dry}^j \times 24$), is calculated inside column 7 for modeling the upper bound of storage volume for the current time step (S_1). The upper bound of storage (S_1) for the current time step is a storage volume before releasing any stormwater during that time step. The lower bound of storage (S_2) is a volume after releasing stormwater during the

time step. Thus, S_2 is the actual storage at the end of the time step. Initial storage volume is assumed as zero at the beginning of the simulation. Stormwater runoff can enter until the maximum storage volume is reached. The entered runoff will be treated in the system and the volume of treated stormwater for each pulse is calculated in column 10. If runoff is less than the available storage at the beginning of the time step j, which is $S_{max} - \text{Max}(0, S_2^{j-1} - Q_{rls} \times t_{dry}^j \times 24)$, then the entire stormwater runoff will be treated. Otherwise, the runoff will be treated up to the available storage. Runoff beyond the available storage volume is bypassed and the bypassed volume is modeled in column 11.

Water quality simulation

Pollutants will be removed in the storage facility through various physical, chemical, and biological processes. These removal processes may occur continuously during wet and dry periods if the pulse stayed in the WWC system. In SS STORM, pollutant removal is modeled for each entering stormwater pulse using the pulse's own detention time. The continuous pollutant treatment procedure is simulated using a first order plug-flow model in columns 12 to 14 as follows:

12 t_d: average detention time for each pulse based on plug-flow model

$$t_d^j = \frac{\text{MAX}(0, S_2^{j-1} - Q_{rls} \times t_{dry}^j \times 24)}{Q_{rls}} + 0.5 \frac{Trt^j}{Q_{rls}}$$ **Equation 8-13**

13 C_{out}: outflow pollutant concentration from the storage/treatment system, [mg/L]

$$C_{out}^j = C_{in} e^{-k \times t_d^j}$$ **Equation 8-14**

14 M_{rmvd}: removed pollutant mass for each pulse, [mg/m^2]

$$M_{rmvd}^j = Trt^j (C_{in} - C_{out}^j)$$ **Equation 8-15**

where t_d = hydraulic detention time for each plug [hr]; C_{out} = released outflow concentration [mg/L]; C_{in} = stormwater inflow concentration [mg/L]; k = first-order reaction constant [/hr]; and M_{rmvd} = area normalized pollutant mass removal [mg/m^2].

Calculating a correct hydraulic detention time, i.e., residence time, is one of the most important steps in this model. Detention time is simulated for each stormwater pulse as a function of storage volume and release rate. Based on the pure plug-flow modeling concept, the initial or existing storage at the beginning of the time step, i.e., $\text{MAX}(0, S_2^{j-1} - Q_{rls} \times t_{dry}^j \times 24)$, will be released first, and then the entering stormwater pulse to the storage system, i.e., Trt^j, will be released after being treated. The total detention time of the entering pulse is calculated as the sum of the existing storage residence time and the current pulse average residence time as described in Figure 8-3.

The first part of Equation 8-13 represents the residence time for the existing storage ($t_1 - t_0$ in Figure 8-3) and the second part represents the average residence time for the entering pulse ($t_{avg} - t_1$ in Figure 8-3). The center of a time span is used to obtain an average residence time for each entering pulse. That is why 0.5 is multiplied in the second part of Equation 8-13. The residence time, i.e., detention time, is calculated in column 12, and then used in column 13 to calculate the treated outflow concentration. The outflow concentration is calculated using the plug-flow equation (Equation 8-14). The removed pollutant mass is calculated using each treated volume, i.e., volume of the entering pulse, and the difference of inflow and outflow concentrations for each pulse as shown in Equation 8-15 and column 14.
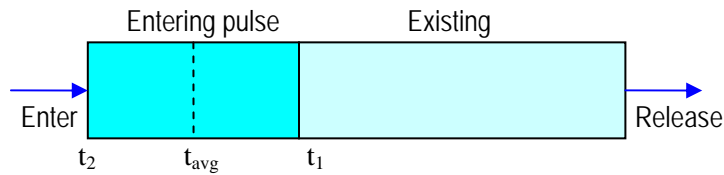
t_0 = beginning of the time step; release starting time for the existing storage

t_1 = release ending time for the existing storage; release starting time for the entering pulse

t_2 = release ending time for the entering pulse

t_{avg} = average release time for the entering pulse

$t_1 - t_0$ = release or residence time for the existing storage

$$= \frac{\mathrm{MAX}(0,\ S_2^{j-1} - Q_{rls} \times t_{dry}^{j} \times 24)}{Q_{rls}}$$

$t_2 - t_1$ = release or residence time for the entering pulse only

$$= \frac{Trt^{j}}{Q_{rls}}$$

$t_{avg} - t_0$ = total average residence time for the entering pulse

$$= t_d^{j} = \frac{\mathrm{MAX}(0,\ S_2^{j-1} - Q_{rls} \times t_{dry}^{j} \times 24)}{Q_{rls}} + 0.5\frac{Trt^{j}}{Q_{rls}}$$

Figure 8-3 Plug-flow modeling of a hydraulic detention time in SS STORM.

Modeling results

The overall modeling results are presented in the upper right corner of the model (see Figure 8-2). Items for the modeling results are described below:

- V_{prcp}:   sum of the entire precipitation volume from column 2, [mm]
- V_{rff}:   sum of the entire runoff volume from column 6, [mm]
- V_{trt}:   sum of the entire treated volume from column 10, [mm]
- V_{bpsd}:   sum of the entire bypassed volume from column11, [mm]
- Capt%: volume-based overall stormwater capture rate by the WWC (Capt% = $100 \times V_{trt} / V_{rff}$), [%]
- $t_{d\text{-avg}}$:   average detention time for all of the pulses, [hr]
- $C_{out\text{-avg}}$: average outflow concentration for all of the pulses, [mg/L]
- M_{rff}:   total pollutant mass loading by stormwater runoff ($M_{rff} = V_{rff} \times C_{in}$), [mg/m^2]
- M_{in}:   sum of the inflow pollutant mass to the WWC ($M_{in} = V_{trt} \times C_{in}$), [mg/m^2]
- M_{rmvd}: sum of the removed pollutant mass from column 14, [mg/m^2]
- M_{out}:   total pollutant mass loading to the receiving water ($M_{out} = M_{rff} - M_{rmvd}$), [mg/m^2]
- Rmvd%: overall pollutant removal rate by the WWC (Rmvd% = $100 \times M_{rmvd} / M_{rff}$), [%]

The above modeling results are mostly the sums from the continuous simulation for each item. The performance of a WWC can be evaluated based on captured volume and/or pollutant mass removal. The total pollutant mass loading to the receiving water (M_{out}) is contributed by both treated and bypassed stormwater. Pollutant concentration of the bypassed stormwater is assumed to be the same as the inflow concentration (C_{in}).

Production Function from SS STORM

SS STORM can be used to estimate performance for various combinations of storage (S) and release rate (Q_{rls}). This calculation can be expedited using the two-variable data table feature in Excel, wherein Excel calculates the performance of the system for prescribed values of S and Q_{rls}. The procedures to create a two-variable data table for

various combinations of S and Q_{rls} are summarized below:

1. In a cell on the worksheet, enter the performance estimating formula.
2. Type one list of input values in the same column, below the formula. (Note: The list in the column can represent either S or Q_{rls}.)
3. Type the second list in the same row, to the right of the formula. (Note: The list in the row can also represent either S or Q_{rls}. If the values in the column represent S, the values in the row should represent Q_{rls}, and vice versa.)
4. Select the range of cells that contains the formula and both the row and column of values.
5. On the **Data** menu in Excel, click **Table**.
6. In the **Row input cell** box, enter the reference to the input cell for the input values in the row.
7. In the **Column input cell** box, enter the reference to the input cell for the input values in the column.
8. Click **OK**.

An example of the two-variable data table using SS STORM is presented in Figure 8-4.

58.8%	3	6	9	12	15	18	21	24	27	30	33	36	39	42	45	48	51	54	57	60
0.3	17.2%	27.9%	35.3%	40.8%	45.3%	49.5%	53.1%	56.3%	58.8%	60.7%	62.3%	63.8%	65.3%	66.9%	68.4%	69.9%	71.4%	72.5%	73.4%	74.3%
0.6	16.4%	28.5%	36.9%	44.0%	50.0%	55.5%	60.1%	63.6%	66.6%	68.8%	70.5%	71.7%	72.9%	74.1%	75.3%	76.5%	77.7%	78.6%	79.5%	80.4%
0.9	14.6%	27.0%	35.9%	43.3%	49.5%	55.7%	60.6%	64.5%	68.0%	70.4%	72.1%	73.6%	75.1%	76.6%	78.1%	79.6%	80.9%	81.9%	82.8%	83.7%
1.2	13.0%	24.9%	34.4%	42.1%	48.5%	54.6%	59.6%	64.0%	67.9%	70.4%	72.2%	74.2%	76.1%	77.8%	79.3%	80.8%	82.2%	83.1%	84.0%	84.9%
1.5	11.5%	23.1%	32.6%	40.8%	47.5%	53.7%	58.5%	62.8%	66.7%	69.5%	71.7%	73.8%	75.6%	77.5%	79.4%	81.2%	82.5%	83.5%	84.3%	84.9%
1.8	10.4%	21.7%	31.1%	39.5%	46.5%	52.8%	57.5%	61.9%	65.6%	68.2%	70.5%	72.7%	74.9%	77.1%	79.1%	80.7%	82.0%	83.1%	83.7%	84.3%
2.1	9.4%	20.4%	29.7%	38.1%	45.4%	51.6%	56.4%	60.8%	64.5%	67.0%	69.1%	71.4%	73.6%	75.9%	78.1%	79.9%	81.5%	82.4%	83.0%	83.6%
2.4	8.5%	19.2%	28.5%	36.7%	44.0%	50.1%	55.0%	59.4%	63.1%	65.6%	67.7%	69.9%	72.0%	74.3%	76.4%	78.4%	79.9%	80.8%	81.4%	82.0%
2.7	7.7%	18.1%	27.3%	35.4%	42.6%	48.6%	53.6%	57.9%	61.7%	64.1%	66.3%	68.5%	70.6%	72.7%	74.7%	76.7%	78.1%	79.0%	79.6%	80.2%
3.0	7.0%	17.1%	26.2%	34.3%	41.3%	47.3%	52.3%	56.6%	60.4%	62.8%	65.0%	67.2%	69.4%	71.5%	73.4%	75.1%	76.6%	77.4%	78.0%	78.5%
3.3	6.4%	16.1%	25.1%	33.2%	40.1%	46.0%	51.0%	55.3%	59.1%	61.5%	63.8%	66.0%	68.1%	70.2%	72.1%	73.8%	75.1%	75.7%	76.3%	76.7%
3.6	5.9%	15.3%	24.1%	32.0%	38.9%	44.8%	49.7%	54.1%	57.9%	60.3%	62.5%	64.8%	66.9%	69.0%	70.8%	72.5%	73.7%	74.2%	74.5%	74.8%
3.9	5.5%	14.5%	23.1%	30.8%	37.6%	43.5%	48.5%	52.8%	56.7%	59.1%	61.3%	63.6%	65.8%	67.8%	69.6%	71.3%	72.1%	72.5%	72.8%	73.1%
4.2	5.2%	13.7%	22.2%	29.6%	36.4%	42.3%	47.2%	51.5%	55.4%	57.8%	60.1%	62.4%	64.6%	66.5%	68.4%	69.8%	70.5%	70.9%	71.2%	71.5%
4.5	4.9%	13.0%	21.2%	28.5%	35.0%	41.0%	45.8%	50.1%	54.0%	56.5%	58.8%	61.1%	63.3%	65.2%	67.0%	68.2%	68.9%	69.2%	69.5%	69.8%
4.8	4.6%	12.3%	20.2%	27.4%	33.7%	39.5%	44.4%	48.7%	52.6%	55.1%	57.4%	59.7%	61.9%	63.8%	65.6%	66.6%	67.2%	67.6%	67.9%	68.2%
5.1	4.3%	11.6%	19.3%	26.4%	32.5%	38.1%	42.9%	47.3%	51.2%	53.7%	56.1%	58.4%	60.5%	62.4%	64.1%	65.0%	65.6%	66.0%	66.3%	66.6%
5.4	4.1%	11.0%	18.4%	25.4%	31.4%	36.7%	41.5%	45.9%	49.9%	52.4%	54.8%	57.1%	59.2%	61.1%	62.6%	63.4%	64.1%	64.4%	64.7%	65.1%
5.7	3.9%	10.4%	17.6%	24.5%	30.3%	35.5%	40.2%	44.5%	48.6%	51.1%	53.5%	55.9%	57.9%	59.7%	61.1%	61.9%	62.5%	62.9%	63.2%	63.5%
6.0	3.7%	9.8%	16.9%	23.7%	29.4%	34.4%	38.9%	43.3%	47.3%	49.9%	52.3%	54.7%	56.7%	58.4%	59.6%	60.4%	61.1%	61.4%	61.7%	62.1%

Figure 8-4 An example of a two-variable data table using SS STORM.

An Excel pop-up window for running the two-variable data table feature is shown in upper part of Figure 8-4. In the table part of Figure 8-4, the most upper left corner cell contains the formula for estimating WWC performance, which is the percent pollutant removal (M_{rmvd} / M_{rff}) in this example. Except this cell, the values in the first row represent different sizes of S; and the values in the first column represent various levels of Q_{rls}. The main part of the table except the first row and column represent the whole range of pollutant removal performance based on the two design variables (S and Q_{rls}), which are listed in the first row and column. Each cell of this part is a result from one continuous SS STORM simulation using a specified S in the first row and a specified Q_{rls} in the first column respectively. To create the production function table shown in Figure 8-4, 400 continuous simulations were performed and took only a couple of seconds.

The production function was graphically derived using the data in the two-variable table. This is shown in Figure 8-5 wherein performance (% pollutant control) is expressed as a function of S and Q_{rls}. Interpolation techniques are

required to derive the 3-dimensional contour lines. It can be done within Excel but would require considerable effort. A number of graphic programs, e.g., Surfer or SigmaPlot, can create this kind of 3-D production function without any additional calculations. SigmaPlot was used to produce Figure 8-5.
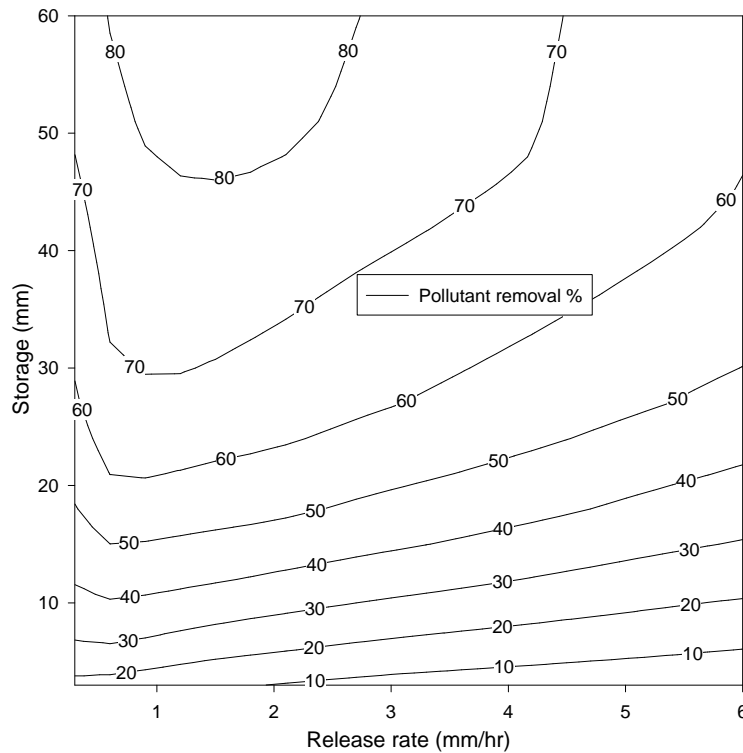
Figure 8-5 Production function of pollutant removal.

The production function summarizes the performance of the WWC in terms of "what if" analysis of the effects of changes in storage capacity and release rate on performance. The U-shaped isoquants at higher levels of control illustrate the tradeoff between slower and faster release rates. However, it doesn't tell us "what's best." To derive the most cost-effective solution, the life cycle cost of storage and release rate needs to be incorporated into the analysis as described in the next section.

Appling SS STORM for WWC Optimization
The SS STORM model can be readily integrated with spreadsheet optimization tools. An example of the WWC optimization procedure using the developed SS STORM is presented in Figure 8-6. In this example as described below, storage volume and a fixed release rate are the design variables to select for minimizing the cost of attaining a specified performance level:

Minimize: Cost $= f(Storage) = a \times S^b$
Subject to: Pollutant Mass Removal Rate (%) $\geq$ Targeted Performance Level (%)
Design Variables:Storage Volume, Release Rate

A power function is used for estimating the total cost and the two parameters for the cost function are assumed. The developed model can directly work with spreadsheet optimization tools, e.g., Excel-Solver add-ins, to find the best WWC strategy. Developing a graphical and/or analytical representation of system performance and cost relationships is not necessary.   Other input parameters, such as depression storage and infiltration rate of a catchment, can be used as additional design variables without developing any other program interface. About a year of time-series data are

used for performing a continuous simulation for this example. The entire optimization procedure can be done within just a couple of seconds. The setup for doing the optimization using the Excel Solver add-in from Frontline Systems is shown in Figure 8-6. In this example, the least costly way to remove 60% of the pollutant mass is to have a 20.6 mm of storage and a release rate of 0.89 mm/hr. These optimized values can be confirmed using the derived production function shown in Figure 8-5. The lowest point of the 60 % pollutant removal line in Figure 8-5 is the minimum cost option because the cost is a function of storage.

	A	B	C	D	E	F	G	H	I	J	K	L	M	N
1	Color Legend		Model Input Data						Modeling Results					
3	Description		Parameters			Monthly ET rate			Volume-based			Pollutant mass		
4	Input data		DS	10	(mm)	Month	(mm/d)		V_{prcp}	1771.9	(mm)	M_{rff}	99667	(mg/m^2)
5	Performance		f	1.2	(mm/hr)	1	1.516		V_{rff}	996.7	(mm)	M_{in}	64630	(mg/m^2)
6	Initial asumption		del-T	1	(hr)	2	1.786		V_{trt}	646.3	(mm)	M_{rmvd}	59800	(mg/m^2)
7						3	2		V_{bpsd}	350.4	(mm)	M_{out}	39867	(mg/m^2)
8			Plug-flow model			4	3.433		Capt%	64.8%		Rmvd%	60.0%	
9			$C_{out}=C_{in}*exp(-k*td)$			5	.387					Traget%	60%	
10			C_{in}	100	(mg/L)	6	5.400							
11			k	0.6	(/hr)	7	5.548							
12					Design	8	5.419		Cost function					
13			Str-Trt-Rls		variables	9	4.9		Cost=a*S^b					
14			S	20.6	(mm)	10	3.74		a	10000				
15			Q_{rls}		(mm/hr)	11	2.533		b	0.7				
16			Empty	23.0	(hr)	12	1.839		Cost	$82,993				

Target removal rate (callout for M9)
Minimize cost (callout for J16)

19	Continuous Simulation													
20	1	2	3	4	5	6	7	8	9	10	11	12	13	14
21	Total =	1771.9	341.0			996.67				646.3	350.4			59800
22	Prcp time series		Dry	ET	DS	Runoff	Storage-Release-Bypass					Pollutant Removal		
23	Time	Prcp	days	rate	avail		S_1	Rlsd	S_2	Trt	Bpsd	t_d	C_{out}	M_{rmvd}
24		(mm)	(d)	(mm/d)	(mm)	(mm)	(mm)	(mm)	(mm)	(mm)	(mm)	(hr)	(mg/L)	(mg/m^2)
25									0.00					
26	2001/01/04 02:00	0.51	0.00	1.516	10.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	100.00	0.00
27	2001/01/04 03:00	1.02	0.00								0.00	0.00	100.00	0.00
28	2001/01/04 04:00	0.76	0.00								0.00	0.00	100.00	0.00
29	2001/01/16 08:00	0.25	12.13								0.00	0.00	100.00	0.00
30	2001/01/20 03:00	0.51	3.75								0.00	0.00	100.00	0.00
31	2001/01/20 04:00	3.81	0.00								0.00	0.00	100.00	0.00
32	2001/01/20 13:00	7.62	0.33								0.00	0.13	92.49	1.75
33	2001/01/20 14:00	0.25	0.00								0.00	0.00	100.00	0.00
34	2001/01/20 15:00	0.25	0.00								0.00	0.00	100.00	0.00
35	2001/01/22 15:00	0.25	1.96								0.00	0.00	100.00	0.00
36	2001/02/04 18:00	0.51	13.08								0.00	0.00	100.00	0.00
37	2001/02/04 21:00	0.51	0.08								0.00	0.00	100.00	0.00
38	2001/02/19 21:00	0.25	14.96								0.00	0.00	100.00	0.00
39	2001/03/04 21:00	4.57	12.96								0.00	0.00	100.00	0.00
40	2001/03/04 22:00	8.13	0.00								0.00	0.84	60.44	59.34
41	2001/03/04 23:00	3.05	0.00	2.387	0.00	1.85	2.45	0.89	1.56	1.85	0.00	1.71	35.79	118.65

Solver Parameters V5.5

Set Cell: $J$16
Equal To: ○ Max ● Min ○ Value of: 0
By Changing Variable Cells: $D$14:$D$15
Subject to the Constraints: $M$8 >= $M$9
Standard GRG Nonlinear
Buttons: Solve, Close, Model, Options, Variables, Add, Change, Reset All, Delete, Help

Figure 8-6 Appling SS STORM for WWC optimization.

This process can be repeated for various assumed values of the targeted percent performance level to generate the final performance curve as shown in Figure 8-7.

Figure 8-7 Total cost as a function of percent control.

Summary and Conclusions

Every component of urban surface can work as a WWC system as shown in Figure 8-1: on-site depression storage/infiltration system and off-site storage-release system. Developing a physically sound process simulator is critically important for analyzing WWC alternatives. A continuous simulation model for rainfall-runoff and storage-treatment was described in this chapter. This SS STORM model is used to create a two-variable production function, which is the result of a great number of continuous simulations, and is linked directly to an optimizer to find the most cost effective WWC. More applications of the SS STORM are presented in Chapters 12 and 13.

Chapter 9 Cost Analysis

Introduction

An extensive cost analysis of wet-weather controls was done as part of our earlier research for EPA (Heaney et al. 2002). Sample et al. (2003) summarize this research. The purpose of this section is to update this earlier study with results from other studies that were not included in the 2002 report.

Additional Cost Information

EPA (1999) summarizes WWC cost data based on a literature review. Their results were not incorporated into our 2002 report. Also, an update of the cost estimates of the Stormwater Center (2000) provides other current cost data. The primary sources of data for these cost studies are the 1986 study of Wiegand et al. (1986) who evaluated 68 stormwater management ponds in the Washington, D.C. area. The Stormwater Center (2000) updated this earlier study with an evaluation of 73 stormwater practices in the Mid-Atlantic area using bond estimates, engineering estimates, and actual construction costs. The breakdown of total costs into the major components is shown in Table 9-1 (Stormwater Center 2000). They estimate that approximately one-third of the cost of stormwater controls is due to water quality protection.

Table 9-1 Relative importance of basic component costs for stormwater controls.

Component	Ponds	Sand Filters	Bioretention
Excavation/Grading	48%	21%	25%
Control Structure	36%	68%	50%
Appurtenances	16%	11%	25% *

*Includes landscaping costs.

The Stormwater Center (2000) has developed the following equations based on their 1996 studies, which are shown in Table 9-2.

Table 9-2 Capital cost predictive equations (Stormwater Center 2000).

Category	1996 Capital Cost Equation in $*	R^2
All Ponds	$C = 27.5V^{0.70}$	0.77
Dry ED Ponds	$C = 10.8V^{0.78}$	0.93
Bioretention	$C = 7.48V^{0.99}$	0.92

*Includes engineering design, construction, and contingencies. V is volume in cubic feet.

An updated summary of WWC cost information is shown in Table 9-3. This information provides general guidelines for estimating control costs, exclusive of land costs.

Current summaries of urban stormwater activities in the Pacific Northwest are available through a bi-monthly newsletter called Stormwater Treatment Northwest. Information on treatment costs is presented in their June 2002 issue. A summary of the range of unit cost per pound of suspended solids removed is shown in Table 9-4.

Table 9-3 Typical base capital construction costs for BMPs (EPA 1999).

BMP Type	Units	1997 Base Cost* Low	1997 Base Cost* High	Comment
Retention and detention basins	$/ft^3	$0.50	$1.00	15,000-150,000 ft^3 range
Constructed wetland	$/ft^3	$0.60	$1.25	Little data. 25% more than ponds
Infiltration trench	$/ft^3	$4.00	$4.00	Cost for a 100 foot long trench
Infiltration basin	$/ft^3	$1.30	$1.30	A 0.25 acre infiltration basin
Sand filter	$/ft^3	$3.00	$6.00	Range is due to different designs
Bioretention	$/ft^3	$5.30	$5.30	Relatively constant in cost
Grass Swale	$/ft^2	$0.50	$0.50	6 in. of storage
Filter Strip	$/ft^2		$3.00	7 in. of storage
Porous Pavement	$/acre	$50,000	$80,000	

*Base cost excludes the cost of land.

Table 9-4 Unit costs of stormwater treatment in terms of suspended solids removal (Stormwater Treatment Northwest 2002).

BMP	Number of projects	Cost, $/lb of TSS Removed Low	Cost, $/lb of TSS Removed High	Cost, $/lb of TSS Removed Average
Wet pond	5	$2.04	$14.96	$8.50
Wet vault	7	$4.30	$61.32	$32.81
O/W Separator	6	$2.81	$24.12	$13.47
Sand filter	4	$4.04	$26.05	$15.05
Swale	5	$0.50	$4.38	$2.44
StormFilter	1	$7.79	$7.79	$7.79
Vortex separator	1	$4.44	$4.44	$4.44

Conclusions

Relatively reliable information is available on the initial cost of manufactured WWCs and detention systems that are constructed with concrete and steel. Operating cost information can be obtained by running a continuous simulation model that allows the long-term performance of the control to be estimated. Alexander (2003) used this approach to compare the life cycle cost of conventional vs. low impact development. The most critical element in the cost of WWCs is how land is valued. Most estimates ignore land cost since it is highly variable. Heaney et al. (2002) provide guidelines on how land valuation can be done. Site-specific analysis of land costs is essential since it is often a matter of policy as to whether land costs are included.

Chapter 10 Primer on Optimization

Introduction

The previous chapters describe spreadsheet-based simulation tools that can be used to do trial and error evaluations of WWC options. A key principle of this analysis is that control can occur anywhere by purchasing better parameters for the simulation model, e.g., a lower runoff coefficient, a higher infiltration rate. Given this broader framework, the number of decision variables increases considerably. SS based optimization tools are essential to have an efficient way to compare the relatively large number of alternatives that are now available. The purpose of this chapter is to provide a primer on how to use these tools.

Primer on Spreadsheet Optimization Tools

The purpose of this primer is to introduce the various options for doing optimization using the Excel Solver and its add-ins. Frontline Systems, the developer of the Excel Solver, has add-in products that greatly extend the capability of the basic Solver, as shown in Table 10-1 (http://www.solver.com). All of the optimization problems that are addressed in this report are solved using Version 5.0 of the Premium Solver Platform. Two useful references on optimization methods that include a student version of Premium Solver are listed below:

Ragsdale, C.T. 2003. *Spreadsheet Modeling and Decision Analysis: A Practical Introduction to Management Science, 4rd Ed.*, South-Western College Publishing.
Winston, W., Albright, S.C. and M. Broadie. 2003. *Practical Management Science, 3nd Ed.*, Duxbury Press.

They are excellent sources of information regarding optimization techniques and tips related directly to the spreadsheet Solvers. The documentation that comes with the software is also very helpful.

Until the inclusion of optimizers within spreadsheets, it was relatively difficult to set up and solve optimization problems because of the required mathematical sophistication and the relatively cumbersome computing environment. Traditionally, the entire optimization problem was structured with the objective function and constraint set directly coupled. All of the components of the optimization problem had to be well behaved mathematically for the algorithm to work. Recent breakthroughs have made it much easier to deploy optimization methods. The development of ESs such as genetic algorithms, tabu search, and simulated annealing allow the simulator ("what-if" model) and the optimizer ("what's best" model) to be decoupled and remove many, if not all, of the traditional restrictions on the mathematical structure of the problem. Thus, optimization can be viewed as simply automating the "what-if" trial and error approach by having the optimizer guide the repetitive simulation runs towards a pre-specified goal.

Many of the papers in the literature on using ESs present the authors' personalized, domain dependent, version that is not transferable to other users. This is a major limitation on the use of ESs. During the past several years, we have relied on commercially available ES software from companies such as Frontline Systems, e.g., see Lippai et al. (1999). These ES codes are directly integrated with classical optimization methods and are developed by leaders in the field. We have had excellent experience with them including using them in undergraduate and graduate classroom

environments for the past several years. Thus, we can strongly recommend them as being usable by practicing professionals. The examples presented below introduce the reader to these methods. The best way to learn them is to follow along using a SS.

Table 10-1 Features and performance of Frontline Systems' Solver (www.solver.com).

FEATURES AND PERFORMANCE	Standard Excel Solver	Premium Solver	Premium Solver Platform
Built-in Engines:			
Simplex Linear Solver	LP Only	LP Only	LP/Quadratic
GRG Nonlinear Solver	Yes	Yes	Yes (fastest)
Mixed-Integer Solver	Yes	Yes (fast)	Yes (fastest)
Evolutionary Solver	No	Yes	Yes
Interval Global Solver	No	No	Yes
Field-Installable Solver Engines	No	No	Yes
Problem Size:			
Linear Variables x Constraints	200 x 200	1000 x 8000	2000 x 8000
Nonlinear Variables x Constraints	200 x 100	400 x 200	500 x 250
Non-Smooth Variables x Constraints	N/A	400 x 200	500 x 250
Solver Engine Variables x Constraints	N/A	N/A	Unlim. x Unlim
Speed (approximate):			
Problem Setup	1X	1-50X	2-100X
Linear Problems	1X	3X	6X
Nonlinear Problems	1X	1X	7-15X
Mixed-Integer Problems	1X	10-20X	10-50X
Non-Smooth Problems	N/A	1-10X	1-10X
Solver Engines (On Various Problems)	N/A	N/A	10-1000X
Global Optimization:			
Non-Smooth Problems (Evolutionary)	No	Yes	Yes
Nonlinear Problems (Interval Global)	No	No	Yes
Nonlinear Problems (Multistart)	No	No	Yes
Solver Engines (Several Approaches)	No	No	Yes
Constraint Programming:			
Built-In Engines (alldifferent constraint)	No	Yes	Yes
Solver Engines (alldifferent constraint)	No	No	Yes
Solver Reports:			
Answer Report	Yes	Yes	Yes
Sensitivity Report	Yes	Yes	Yes
Limits Report	Yes	Yes	Yes
Dependents Report	No	No	Yes
Scaling Report	No	No	Yes
Solutions Report (Interval Global)	No	No	Yes
Linearity Report	No	Yes	Yes
Feasibility Report	No	Yes	Yes
Population Report	No	Yes	Yes
Report Outlining	No	Yes	Yes
Block Selection and Comments	No	Yes	Yes
Solver Engine Custom Reports	No	No	Yes

Categories of Optimization Problems

Simple example problems are used to introduce the reader to optimization approaches that are directly relevant to urban stormwater problems. A total of nine categories of problems are evaluated using a simple example. They collectively embody the suggested strategy for using Solver under various circumstances. The matrix of three types of variables and three types of mathematical structure are shown in Table 10-2 where the options are linear program (LP), nonlinear program (NLP), and ES.

Table 10-2 Categories of optimization problems to be evaluated.

Variables	Linear	Smooth Nonlinear	Non-smooth Linear
Continuous	LP	NLP	ES
Consecutive integers	LP	ES/NLP	ES
Non-consecutive integers	LP	ES	ES

By way of definition, continuous variables can assume an infinite number of values and are usually written as $x \geq 0$. Consecutive integers are discrete variables, e.g., $x = \{3, 4, 5, 6\}$. Discrete variables that are simple multiples of each other can also be considered to be consecutive since they are consecutive integers multiplied by a constant, e.g., 2, yielding $\{6, 8, 10, 12\}$. Non-consecutive integers are an irregular set of integers, e.g., pipe sizes with diameters of $\{12, 15, 18, 24, 30\}$. Non-consecutive integers can be expressed as tabular functions of consecutive integers. For example, the set of pipe diameters can be found from a lookup table, as illustrated in Table 10-3. In this case, the optimization variable can take on integer values in the following range $x = \{1, 2, 3, 4, 5\}$.

The corresponding value of the variable is found using a lookup function of the form:

VLOOKUP(x, *range*, *column*)

Table 10-3 Example of a lookup table for non-consecutive integer variables.

Variable	Corresponding integer
1	10
2	12
3	15
4	18
5	24
6	30

If a lookup table is used, then the problem is non-smooth and the ES must be used. Alternatively, the problem can be formulated using binary (0,1) variables. Let x_i = value of ith variable where $i = 1, 2,…, 6$.

Let x_i = binary (0,1) i^{th} option, $i = 1, 2,…, 6$, and

V_i = value of the ith option.

Then, the binary programming formulation is as follows:

$$x = \sum_i x_i V_i \text{ , and} \qquad\qquad \textbf{Equation 10-1}$$

$$\sum_i x_i = 1 \qquad\qquad \textbf{Equation 10-2}$$

Thus, the calculation picks exactly one of the six options. The binary programming option is preferable for linear programs with integer variables. The lookup table option is probably better for nonlinear integer problems.

The distinction of whether the model is linear or nonlinear is fairly simple. If the model contains one or more nonlinear terms, it is nonlinear. The only exception is when the variable is quadratic, in which case it can be solved using LP methods. A problem is non-smooth if it contains logical statements such as IF or LOOKUP. Logical statements can be removed by replacing them with binary variables. This option is attractive if the problem is otherwise linear.

Several solved examples are presented below to illustrate these cases.

Examples

It is best to load up the example files and follow along with the text shown below that describes each example.

Linear program with continuous variables

Example 10-1 Simple Low Impact Development Problem
A single parcel of land is required to have an average of 6 cm of soil storage on its pervious area. Two land use options are available: forest or grass. The total land area is 100 m². At least 25% of the area must be forest. Forest has a storage capacity of 0.1 meters while grass has a storage capacity of 0.03 meters. The unit cost of forest is \$15/m² while the unit cost of grass is \$8/m². What mix of land uses minimizes the cost of meeting these requirements?

This problem can be solved using linear programming as shown below:

$$\text{Minimize} \quad Z = \$15F + \$8G$$

Subject to: $F + G = 100$ (land area)

$F \geq 25$ (at least 25% of land must be forest)   **Equation 10-3**

$0.1F + 0.03G \geq 6$ (total storage must exceed 6 m³)

$F, G \geq 0$
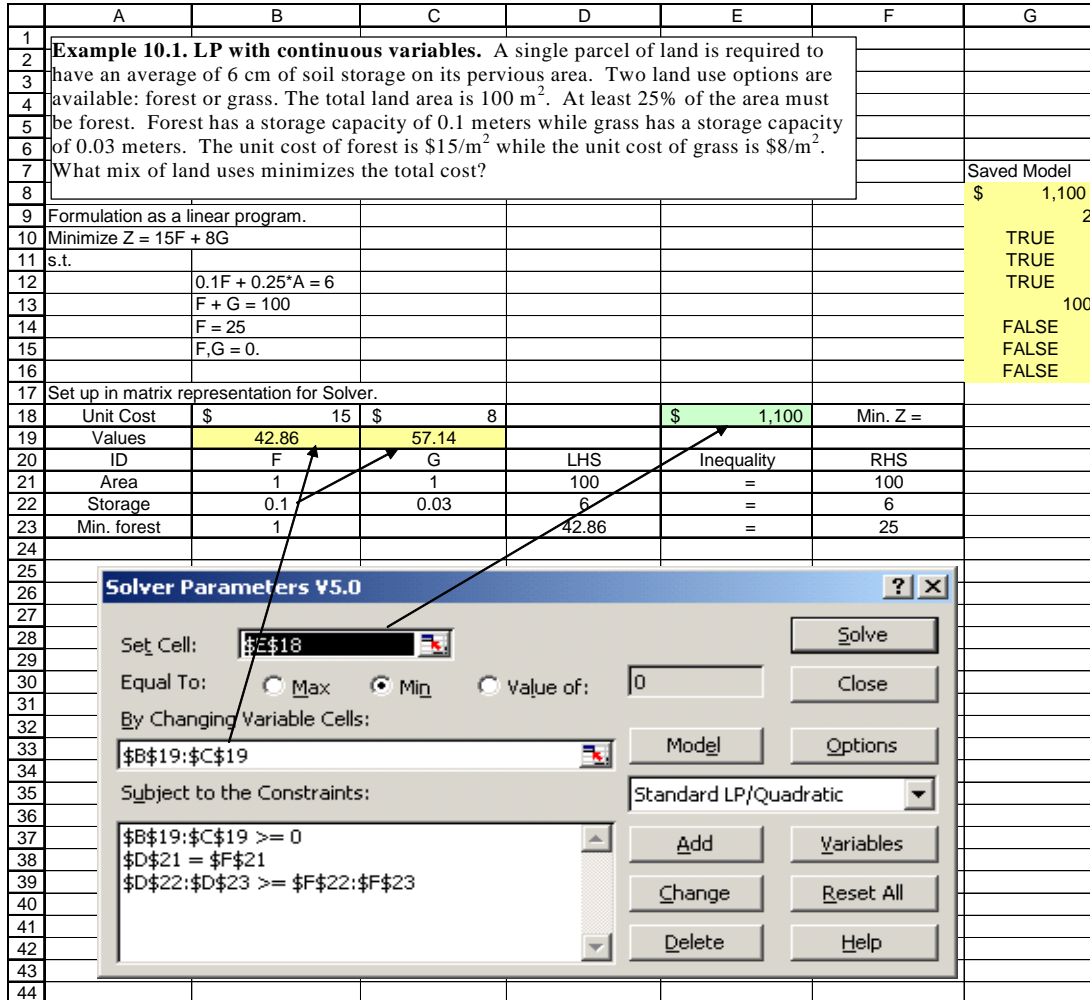
Figure 10-1 Spreadsheet formulation of Example 10-1.

The SS solution for a simple two-variable problem is shown in the SS file, Example 10-1, LP with continuous variables, and presented in Figure 10-1. The problem is to find the least cost combination of forest and grass that satisfy the constraints at a minimum cost. It is convenient to set up optimization models in matrix form as shown in Example 10-1. Solver is an option under Tools. The sumproduct function is a very handy Excel formula to use in setting up optimization problems. For example, the value of the objective function, Z, is calculated using the equation, $Z = 15F + 8G$. This equation can be entered term-by-term, but it is much quicker to use sumproduct(array 1, array 2). The resource utilization is represented as the left hand side (LHS) and it is compared to the right hand side (RHS) availabilities. If the inequality is the same for constraints, they can be entered on one line as illustrated for the two adjacent $\geq$ conditions. This option is chosen because this problem is a linear program. The resulting least cost solution from Solver is \$1,100 by using 42.87 m^2 of forest and 57.14 m^2 of grass. This solution satisfies all of the constraints, i.e., the total land sums to 100.00, 6 m^3 of storage are provided, the 42.87 m^2 of forest easily exceeds the minimum of 25 m^2, and both variables are non-negative. This example illustrates an option of source control using on-site infiltration. Rather than accepting land use as a given, the analysis shows how land uses can be blended to attain the required on-site storage.

It is helpful to solve this problem graphically, as shown in Figure 10-2 to illustrate the idea of a feasible region and how the objective function can be moved to the optimal point on this feasible region. The feasible region is the convex set located above the three constraint lines. A set is convex if the line connecting any two points in the set lies within the set. The problem is solved graphically by plotting a trial value of the objective function and then moving parallel to it until the least costly feasible solution is found. Because the problem is linear and the variables are continuous, the least cost solution will occur at one or more corners of the feasible region.

Linear programming codes are very fast and reliable, and are the best case scenario as far as ease of optimization goes. Their primary limitation is that many problems cannot be reduced to a set of linear relationships with continuous variables. Key complexities include:

1. The requirements that some or all of the variables be integers.
2. The inclusion of nonlinear relationships in the formulation.
3. The inclusion of logical statements such as if, choose, and/or lookup in the formulation.

Each of these complications requires a different type of optimization procedure, as will be described in the next examples.
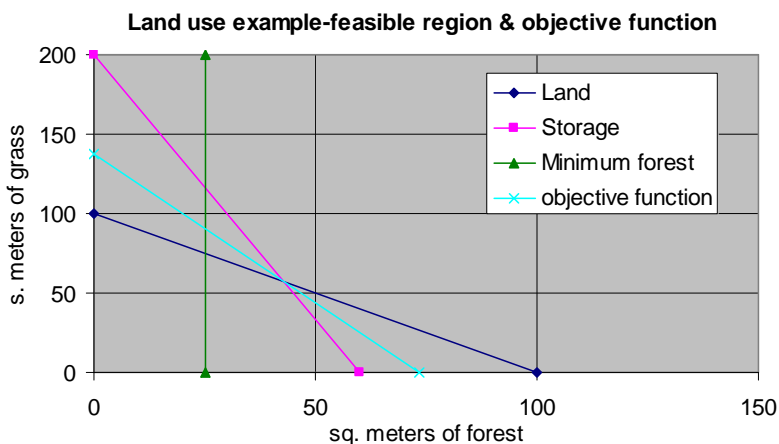
Figure 10-2 Linear program with continuous variables.

Linear program with consecutive integer variables
It may be more realistic to require that some or all of the variables assume integer values. This seemingly innocuous requirement makes the problem much more difficult for the computer to solve. However, it is very simple to set up

the problem. A popular approach is to ignore the integer requirement and hope that the answer is an integer solution. This is the best-case scenario. If the solution doesn't satisfy the integer requirements, then the algorithm continues until an optimal integer solution is found. The non-integer solution represents a lower bound if the objective function is being minimized or an upper bound if a maximization objective is being used. Only a minor modification is required in Solver. However, the user should be aware that a different, much less efficient and reliable, algorithm is being used. Example 10-2, shown in Figure 10-3, is the same as Example 10-1 except that the variables are required to be continuous integers. The resulting solution is Z = $1,101 with forest = 57 and grass = 43, a simple round-off of the original solution. In this case, the total cost increases only by one unit.

	A	B	C	D	E	F	G	H	I
1	**Example 10.2. LP with consecutive integer variables**								
2	A single parcel of land is required to have an average of 6 cm of soil storage on its								
3	pervious area. Two land use options are available: forest or grass. The total land area								
4	is 100 m^2. At least 25% of the area must be forest. Forest has a storage capacity of								
5	0.1 meters while grass has a storage capacity of 0.03 meters. The unit cost of forest is								
6	$15/m^2 while the unit cost of grass is $8/m^2. All variables are consecutive integers.								
7	Find the least cost solution.								
8									
9	Formulation as an integer linear program.						Saved Model	
10	Minimize Z = 15F + 8G							$ 1,101	
11	s.t.							2	
12		0.1F + 0.03G = 6						TRUE	
13		F + G = 100						TRUE	
14		F = 25						TRUE	
15		F,G = 0.						TRUE	
16		F,G integers						100	
17								FALSE	
18	Unit Cost	$ 15	$ 8		$ 1,101	< Min. Z =		FALSE	
19	Values	43.00	57.00					FALSE	
20	ID	F	G	LHS	Inequality	RHS			
21	Area	1	1	100	=	100			
22	Storage	0.1	0.03	6.01	=	6			
23	Min. forest	1		43.00	=	25			

Solver Parameters V5.0

Set Cell: $E$18

Equal To: ○ Max ● Min ○ Value of: 0

By Changing Variable Cells:
$B$19:$C$19

Subject to the Constraints:
$B$19:$C$19 = integer
$B$19:$C$19 >= 0
$D$21 = $F$21
$D$22:$D$23 >= $F$22:$F$23

Standard LP/Quadratic

[Solve] [Close] [Model] [Options] [Add] [Variables] [Change] [Reset All] [Delete] [Help]

Figure 10-3 Spreadsheet formulation of Example 10-2.

Linear program with non-consecutive integer variables
Example 10-3 is the same as Example 10-2 except that the variables are required to be non-consecutive integers as shown in Figure 10-4. Two options exist for handling this case: 1) Use binary variables and solve the problem using the LP option, or 2) Use a lookup table to define the variables and solve the problem using the ES. Each of these methods is described below.

Binary variable option
First, the problem is solved using binary programming, a special case of integer programming. For example, assume

that forest land use can only take on the values shown in Table 10-4. This problem can be set up as a binary programming problem by replacing the single variable, *F*, with six variables that represent the choices for *F*.

	A	B	C	D	E	F	G	H	I
63	A. Solution as a binary linear programming problem.								
64									
65	Unit Cost	$ 15	$ 8		$ 1,150	< Min. Z =		Saved model	
66	Values	50.00	50.00					$ 1,150	
67	0	0	0	0				12	
68	20	0	0	30				TRUE	
69	50	1	0	40				TRUE	
70	60	0	1	50				TRUE	
71	70	0	0	80				TRUE	
72	100	0	0	100				100	
73	Total	1	1					FALSE	
74	ID	F	G	LHS	Inequality	RHS		FALSE	
75	Area	1	1	100	=	100			
76	Storage	0.1	0.03	6.5	=	6			
77	Min. forest	1		50	=	25			

Solver Parameters [? X]

Set Cell: $E$66
Equal To: ○ Max ● Min ○ Value of: 0
By Changing Variable Cells:
$B$68:$C$73

Subject to the Constraints: Standard Simplex LP

$B$68:$C$73 = binary
$B$74:$C$74 = 1
$D$76 = $F$76
$D$77:$D$78 >= $F$77:$F$78

[Solve] [Close] [Guess] [Options] [Add] [Standard] [Change] [Reset All] [Delete] [Help]

	A	B	C	D	E	F
95	Check by total enumeration					
96		Forest	Grass	Feasible?	Z	
97		0	100	No		
98		20	80	No		
99		50	50	Yes	$ 1,150	
100		60	40	Yes	$ 1,220	
101		70	30	Yes	$ 1,290	
102		100	0	Yes	$ 1,500	

	A	B	C	D	E	F
104	B. Solution using evolutionary algorithm for this linear program.					
105						
106	Unit Cost	$ 15	$ 8		$ 1,150	< Min. Z =
107	Dec. Var.	2.00	3.00			
108	Value	50.00	50.00			
109	ID	F	G	LHS	Inequality	RHS
110	Area	1	1	100	=	100
111	Storage	0.1	0.03	6.5	=	6
112	Min. forest	1		50	=	25

Solver Parameters [? X]

Set Cell: $E$107
Equal To: ○ Max ● Min ○ Value of: 0
By Changing Variable Cells:
$B$108:$C$108

Subject to the Constraints: Standard Evolutionary

$B$108:$C$108 <= 5
$B$108:$C$108 = integer
$D$111 = $F$111
$D$112:$D$113 >= $F$112:$F$113

[Solve] [Close] [Guess] [Options] [Add] [Standard] [Change] [Reset All] [Delete] [Help]

	A	B	C	D	E	F
129	A lookup function, VLOOKUP(value, range, column) is used to convert the integer variables to their values.					
130	Forest option	m^2 of forest	Grass option	m^2 of grass		
131	0	0	0	0		
132	1	20	1	30		
133	2	50	2	40		
134	3	60	3	50		
135	4	70	4	80		
136	5	100	5	100		

Figure 10-4 SS formulation as a binary or evolutionary programming problem.

Table 10-4 Discrete choices for the forest option.

Forest option	Area, m^2
0	0
1	20
2	50
3	60
4	70
5	100

Let F_i = binary (0,1) ith option for forest, i = 1,2,…,6 and

A_i = area of the ith option for forest, i = 1,2,…,6 (m^2).

Then, the binary programming formulation is as follows:

$$F = \sum_i F_i A_i \text{ , and}$$ **Equation 10-4**

$$\sum_i F_i = 1$$ **Equation 10-5**

This formulation requires that Solver picks exactly one of the six discrete choices.

Using the binary variable option, the number of decision variables has gone from 2 to 12. However, relatively fast integer programming codes can be used to solve the problem.

Evolutionary Programming formulation
It is simple to set up the revised problem using a lookup table in Excel. The decision variables are the consecutive integers from 0 to 5. The value of the variable is then found using the VLOOKUP function in Excel where the structure of the function is:

VLOOKUP(*cell*, *table range*, *column*)

For example, if the cell value is 3, VLOOKUP returns the value of 60 based on the data in Table 10-4. The key disadvantage of using a lookup table is that this logical function is not acceptable to the classical optimization codes since it is a non-smooth function. Fortunately, the Frontline add-in ES can be used to solve this problem. Disadvantages of the ES include slower convergence and no guarantee that the final solution is the optimal solution. The Excel SS that solves this problem is shown in Figure 10-4.

In the case, where the problem is linear, it appears to be quicker to use the binary variable option since the LP is faster and more reliable than the ES.

Nonlinear program with continuous variables
The original Example 10-1 is now changed to include a nonlinear condition in the constraint on storage as shown below as Example 10-4:

Minimize $Z = \$15F + \$8G$

Subject to: $F + G = 100$ (land area)

$F \geq 25$ (at least 25% of land must be forest) **Equation 10-6**

$0.1F + 0.03G^{0.5} \geq 6$ (total storage must exceed 6 m^3)

$F, G \geq 0$

With the nonlinear term, the Generalized Reduced Gradient (GRG) Nonlinear option is the preferred solution procedure, although the LP/Quadratic Programming Solver works for this simple problem. The optimal solution is Z = \$1206 with $F = 58.06$ and $G = 41.94$. More complex nonlinear problems can be challenging to solve since there is no guarantee that the solution is a global optimum and not just a local optimum. The Premium Solver Platform has a helpful feature that solves the problem from different starting points in order to reduce the risk of selecting a local optimum. The details are described in the Example 10-4 shown in Figure 10-5.

Nonlinear program with discrete variables-binary or evolutionary programming

Example 10-3 is solved for the case where only selected integer choices are admissible and the problem is nonlinear. This situation occurs frequently in design, e.g., pipe sizes in inches are 10, 12, 15, 18, 24, etc. In this case, the problem can be set up using binary variables or using lookup tables, as was done in Example 10-3. However, now the problem is also nonlinear. The Solver may not converge to a solution if the binary variable option is used as part of a NLP. The ES works much better. Thus, the suggested strategy in this case is to use the ES option with a relatively restricted number of choices in order to keep the run times reasonable.
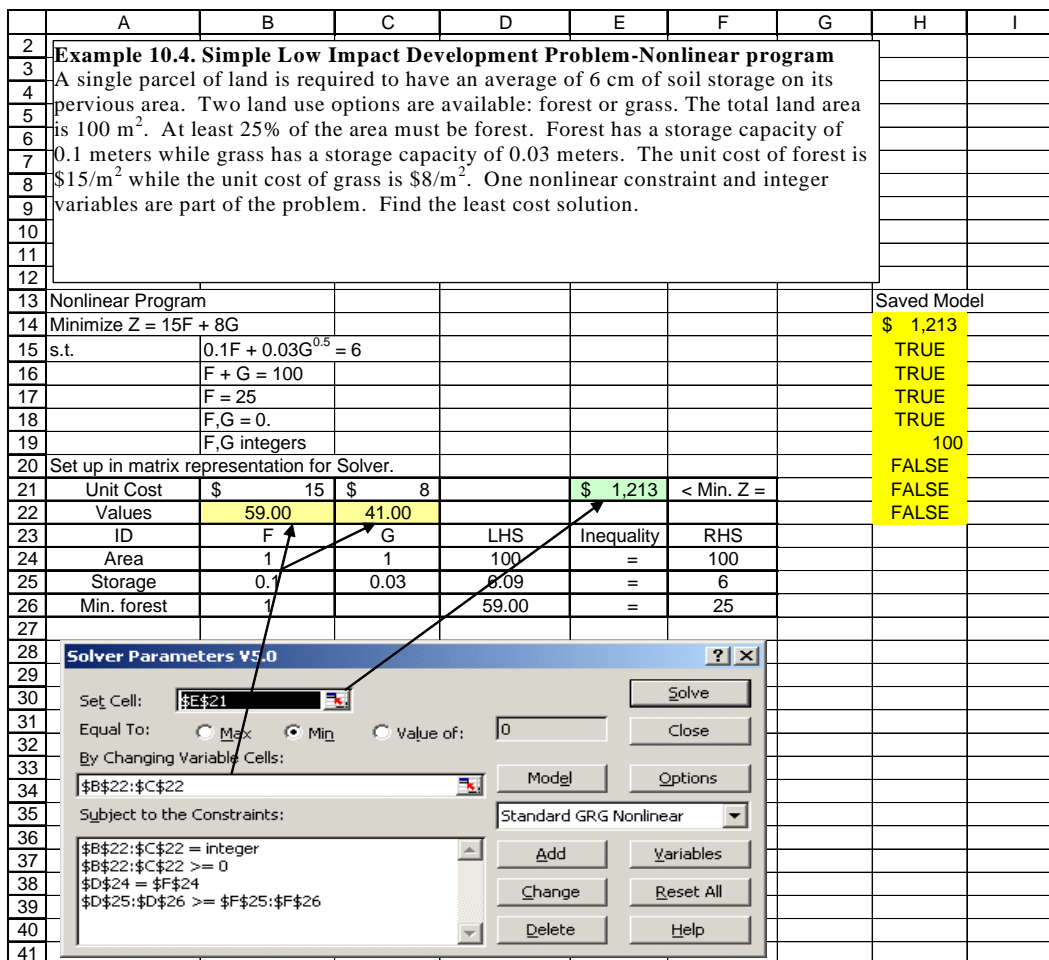
Figure 10-5 Spreadsheet formulation of Example 10-4.

Non-smooth, nonlinear problems with continuous or integer variables

In the most general case where the process simulator contains multiple non-smooth functions such as IF statements and the problem is nonlinear, the ES option needs to be used. Unlike classical optimization where continuous variables are preferable, discrete variables are easier for the ES. Thus, a good strategy is to discretize the continuous variables and restrict the choice set to a relatively small number of options. For example, pipe slope is a continuous variable in the sewer optimization problem. The ES will continue to search for better pipe slopes to an answer with several significant figures, e.g., $S = 0.001342968$. However, pipe slopes cannot be constructed to this accuracy, so this answer would be rounded to $S = 0.013$. A better computational option is to set up a lookup table with discrete slopes, and then the ES can choose from among those slopes. The range of slopes can then be fine tuned as needed.

A good strategy for finding a good initial solution to an ES is to solve the continuous NLP and round off the answer as an initial solution to the ES. An easy way to do this is to set up the ES and then invoke the option of ignoring integer variables in order to get the continuous solution.

Summary and Conclusions

These simple examples can be used to obtain a basic introduction into the various optimization options that are available. The reader should be familiar with these examples, as they cover the spectrum of optimization methods that are available. These methods can be used to solve a variety of wet-weather problems including storm sewer design outlined in the next chapter and BMP optimization described in Chapters 12 and 13.

Chapter 11 Storm Sewer Optimization

Introduction
The purpose of this chapter is to describe how optimization techniques can be used to solve the standard storm sewer design problem. The Maple/Redwood block of Happy Hectares, which is shown in Figure 11-1, will be used as the case study.
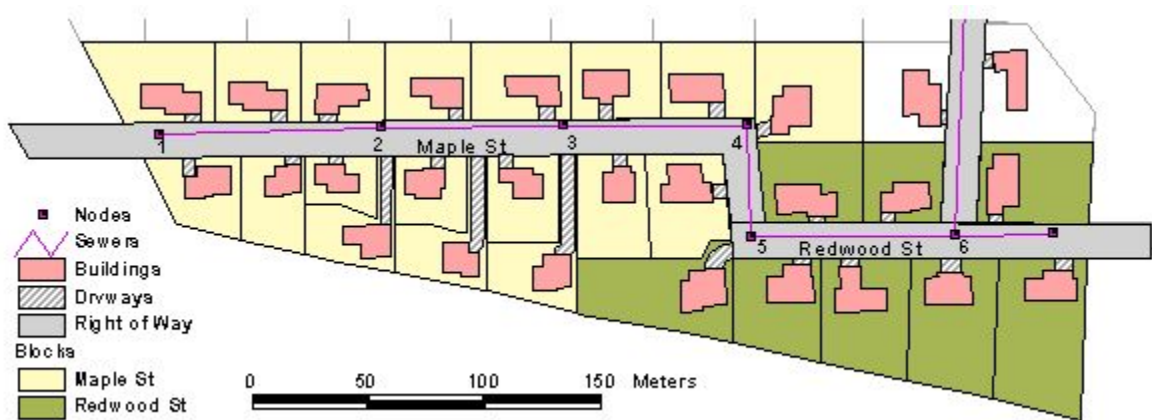
Figure 11-1 Maple/Redwood block of Happy Hectares.

Storm Sewer Design Optimization for Maple/Redwood Block
The spreadsheet model shown in Figure 11-2 is used to describe the methodology for optimizing storm sewer systems. The flow pattern is from node 1 to node 6. The two categories of decision variables are to select the diameter and slope of the pipe if one is needed. The cells are color coded into five categories:

1. Unshaded-input data
2. Yellow-decision variable
3. Blue-calculated value
4. Green-constraint condition
5. Objective function

All of the calculations are placed in three tables-Hydrologic, Hydraulic, and Cost. Two lookup tables are included for pipe costs and slope choices. Each of these tables is described below:

JPH   Example 11.1.   Jun-05

Storm Sewer Design Using Nonlinear Programming

Solve using the ES.

Design of five sections of the Maple/Redwood section of Happy Hectares

Mannings n	0.01	Sground =	0.0025
Frequency, years	5		
Runoff coef., impervious	0.95		
Runoff coef., DCIA	0.95		
Runoff coef., pervious	0.3		

Assumptions

Assume pipe velocity in ft./sec. =	3	
Time, minutes, to upper end of section =	20	Entire area
Time, minutes, to upper end of section =	10	DCIA

Color Code

Code	Meaning
Unshaded	Input data
Yellow	Decision variable
Blue	Calculated value
Green	Constraint check
Red	Objective function

Assume a five year recurrence interval, r = 5.

$$i = \frac{a + b\ln(r)}{(t+c)}$$

a	b	c	r
48.9155	29.4164	12.0989	5.0000

Conventional Rational Method: Q = CiA   Q = C*ADCIA   Lloyd-Davies Method

Table 1. Hydrologic computations for the 5 year storm

1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23
Nodes From	To	Length feet	Area acres	Cum. Acres	Total % Imperv.	Total Imp. Area, IA acres	Cum. IA acres	Perv. Area, PA acres	Cum. Perv. acres	Cum. Wghtd. RC	% Dir. Con. Imp.	Dir. Con. CA	Cum. Dir. Conn. CA	Time to Upper end, min.	Time in Section min.	Total Time min.	Avg. rain Intensity in./hr.	Peak Dis., TA cfs	Total time min.	Avg. rain int. DCIA in./hr.	Peak Dis., DCIA cfs	Design Peak cfs
1	2	279	2.30	2.30	35.0%	0.80	0.80	1.49	1.49	0.53	0.25	0.57	0.57						10.00			
2	3	299	2.40	4.70	50.0%	1.20	2.00	1.20	2.69	0.58	0.4	0.96	1.53	20.00	1.66	21.66	2.85	7.73	11.66	4.05	5.90	7.73
3	4	272	2.20	6.89	40.0%	0.88	2.88	1.32	4.01	0.57	0.3	0.66	2.19	21.66	1.51	23.17	2.73	10.76	13.17	3.81	7.94	10.76
4	5	148	1.51	8.40	30.0%	0.45	3.33	1.06	5.07	0.56	0.2	0.30	2.49	23.17	0.82	23.99	2.67	12.50	13.99	3.69	8.74	12.50
5	6	325	2.20	10.60	60.0%	1.32	4.65	0.88	5.95	0.59	0.5	1.10	3.59	23.99	1.80	25.80	2.54	15.76	15.80	3.45	11.78	15.76

Minimum depth of cover, ft. = 4

Mannings Equation

$$Q_f = \frac{0.463}{n} D^{8/3} S^{1/2}$$

Velocity constraints, ft./sec.
Min. 3
Max. 10

Table 2. Hydraulic pipe design computations for the 5 year design event.

1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20
Nodes From	To	Slope ID	Trial Slope ft./ft.	Pipe ID	Trial Pipe Diameter in.	Diam. Check Diam.? Non-neg.	Length feet	Ground Elev. Upper Feet	Ground Elev. Lower Feet	Upstream depth to top of pipe Feet	Downstream depth to top of pipe Feet	Min. Cover Check Feet	Qf cfs	Required Q cfs	Capacity Check Qf>=Q?	Q/Qf	V/Vf	Full Velocity Vf = Q/A ft./sec	Design Velocity V ft./sec
1	2						279	207.0	206.7										
2	3	1	0.005	3	18		299	206.7	206.7	202.7	201.2	5.5	9.65	7.73	1.92	0.80	1.02	5.46	5.57
3	4	2	0.0075	3	18	0	272	206.7	207.3	201.2	199.2	8.2	11.82	10.76	1.06	0.91	1.04	6.69	6.94
4	5	3	0.01	3	18	0	148	207.3	207.3	199.2	197.7	9.7	13.65	12.50	1.15	0.92	1.04	7.72	8.01
5	6	4	0.015	3	18	0	325	207.3	207.3	197.7	192.8	14.5	16.72	15.76	0.96	0.94	1.03	9.46	9.75

Condition >= 0   Condition, >= 4   Condition >= 0   Min. Condition >= 3   Max. Condition <= 10

Table 3. Least-cost design for the 5 year storm

1	2	3	4	5	6	7	8	9	10	11	12	13
Nodes From	To	Upper Invert Elev. Feet	Lower Invert Elev. Feet	Pipe Cost $	Avg. Depth ft	Avg. Width ft	Total Volume yd^3	Excavation Cost $/yd^3	Total $	Manhole Depth ft	Manhole Cost $	Total Cost $
1	2									5.5		
2	3	201.19	199.70	$ 8,061	8.25	3.00	274	6.14	$ 1,680	7.0	$ 2,951	$ 12,692
3	4	199.70	197.66	$ 7,352	11.09	3.00	335	6.14	$ 2,060	9.7	$ 4,000	$ 13,412
4	5	197.66	196.18	$ 3,986	14.53	3.00	238	6.14	$ 1,463	11.2	$ 4,565	$ 10,014
5	6	196.18	191.31	$ 8,770	18.44	3.00	665	6.14	$ 4,085	16.0	$ 6,396	$ 19,251
	Total			$ 28,169					$ 9,288		$ 17,912	$ 55,369

Assume pipe cost = 1.5*D in inches.   Manhole cost = $482H^{0.9317}$
$/ft.-in. diam. = $ 1.50

v = Q/A

Approximating polynomial for finding v/vf

α0	α1	α2	α3	α4
0.219848	3.823954	-9.61782	11.76443	-5.195203

$$v/v_f = \alpha_0 + \alpha_1(q/q_f) + \alpha_2(q/q_f)^2 + \alpha_3(q/q_f)^3 + \alpha_4(q/q_f)^4$$

Saved Model $ 55,369

Pipe ID	Diam.
0	10
1	12
2	15
3	18
4	21
5	24
6	30

Slope ID	Value
0	0.0025
1	0.005
2	0.0075
3	0.01
4	0.015
5	0.02
6	0.025
7	0.03
8	0.035
9	0.04
10	0.05

$ 55,369
8
TRUE
TRUE
TRUE
TRUE
TRUE
TRUE
TRUE
TRUE
TRUE
1000
FALSE
TRUE
FALSE

Solver Parameters V5.0

Set Cell: $M$52

Equal To: ○ Max ● Min ○ Value of: 0

By Changing Variable Cells:
$C$34:$C$37, $E$34:$E$37

Subject to the Constraints:
$C$34:$C$37 <= 10
$C$34:$C$37 = integer
$C$34:$C$37 >= 0
$E$34:$E$37 <= 6
$E$34:$E$37 = integer
$E$34:$E$37 >= 0

Standard Evolutionary

Buttons: Solve, Close, Model, Options, Add, Change, Reset All, Variables, Delete, Help

Figure 11-2 Spreadsheet formulation of sewer design problem

Hydrologic table

The purpose of the Hydrology table is to calculate the peak discharge that enters the system at the top of the subcatchment. The details of the calculation are shown in Table 11-1. The design peak flow is the maximum of the peak flow for the total area and the peak flow for the directly connected impervious area.

Hydraulic table

Using the estimated peak discharge entering nodes 2 to 5, trial guesses of the pipe slopes and diameters are entered to see if a technically feasible solution can be found. A solution is technically feasible if the following four conditions are satisfied:

1. downstream diameter is ≥ upstream diameter.
2. minimum depth of cover is four feet.
3. pipe capacity ≥ the design inflow.
4. velocity in the pipe is between 3 and 10 feet per second.

Details of the calculations are described in Table 11-2.

Cost table

In the final table, the total cost of each trial scenario is calculated.

$$C(\text{total}) = C(\text{pipe}) + C(\text{excavation}) + C(\text{manhole}) \qquad\qquad \textbf{Equation 11-1}$$

Details of the calculated costs in the Cost Table are presented in Table 11-3.

Table 11-1 Description of hydrologic calculation performed in Example 11-1.

Column Number:
1. Upstream node number.
2. Downstream node number.
3. Length of section (feet).
4. Drainage area (acres).
5. Cumulative drainage area (acres).
6. Total percent imperviousness.
7. Total impervious area (acres) = IA = col. 4*col. 6.
8. Cumulative impervious area.
9. Pervious area (acres).
10. Cumulative pervious area (acres).
11. Cumulative area weighted runoff coefficient.
12. % directly connected impervious area.
13. Directly connected impervious area (DCIA) (acres).
14. Cumulative DCIA (acres).
15. Time to upper end of section. Assumed to be 20 minutes for entire area or 10 minutes for DCIA. Actual times are much shorter but longer times are used to avoid using relatively high average intensities from the intensity-duration-frequency curves.
16. Time in section = L/v. A constant $v = 3$ ft/sec is assumed.
17. Total time (minutes). Equals the time to upper end of section (column 15) plus the time in the current pipe section (column 16).
18. Rainfall intensity (in./hr). Based on an equation fitted to the intensity/duration/frequency information for Boulder CO. (see equation at top of SS worksheet).
19. Peak discharge (cfs) for the total area. Cumulative total acres multiplied by the cumulative RC multiplied by the

rainfall intensity. Based on the approximation that 1 acre inch per hour equals 1 cfs. (less than 1 percent error in calculation).

20. Total time for DCIA only.
21. Calculated average rain intensity for DCIA only.
22. Peak discharge for DCIA only.
23. Design peak discharge = max. (col. 19, col. 22).

Table 11-2 Detailed description of hydraulic calculations performed in Example 11-1.

Column Number:
1. Upstream node number.
2. Downstream node number.
3. Trial slope ID number. This column must be an integer value between 0 and 10 selected by the user (or an optimization algorithm, the ES in this example).
4. This column is a lookup function. Using the ID number is column 3, the lookup function searches the data table given in Table 11-2 and returns the corresponding value of the slope.
5. Trial diameter ID number. Operates exactly like column 3. An integer between 0 and 6 is selected by the user or the optimization algorithm.
6. Trial pipe diameter. Operates exactly like column 4. Using the pipe ID selected in column 5, a lookup function is used to return the pipe diameter in inches corresponding to the ID number in column 5.
7. Diameter check. This column checks whether the pipe selected is equal to or greater than the pipe diameter immediately upstream. The calculations in this column are $D_j - D_{j-1}$ where D_j is the upstream pipe. This constraint is satisfied as long as these differences are non-negative.
8. Length of section, feet.
9. Upstream ground elevation, ft.
10. Downstream ground elevation, ft.
11. Calculates the depth of cover over the crown of the pipe at the upstream node.
12. The downstream depth to the crown of the pipe is calculated as a function of the upstream depth and the slope.
13. The minimum cover is checked by comparing the actual depth of cover against the minimum of 4 feet. Thus, each of these calculated values must be $\geq$ 4 ft.
14. The full pipe flow under current design conditions of slope and diameter based on the Manning equation in U.S. customary units with D in feet, and Q is in cfs.

$$Q = \frac{0.463}{n} D^{8/3} S^{1/2}$$

15. The peak required flowrate is copied from column 23 of the Hydrologic spreadsheet.
16. The capacity check is simply $Q_f - Q_{required} \geq 0$.
17. The ratio of the peak design flow (column 15) and the pipe capacity (column 14).
18. The ratio of the design flow velocity to the full pipe velocity corresponding to the ratio in column 16, based on the approximating polynomial shown in worksheet. The approximating polynomial equation is:

$$v/v_f = \alpha_0 + \alpha_1(q/q_f) + \alpha_2(q/q_f)^2 + \alpha_3(q/q_f)^3 + \alpha_4(q/q_f)^4$$

19. Full pipe velocity. Calculated by the full pipe flow in column 14, divided by the cross-sectional area of the pipe.
20. Design flow velocity. Column 18*column 19. The calculated design velocity is constrained to be between 3 and 10 feet/second.

Table 11-3 Details of the calculations in the cost table of Example 11-1.

Column number:
1. From node.
2. To node.

3. Upper invert elevation is the column 11 from the Hydraulics table plus the diameter of the pipe.
4. Lower invert elevation is the column 12 from the Hydraulics table plus the diameter of the pipe.
5. Pipe cost is the $1.50*D in inches*Pipe length.
6. Average depth is the average of the upstream and downstream depths to the pipe invert.
7. Average width = pipe diameter + 1.5 feet.
8. Total excavation volume = Col. 6*Col. 7*Col. 8 from the Hydraulics Table.
9. Assumed unit cost of excavation in $/yd^3.
10. Total excavation cost, $
11. Manhole depth, ft. = invert depth at upper end.
12. Manhole cost = $482H^{0.9317}$.
13. Total cost = pipe cost (col. 5) + excavation cost (col. 10) + manhole cost (col. 12).

Sewer design optimization
The sewer design optimization problem can be stated as follows:

Minimize the total cost of the sewer system
Subject to the following constraints:
1. Downstream diameter ≥ upstream diameter.
2. Minimum depth of cover ≥ 4 ft.
3. Pipe capacity ≥ peak pipe flow.
4. Pipe velocity is between 3 and 10 ft/sec.
5. Only available pipe diameters can be used, i.e., {10, 12, 15, 18, 21, 24, 30}
6. Pipe slopes can be continuous or discrete variables.

An efficient way to solve this problem is to set it up using the ES option as shown in Figure 11-2. In order to get an approximate answer, select the option in ES that allows the integer constraints to be ignored. The solution to this problem should be a lower bound on the cost. The GRG nonlinear option can also be used for this initial trial. Then, the integer constraints are imposed and the problem is rerun.

For the scenario shown in Figure 11-2, the total cost is $55,369. This solution is technically feasible since all of the constraints are satisfied. The ES is used to vary the assumed slopes and diameters until the least cost solution is found. The user can greatly accelerate the search process by restricting the search space. In this example, the model selects from among seven pipe diameters and eleven slopes. If the integer constraints are ignored, a minimum cost of $52,666 can be attained. Thus, the integer solution is about 6% higher. The integer solution can be improved by revising the assumed pipe slopes and resolving the problem. For example, the ES solution uses slopes of 0.005 to 0.015. The slope table could be revised to focus on this slope range to get a better solution.

Using optimization to find the equation for the IDF curve in the storm sewer design
An equation of the form shown below was used to approximate the IDF information for Boulder, CO that is used in the storm sewer design example (Wenzel 1982):

$$i = \frac{a}{t^b + c}$$ **Equation 11-2**

where i = rainfall intensity (depth/time); t = duration of rainfall (time); a = constant, unique for each rain station and duration; and b, c = constant for each rain station.

The nonlinear solver option in Excel was used to find the parameters that minimize the absolute value of the differences between Equation 11-2 and the values from the Boulder IDF curve (Heaney et al. 1999a and 1999b). These values are given in Table 11-4. As you can see, the parameter estimates for b and c are the same for all recurrence intervals. Furthermore, the value of $b = 0.99$. Thus, a suitable approximation would be to set $b = 1$. A

single general equation can be developed by finding a relationship between the parameter, a, and recurrence interval.

Table 11-4 Intensity/duration/frequency parameters for Boulder, CO. (Heaney et al. 1999a and 1999b).

Recurrence Interval	Rain intensity (in/hr) at time t from Boulder IDF curve			$i = a/(t^b + c)$			Calculated intensity i (in/hr) at time t in minutes		
Years	10 min.	30 min.	60 min.	a	b	c	10 min.	30 min.	60 min.
2	2.9	1.6	0.9	68.3	0.99	13.75	2.9	1.6	0.9
5	4.2	2.3	1.4	98.8	0.99	13.75	4.2	2.3	1.9
10	5.7	3	1.8	134.08	0.99	13.75	5.7	3.1	1.9
25	6.6	3.5	2.1	155.15	0.99	13.75	6.6	3.6	2.2
50	7.3	3.7	2.3	161.84	0.99	13.75	6.9	3.8	2.3
100	8.3	4.2	2.5	192.42	0.99	13.75	8.2	4.5	2.7

A plot of this data and corresponding curve fit, shown in Figure 11-3, indicate that a can be expressed as a logarithmic function of recurrence interval r. Given this information, it follows that a general expression for average intensity for any recurrence interval is an equation of the form:

$$i = \frac{a + b \ln(r)}{(t + c)}$$ **Equation 11-3**
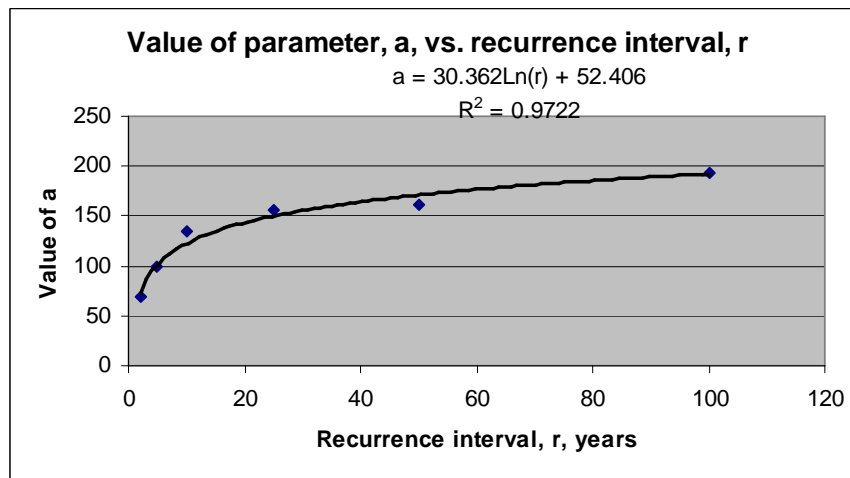
Figure 11-3 Value of parameter, a, as a function of recurrence interval, r.

Using the database in Table 11-4, values for the parameters, a, b, and c can be found that minimize the sum of squares of the errors between the observed and calculated values of i. As shown in Figure 11-4, this problem can be solved as a nonlinear programming problem using the Excel Solver. The final estimating equation for Boulder, Colorado is:

$$i = \frac{48.9155 + 29.6146 \ln(r)}{(t + 12.0989)}$$ **Equation 11-4**

	A	B	C	D	E	F	G	H	I	J	K
1	Example 11.2. Using Solver to find optimal parameter estimates for a storm sewer model.										
2											
3		Recurrence	Rain intensity (in/hr) at time t			$i = a/(t^b+c)$			Calculated intensity i (in/hr)		
4		Interval	from Boulder IDF curve						at time t in minutes		
5		Years	10 min.	30 min.	60 min.	a	b	c	10 min.	30 min.	60 min.
6		2	2.9	1.6	0.9	68.3	0.99	13.75	2.9	1.6	0.9
7		5	4.2	2.3	1.4	98.8	0.99	13.75	4.2	2.3	1.9
8		10	5.7	3	1.8	134.08	0.99	13.75	5.7	3.1	1.9
9		25	6.6	3.5	2.1	155.15	0.99	13.75	6.6	3.6	2.2
10		50	7.3	3.7	2.3	161.84	0.99	13.75	6.9	3.8	2.3
11		100	8.3	4.2	2.5	192.42	0.99	13.75	8.2	4.5	2.7

Row 14: Original model is shown above. Because b = 0.99, assume that
Row 15: it is 1.00.
Row 16: Results of curve fit of a vs. recurrence interval, r, indicate a
Row 17: good fit.

Row 19: Use Solver to find a single general equation for all recurrence
Row 20: intervals by finding optimal parameter estimates for the following
Row 21: equation.

$$i = \frac{a + b\ln(r)}{(t + c)}$$

Value of parameter, a, vs. recurrence interval, r

$a = 30.362 \cdot \ln(r) + 52.406$

$R^2 = 0.9722$

(Value of a vs. Recurrence interval, r, years)

	A	B	C	D	E	F	G	H
26	Case	Meas. i	r	t	Calc. i	e²		
27	1	2.9	2	10	3.14	0.055766		
28	2	4.2	5	10	4.36	0.024288		
29	3	5.7	10	10	5.28	0.177651		
30	4	6.6	25	10	6.50	0.010361		
31	5	7.3	50	10	7.42	0.014611	a* =	48.91552
32	6	8.3	100	10	8.34	0.001896	b* =	29.4164
33	7	1.6	2	30	1.65	0.002139	c* =	12.09889
34	8	2.3	5	30	2.29	0.000182		
35	9	3	10	30	2.77	0.052514		
36	10	3.5	25	30	3.41	0.007904		
37	11	3.7	50	30	3.90	0.038192		
38	12	4.2	100	30	4.38	0.032314		
39	13	0.9	2	60	0.96	0.003752		
40	14	1.4	5	60	1.34	0.004212		
41	15	1.8	10	60	1.62	0.033158		
42	16	2.1	25	60	1.99	0.011717		
43	17	2.3	50	60	2.27	0.000647		
44	18	2.5	100	60	2.56	0.003291		
45					Total	0.474597	<<Minimize sum of squares of errors.	

Row 47: Error² = (measured intensity - calculated intensity)²
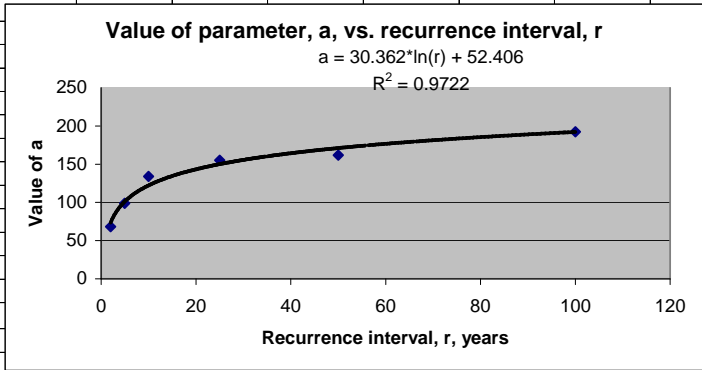
Solver Parameters V5.0

Set Cell: $K$53
Equal To: ○ Max ● Min ○ Value of: 0
By Changing Variable Cells: $H$31:$H$33
Subject to the Constraints:
Standard GRG Nonlinear
Solve | Close | Model | Options | Add | Variables | Change | Reset All | Delete | Help

Figure 11-2 Spreadsheet formulation of parameter estimation problem for IDF curves.

Adding street inlets to the storm sewer design problem

The peak flow in a gutter can be estimated using the following equation (Nicklow 2001):

$$Q = \frac{0.56}{n} S_x^{5/3} S_L^{1/2} T^{8/3}$$

Equation 11-5

where Q = gutter flow rate (cfs); n = Manning's roughness coefficient; S_x = gutter cross slope (ft/ft); S_L = longitudinal slope or grade of roadway (ft/ft); and T = top width of gutter flow on pavement.

It is straightforward to add columns to the Hydrology Spreadsheet to calculate the need for inlets as illustrated in Figure 11-5.

Inlet Data

Cross-slope inverse, S_x, =	0.0417
Assume longitudinal slope, S_L, =	0.010
Manning's n =	0.013
Inlet efficiency, E, =	0.8
Maximum spread on street, T, ft., =	10

1	2	3	4	5	6
Peak Discharge cfs	Longitudinal Slope, S_L ft./ft.	Spread of Water, T ft.	Depth at Curb, d ft.	Peak Q in Street cfs	Peak Q in Sewer cfs
	0.01				
6.76	0.01	8.66	0.361	6.76	0.00
9.54	0.01	9.86	0.411	9.54	0.00
11.39	0.01	10.54	0.439	2.28	9.11
13.75	0.01	11.31	0.471	4.64	9.11

Figure 11-5 Input data and SS calculations for street inlet flow rates.

Summary and Conclusions

Finding the optimal design of storm sewers is a problem of fundamental importance in urban stormwater management. This spreadsheet template illustrates the basic idea of the tradeoff between using larger pipes in order to reduce excavation costs or vice versa. The ES is needed to handle the nonlinearities and discontinuities in the formulation. This method has been tested in engineering classes at the University of Colorado for the past three years and the students have little trouble using it. Heaney et al. (1999a and 1999b) have also used it to optimize the entire Happy Hectares sewer network of over 50 sub-areas.

Chapter 12 WWC Optimization Using SS STORM

Introduction

The purpose of this chapter is to show how to optimize the integrated water quality performance of a WWC using spreadsheet tools presented earlier in this report. This example shows how the optimal size of detention basins can be determined. Methods for evaluating the water quality performance of detention basins as a function of cost have been developed, e.g., Nix et al. (1988). These studies relied on graphical methods or spatial analysis software to determine the optimal storage-release strategy as a function of cost. This process is now much easier computationally with the use of computer optimization methods such as the Excel Solver. The main steps in the analysis are as follows:

1. Download the long-term precipitation data from NCDC, and put it in the proper format for SS analysis using the SS Precip Analyzer described earlier in the report. Also, obtain local estimates of evaporation rates.
2. Using available treatability data, find the parameters of the pollutant removal equation.
3. Develop cost estimating equations for the WWC of interest.
4. Set up the problem using the SS STORM template described earlier in this report.
5. Optimize the WWC design for a specified performance level.
6. Run a sensitivity analysis on the key assumptions.

Each of these steps is described below.

Precipitation and Evaporation Data

One of the inputs for the model is time-series precipitation data. The time increment of the precipitation data is used as the defined time step for the rest of the model. The example in this study is set up using hourly precipitation data and a one hour time step. The model converts the precipitation data into runoff for each time step. The date and time of precipitation events are also used to calculate the number of dry days between events. During the dry periods, water is lost due to evaporation from depression storage and the detention basin surface. At the end of a precipitation event, any water remaining in the detention basin continues to be released until it is completely empty or the next precipitation event starts.

Local precipitation data are available from the NCDC as CDs or may be downloaded directly from their website (http://www.ncdc.noaa.gov). Hourly and 15-minute data sets are available for most areas of the United States (NCDC 2003). The data formats vary for different regions and may need to be converted to run the continuous simulation. This example uses hourly precipitation data from Miami, Florida for the 2001 calendar year.

Another input for the model is monthly evaporation data. Daily and/or monthly evaporation rates based on monitored climatic data are available from several agencies but it differs from region to region (Lee 2003). Depression storage and basin storage are recovered by evaporation during dry periods. The evaporation data used in this example are monthly averages for Miami, Florida (Muller 1983).

Treatability Studies

Important information for evaluating the water quality performance of detention basins is the rate of removal of pollutants. Very few studies have measured particle size distribution and sedimentation rates in stormwater runoff. A study by Randall et al. (1982) provides data from seven urban runoff sedimentation tests which are useful to develop numerous settling rate curves. Pollutant removal in a detention basin is often modeled using plug flow reaction kinetics. A first order plug flow model is typically used because the solution for concentration as a function of time can be easily represented as

$$c = c_o e^{-k\,t_d}$$ **Equation 12-1**

where c is the final concentration (mg/L), c_o is the initial concentration (mg/L), k is the reaction rate constant (hr^{-1}), and t_d is the detention time (hr) of each pulse.

As described in Chapter 7, the Excel Solver can be used to find the optimal parameter estimates. These optimal estimates are used in this example.

Cost Estimating Equation

Total cost is a function of storage volume only. The total cost of constructing a detention basin is estimated using a single variable power function (Sample et al. 2003).

$$Cost = 21{,}950 \times V^{0.69}$$ **Equation 12-2**

Continuous Simulation Model

This study uses a continuous rainfall-runoff spreadsheet model originally developed by Lee (2003). The spreadsheet model is based on the Storage, Treatment, Overflow, Runoff Model (STORM) (Hydrologic Engineering Center 1977). The spreadsheet model converts time-series precipitation data into runoff and routes it through a storage-release device. The model represents volume in terms of depth normalized by the catchment area. The model incorporates depression storage, pond surface evaporation, and pollutant removal using first order kinetics and plug flow.

The flow path of water in the spreadsheet model is presented in Figure 12-1. Precipitation that falls on the catchment initially fills depression storage to capacity and any remaining precipitation volume becomes runoff. Depression storage (or initial abstraction), which must be filled prior to runoff, is caused by interception, surface wetting, ponding, and instant evaporation (Huber and Dickinson 1988). Runoff is diverted to and captured by the detention basin where it is treated and then released at a fixed release rate. Any runoff in excess of the detention basin capacity is bypassed. Both the treated and bypassed water enter receiving water downstream. During dry periods between rain events the pond continues to release water through the outlet structure as well as lose water to evaporation from the pond surface. Evaporation also removes water from depression storage during the dry periods.

(a) during wet period   (b) during dry period
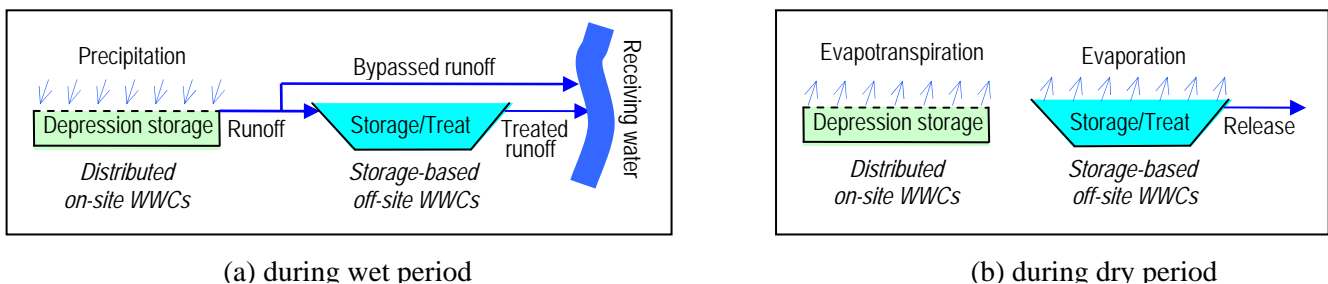
Figure 12-1 Schematic representation of on- and off-site WWCs during wet and dry periods.

Any water captured in the detention basin is treated through sedimentation. Pollutant removal is a function of both volume and concentration. The model uses plug flow to calculate the pollutant load removed. Each pulse of water entering the detention basin remains in the basin for a period of time referred to as detention time. The detention time is dependent on the volume of water already in the basin, the assumed mixing regime, and the constant release rate. The concentration leaving the basin is determined using Equation 12-1. The volume and concentration leaving the detention basin determine the mass of pollutant released downstream. The mass of pollutant captured in the detention basin is the difference between the mass that entered and the mass released.

Flow routing

Flow routing for the model follows a basic mass balance approach. The flow volumes are represented as an equivalent depth normalized by the entire catchment area. The model assumes that the total depression storage and the detention basin storage are initially empty.

The maximum depression storage in the catchment, DS_{max} (mm), is 2.54 mm for the example. This value is based on an average value for depression storage on pavement. The available depression storage, DS_{avail} (mm), is calculated at each time step, j in the simulation using Equation 14-3.

$$DS_{avail}^j = \text{MIN}\{DS_{max}, \text{MAX}(DS_{avail}^{j-1} - Prcp^{j-1}, 0) + Evap^j\} \qquad \textbf{Equation 12-3}$$

This equation returns either the available depression storage or zero if the depression storage is at capacity. Evaporation (*Evap*) empties the depression storage during dry periods. Evaporation is calculated as a function of the evaporation rate (mm/day) for the current month and the number of dry days between precipitation events.

Runoff, Rff^j (mm), for each time step is calculated using Equation 12-4.

$$Rff^j = \text{MAX}(0, Prcp^j - DS_{avail}^j) \qquad \textbf{Equation 12-4}$$

where runoff equals zero only when the depression storage is able to capture all of the precipitation for a particular time step. When depression storage is at capacity, all of the precipitation becomes runoff.

Detention basin storage is calculated at the beginning and end of each time step to determine an average storage. The storage at the beginning of the time step, S_1^j, is equal to the storage in the previous time step storage, S_2^{j-1}, plus any new runoff entering the basin. If there was a dry period since the last time step, then water released (*Rlsd*) from the basin and evaporation losses are subtracted to find the new storage volume.

$$S_1^j = \text{MIN}\{S_{max}, \text{MAX}(0, S_2^{j-1} - Rlsd^j - Evap^j) + Rff^j\} \qquad \textbf{Equation 12-5}$$

Equation 12-5 also takes into account that if the basin capacity is not large enough to contain the previous storage volume and the new runoff, then part of the new runoff bypasses the system and the storage equals the storage capacity.

The volume of water released during a time step, $Rlsd^j$ (mm), is simply equal to the minimum of the constant release or the available water still left in storage. The constant release during a time step is equal to the multiplication of the constant release rate, Q_{rls} (mm/hr), and the time step, Δt (hr).

$$Rlsd^j = \text{MIN}(S_1^j, Q_{rls} \times \Delta t) \qquad \textbf{Equation 12-6}$$

The storage at the end of the time step, S_2^j, is simply the storage at the beginning of the time step, S_1^j, minus the water released during the time step.

$$S_2^j = S_1^j - Rlsd^j \qquad \textbf{Equation 12-7}$$

The volume of water treated in a time step, Trt^j (mm), is how much of the incoming runoff is captured and treated.

$$Trt^j = \text{MIN}\{Rff^j, S_{max} - \text{MAX}(0, S_2^{j-1} - Rlsd^j - Evap^j)\}$$ **Equation 12-8**

If there is no bypass for a time step, then 100% of the runoff in that pulse will be treated. If some of the incoming runoff has to be bypassed because the detention basin is at capacity, then only a fraction of that runoff pulse will be treated.

The volume of water bypassed in a time step, $Bpsd$ (mm), is simply equal to the amount of runoff minus the amount that is captured and treated.

$$Bpsd^j = Rff^j - Trt^j$$ **Equation 12-9**

Pollutant removal
Pollutant removal in a detention pond occurs by various physical, chemical, and biological processes. This study uses suspended solids as a measure of pollutant concentration. Pollutant removal is simulated in the SS model using a first-order plug flow model. The pollutant concentration leaving the detention basin is calculated using Equation 12-1. The values of c_o and k are found using the settleability data as discussed in the reaction rate constant section earlier (see Chapter 7). The detention time (t_d^j) of each runoff pulse is measured as the amount of time it takes for the pulse to travel through the basin. All of the water already in the basin must drain at the constant release rate (Q_{rls}) first, and then the pulse of concern will exit the basin. Only half of the volume of the current pulse is considered to give an average detention time for the entire pulse. This is shown mathematically in Equation 12-10.

$$t_d^j = \frac{S_1^j}{Q_{rls}} + 0.5\frac{Trt^j}{Q_{rls}}$$ **Equation 12-10**

The total mass of suspended solids removed, M_{rmvd} (mg), from the runoff is equal to the difference between the initial and calculated concentrations multiplied by the volume of water treated in the detention basin as shown in Equation 12-11.

$$M_{rmvd}^j = Trt^j(c_o - c^j)$$ **Equation 12-11**

The mass of suspended solids removed is then used to determine the rate at which the detention basin accumulates solids. A maintenance schedule can be developed to remove accumulated solids based on the continuous simulation.

SS Optimization Procedure
The continuous process-simulation model minimizes the total cost of the detention basin by varying the storage volume and release rate while maintaining a performance level of 60% pollutant removal. The model is optimized directly using the non-linear GRG Solver add-in from Excel. The optimization problem can be stated as follows:

Minimize:	Cost $= f(Storage\ Volume)$
Design Variables:	Storage Volume
	Release Rate
Constraints:	Pollutant Load Removal $\geq$ Target Performance Level

Results
Results from the optimization procedure for different values of initial concentration, c_o, and reaction rate constant, k, are presented in Table 12-1 (Rapp et al. 2004). The values of c_o and k are determined using the settleability data as previously described and are shown in rows two and three of Table 12-1. An example is presented for the values of c_o and k from the grouped settleability data (average of all sedimentation tests simultaneously). The values for the

grouped data are $c_o = 181.97$ (mg/L) and $k = 0.6837$ (hr^{-1}), as shown in the far right column of Table 12-1.

Table 12-1 Optimization Results for Multiple Values of c_o and k (Rapp et al. 2004).

Test Number	1	2	3	4	5	6	7	Grouped
c_o (mg/L)	15	35	38	100	155	215	721	181.97
k (/min)	0.0226	0.0555	0.1851	0.1881	0.9761	0.4935	0.9632	0.6837
Storage volume (mm)	48.83	40.55	25.2	25.09	17.83	20.22	17.88	19.08
Release rate (mm/min)	0.065	0.3514	0.6074	0.6102	1.3155	0.8766	1.3095	0.9697
Total cost ($)	$152,090	$133,525	$95,706	$95,411	$75,142	$82,053	$75,275	$78,786
Volume captured (%)	62.3%	70.6%	67.3%	67.2%	64.2%	64.7%	64.2%	63.8%
Mass removed (mg)	13,603	31,741	34,462	90,689	140,568	194,981	653,866	165,025
Mass removed (%)	60%	60%	60%	60%	60%	60%	60%	60%

The spreadsheet optimization model for the grouped data is shown in Figure 12-2. The Excel Solver determines the minimum cost of the detention basin by varying the storage capacity and release rate while maintaining 60% pollutant removal. The results of the grouped values show that a storage capacity of 18.15 (mm) and a release rate of 0.87 (mm/hr) are able to achieve 60% pollutant removal for a total cost of $76,061. This is the least expensive solution able to achieve the target removal level.

The cost of the detention basin is directly dependent on the storage volume of the basin because of the form of the cost function. The storage volume and release rate are decision variables that are changed to achieve the specified performance level. Pollutant removal is achieved by the first order reaction. Therefore, the cost of the basin is indirectly a function of the reaction rate constant, k. The relationship between k and the total cost is shown in Figure 12-3. It can be seen from this curve that the cost is heavily dependent on the choice of k. It is very important to know the site specific settling characteristics so that an accurate value of k can be determined.

The cost of the detention basin is also heavily dependent on the performance level chosen. The cost of the detention basin for the grouped settleability data and various pollutant removal levels is shown in Figure 12-4. It can be seen that the knee of the curve is around 80% pollutant removal. The cost begins to increase rapidly for any improved performance above 80%, e.g., the cost nearly doubles to improve removal from 80 to 90%.

Summary and Conclusions
Stormwater detention ponds are frequently used to control urban runoff. Detention ponds need to be sized in order to reduce peak flow as well as improve water quality. Site specific characteristics are very important in determining the effectiveness of a detention pond. Local rainfall-runoff data is needed to determine the basin volume necessary to provide sufficient peak attenuation. Stormwater characterization data such as particle size distribution or settleability data provide useful information to model pollutant removal. Having this site specific information allows an accurate prediction of the effectiveness of a detention basin. This example demonstrates the importance of how important the reaction rate equation and performance level are when designing a detention basin.

The SS STORM model provides an easy way to optimize continuous simulation results. Unlike our earlier work wherein a pre-specified number of trials were run and then a production function was fit to the output, this model works directly with the simulator to find the best design. It is straightforward to extend this analysis to allow other important parameters such as the rate constant to become decision variables.

	A	B	C	D	E	F	G	H	I	J	K	L	M
1	Minimize Total Cost				Total cost			Cost function = a * (Storage)^b					
2	By changing Storage and Release					Cost		Storage	a	b			
3	Subject to 60% Pollutant Removal				Off-Str	$76,061		Off-site	10000	0.7			
4					Total	$76,061							
5		Evaporation Rate						Volume-based					
6		Month	(mm/d)		Total Depression Storage			Precip	1771.904				
7		1	1.516		DS_{total}	2.54	(mm)	DS	260.42				
8		2	1.786					Runoff	1511.48				
9		3	2.387		Off-site str/rls			Trt	952.94				
10		4	3.433		Storage	18.14701	(mm)	Bpsd	558.54				
11		5	4.387		Release	0.86618	(mm/hr)	Captured	63.0%				
12		6	5.400		Empty	0.872943	(d)						
13		7	5.548					Pollutant mass					
14		8	5.419		Pollutant Removal			M (runoff)	275042.7				
15		9	4.900		(1st order PFR)			M (in)	173405				
16		10	3.742		$C = C_o * \exp(-k * td)$			M (bypass)	101637.7				
17		11	2.533		C_o	181.9689	(mg/L)	M (out)	8379.65				
18		12	1.839		k	0.683715	(hr^{-1})	M (removed)	165025.3			Target	
19								% removed	60.0%	>=		60.0%	
20	Continuous Simulation												
21	Total =	1771.904	340.9584			1511.482			952.9374	558.5443			8379.65
22								Storage-Release-Bypass				Pollutant Removal	
23	Prcp time series		Number of	Evaporation	Depression	Runoff	Beginning	Depth	End	Depth	Depth	Detention	Pollutant
24	Time	Precip.	Dry Days	Rate	Storage	Depth	Storage$_1$	Released	Storage$_2$	Treated	Bypassed	Time, td	Mass Out
25	(yyyy/mm/dd hh:mm)	(mm)	(days)	(mm/day)	(mm)	(mm)	(mm)	(mm)	(mm)	(mm)	(mm)	(hr)	(mg)
26									0				
27	2001/01/04 02:00	0.508	0	1.516129	2.54	0	0	0	0	0	0	0	0
28	2001/01/04 03:00	1.016	0	1.516129	2.032	0	0	0	0	0	0	0	0
29	2001/01/04 04:00	0.762	0	1.516129	1.016	0	0	0	0	0	0	0	0
30	2001/01/16 08:00	0.254	12.125	1.516129	2.54	0	0	0	0	0	0	0	0
31	2001/01/20 03:00	0.508	3.75	1.516129	2.54	0	0	0	0	0	0	0	0
32	2001/01/20 04:00	3.81	0	1.516129	2.032	1.778	1.778	0.86618	0.91182	1.778	0	1.026346	160.3881
33	2001/01/20 13:00	7.62	0.33333	1.516129	0.505371	7.114629	7.114629	0.86618	6.248449	7.114629	0	5.159593	38.027
34	2001/01/20 14:00	0.254	0	1.516129	0	0.254	6.502449	0.86618	5.636269	0.254	0	7.360423	0.30149
35	2001/01/20 15:00	0.254	0	1.516129	0	0.254	5.890269	0.86618	5.02409	0.254	0	6.653665	0.488802
36	2001/01/22 15:00	0.254	1.95833	1.516129	2.54	0	0	0	0	0	0	5.800286	0
37	2001/02/04 18:00	0.508	13.08333	1.785714	2.54	0	0	0	0	0	0	0	0
38	2001/02/04 21:00	0.508	0.08333	1.785714	2.180804	0	0	0	0	0	0	0	0
39	2001/02/19 21:00	0.254	14.95833	1.785714	2.54	0	0	0	0	0	0	0	0
40	2001/03/04 21:00	4.572	12.95833	2.387097	2.54	2.032	2.032	0.86618	1.16582	2.032	0	1.172967	165.8164

Figure 12-2 Spreadsheet WWC optimization model (Rapp et al. 2004).
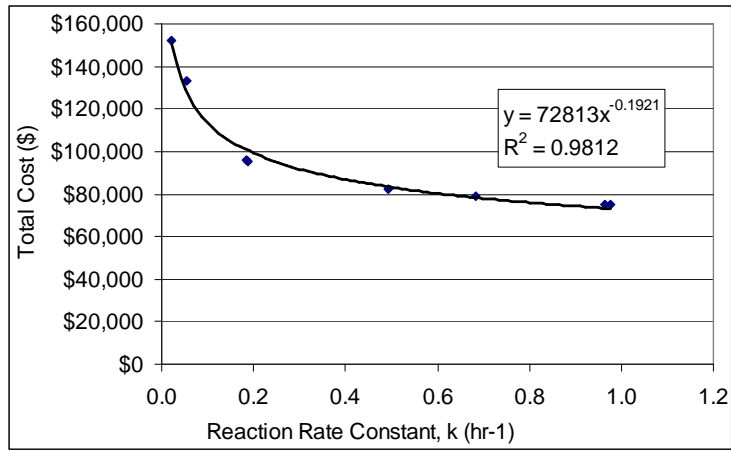
Figure 12-3 Cost as a function of k (Rapp et al. 2004).
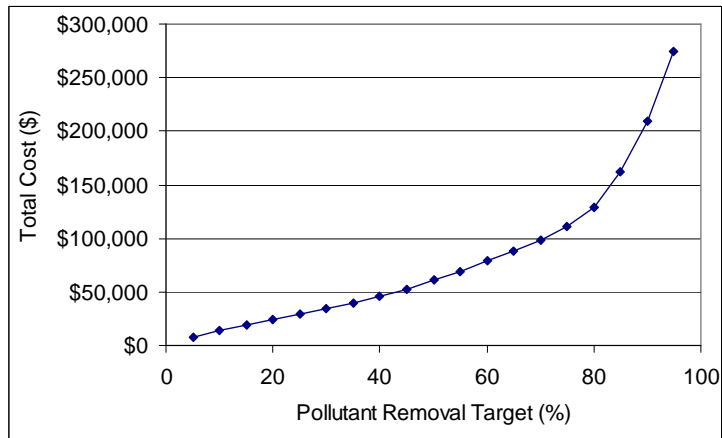
Figure 12-4 Cost vs. pollutant removal (Rapp et al. 2004).

Chapter 13 Optimization of Integrated Urban WWC Strategies

Introduction

A wide variety of urban WWC alternatives have been applied to abate stormwater-induced water quantity and quality problems. In recent years, more distributed approaches or on-site urban WWCs, such as LID, have been promoted as urban stormwater management options in addition to traditional off-site stormwater storage-release systems. Several process simulation models and optimization approaches have been introduced in the previous chapters for analyzing these urban WWCs. In this chapter, optimization approaches for evaluating on-site land use options as well as off-site storage-release systems are described. Continuous stormwater rainfall-runoff and storage-release simulations are embedded in this model using GIS techniques and long-term precipitation data analysis tools. The process modeling is done on a spreadsheet in order to expedite linkage to powerful optimization methods that are available with spreadsheets. The optimization model directs the selection of simulation scenarios in order to find the optimal solution in an efficient manner.

Existing Methods for Optimizing Urban WWCs

Stormwater storage-release systems

Storage-based stormwater management systems, e.g., detention ponds, have widely been implemented to reduce the adverse impacts of urban stormwater quantity and quality problems. The long-term performance of stormwater storage-release system has been analyzed by using empirical (Brune 1953), statistical (Segarra-Garcia and Basha-Rivera 1996; Behera et al. 1999; Adams and Papa 2000), or continuous simulation (Heaney et al. 1978; Nix and Heaney 1988). Economic production function theory (Heaney et al. 1977; Wycoff 2003) has been applied to find the least-cost solution based on estimating the benefits of the outputs and the costs of the inputs. In the application of stormwater storage-release systems, the storage capacity and the release rate can be the inputs, and the runoff volume capture and/or the pollutant removal can be the outputs (Nix and Heaney 1988; Segarra-Garcia and Basha-Rivera 1996; Behera et al. 1999; Adams and Papa 2000). As was shown in Figure 8-1, urban runoff is treated if possible. If the inflow rate exceeds the treatment capacity, then the excess is diverted to storage up to the storage capacity. Water is released from storage when capacity is available in the treatment plant. When both storage/treatment capacities are exceeded, excess inflow is bypassed and discharged directly to receiving water. Performance is measured as percent pollutant mass control over the simulation period.

A typical performance function for pollutant control as a function of detention time is shown in Figure 13-1. In this illustration, about 80% of the pollutant control occurs during the first four hours. Additional removal occurs beyond this knee of the curve but at a greatly reduced rate. Measuring performance in terms of pollutant control complicates the simulation since it is necessary to estimate the detention time for each parcel of water and to assume a mixing regime ranging from plug flow to complete mixing. The operational dilemma is the trade-off between using a low release rate in order to increase detention time and the need to drain the storage device so that it is empty before the next storm event arrives.
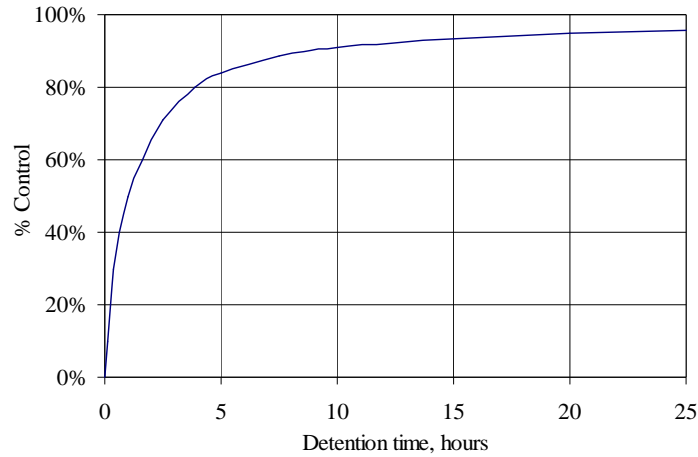
Figure 13-1 Production function for pollutant mass control as a function of detention time.

The production function shown in Figure 13-2 was developed by making simulation runs for assumed values of storage and release rate, and calculating percent pollutant control. This two-variable production function can be graphed as a response surface or contour plot (Myers and Montgomery 2002). It shows the relationship between the system yield, i.e., percent pollutant removal, and the two process variables, i.e., release rate and storage capacity. Early researchers struggled with the limitations of computers and associated run times to get the simulation results. A related problem was how to find approximating equations for the simulation output. A variety of methods ranging from single explicit equations to numerical approximations using splines or kriging have been used to solve this problem (Nix and Heaney 1988).

The simulation results shown in Figure 13-2 indicate that the pollutant control isoquants for the higher percent control, e.g., 85%, actually reach a minimum and then increase as the release rate continues to increase. The economic optimum for this problem depends also on the relative costs of storage and release rate.

The isoquants in Figure 13-2 were drawn using spatial analysis software. This production function can be represented as $y = f(R,S)$. Given the production function and a cost function, $C = g(R,S)$, then the optimal solution can be found by solving the following optimization problem:

$$\text{Minimize} \quad Z = C = g(R,S)$$

$$\text{Subject to} \quad y = f(R,S) \geq y_{\min} \qquad \qquad \textbf{Equation 13-1}$$

$$R,S \geq 0$$

where C = cost; R = release rate; S = storage capacity; and y = percent control.

If analytical expressions are available, then an explicit solution to the optimization problem can be determined as was demonstrated by Heaney et al. (1978). The final expansion path, i.e., locus of minimum cost solutions for any pre-specified level of pollutant control, $C = f(y)$, can then be determined. However, these analytical expressions may be inaccurate. By finding a numerical approximation of the production function, then the minimum cost solution can be found either graphically or by simply calculating the total cost for all of the data points and selecting the optimal solution for this discrete solution space.
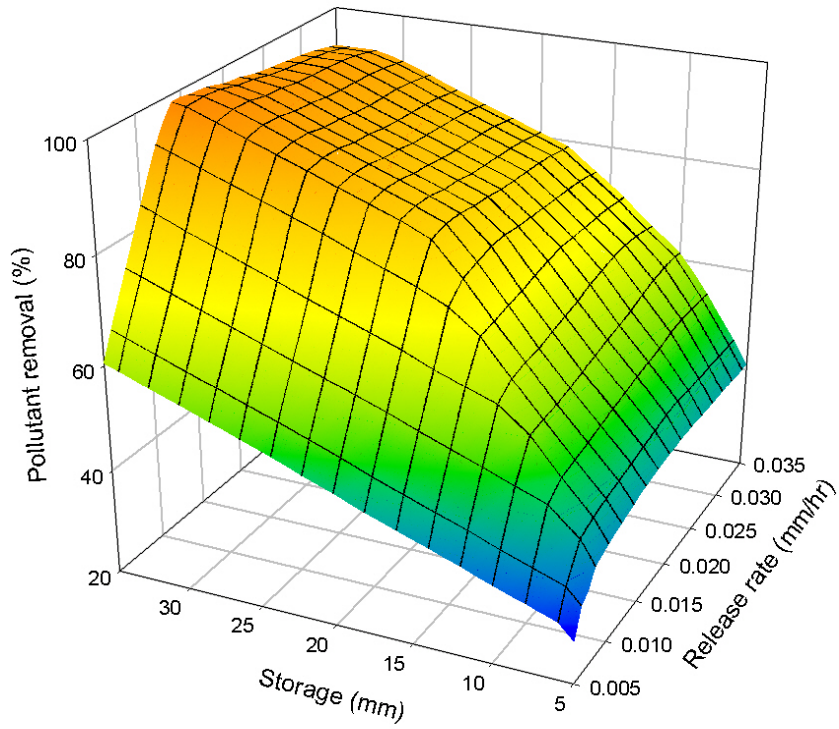
(a) response surface
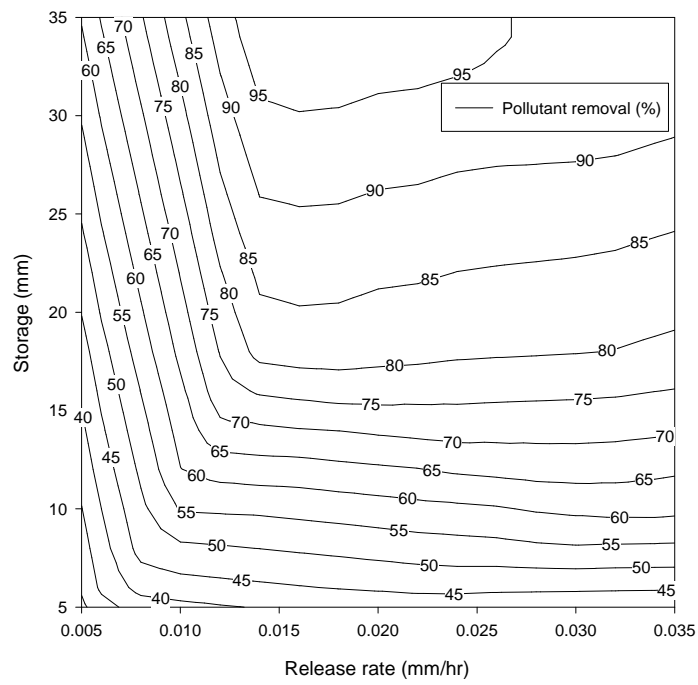
(b) contour plot

Figure 13-2 Production function of pollutant control as a function of storage and release rate.

Nix and Heaney (1988) presented optimization procedures to find a least-cost option for the design and management strategies of stormwater storage-release systems. Performance of a storage-release system was evaluated using a continuous, deterministic stormwater simulation model (Nix and Heaney 1988). The EPA Storm Water Management Model version III (SWMM III) was used to develop the production function that represents the system's performance. They ran the simulation model with many different combinations of storage capacity and release rate, and then attempted to find a certain mathematical form to represent functional storage-release isoquants. After attempting to fit a variety of functions to output from storage-release simulations, they concluded that no single mathematical form could accurately represent the actual storage-release relationship. Due to the inapplicability of using a functional form, they described two alternative methods for developing the production function: a graphical solution procedure and a way to use spatial analysis software. Using the graphical solution procedure, isoquants are manually fit to the data points from the simulation results. Then, the optimal solution is found by manually overlaying the objective function onto the production function surface. The spatial analysis procedure uses the database generated by the process simulator for approximating the response surface and uses software to automate the process of drawing isoquants. The response surface software uses kriging and/or spline fitting techniques to generate a much larger database of response surface values.

Distributed land use optimization model for stormwater management
In recent years, distributed and/or on-site stormwater control efforts have received more attention. While distributed on-site WWCs are widely discussed, little performance analysis and optimization approaches for those types of efforts have been implemented. A linear programming (LP) model to find the combination of cost-effective functional land use options was introduced by Heaney et al. (2000) and Sample et al. (2001). This optimization model minimizes the total cost of providing the required amount of roof, driveway, yard, and patio while satisfying the requirements for on-site depression storage at a residential lot. The model is developed based on a parcel-level detail spatial database and a linear representation of depression storage and a land development cost function for each land use category. This model is set up in a spreadsheet and solved by a spreadsheet-based optimization tool using linear or nonlinear programming. This land use optimization model will be combined with a storage-release model to demonstrate the new approach.

Decoupled optimization procedure
Traditional optimization methods have seen limited use in solving WWC problems due to problem complexities and relatively rigid constraints on the types of processes that may be optimized. Typical pollution control problems exhibit non-smooth, non-convex solution spaces, nonlinear and/or discrete cost functions, and are combinatorially complex because of the number of possible solutions. This has posed significant hurdles for the use of optimization approaches in WWC planning.

State-of-the-art meta-heuristic (MH) optimization techniques represent a substantial departure from traditional methods in how physical processes and optimization are considered. MH techniques such as tabu search (TS) and genetic algorithms (GA) allow for independent process simulation. This decoupled procedure of optimization is fundamentally different from the traditional approach. The optimizer no longer places a constraint on the nature of the functional relationships that are allowed in the process model. For example, if the process simulator includes discontinuous functions, then the meta-heuristic optimization option can be used to solve the problem. MH decoupled optimization has been applied to several WWC problems (Heaney et al. 2002, 1999a; Wright et al. 2001) where the physical processes have been simulated directly in a spreadsheet environment.

A schematic representation of the information flow between optimization and process simulation in the decoupled optimization procedure is described in Figure 2-2. A limitation of using the decoupled approach is that it is necessary to write an interface between the process simulator and the optimizer. Lippai et al. (1999) show how water distribution system optimization can be accomplished by writing an interface between EPANET and a genetic algorithm. For stormwater applications, one could write an interface between EPA SWMM and an optimizer. However, SWMM is a legacy FORTRAN program and it is not easy to write this interface. The new SWMM 5.0 will be easier to link with an optimizer. If the process simulator can be set up on a spreadsheet, then the linkage with the

optimizer is trivial to set up. Fortunately, many of the stormwater processes can be simulated directly on a spreadsheet. This approach was used in this paper as described in the next sections.

Components of Optimization Model
Spatial information and meteorological monitoring data
Functional spatial database
Shamsi (1996; 2002), Heaney et al. (2000) and Sample et al. (2001) summarize GIS applications to urban stormwater problems. Sample et al. (2001) also describe the applicability of a parcel-level detail GIS database for urban stormwater management purposes. Urban wet-weather flow water quantity and quality responses depend strongly upon the catchment's spatial characteristics. Functional spatial information that can represent complicated urban surfaces is critically important for the analysis and optimization of urban stormwater management options, especially distributed WWC alternatives. It is necessary to develop a functional spatial database based on different stormwater response functions for a number of hydrologic functional sub-areas. For example, a residential lot can be divided into several hydrologic functional sub-areas such as rooftop, driveway, patio, and vegetated yard as shown in Figure 13-3.
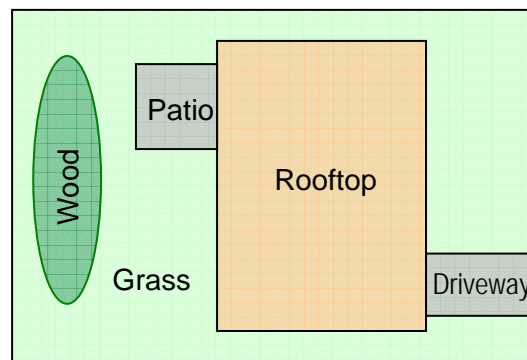
Figure 13-3 An example of functional spatial segments for a residential lot.

Lee (2003) shows how a functional spatial database can be used to analyze wet-weather flow dynamics and control effectiveness. Modeling techniques and reliability of the modeling results can be improved using a functional spatial database. Performance of distributed WWCs and LID scenarios can also be analyzed using a functional spatial database with reasonably high accuracy.

Process simulation models
Distributed land use optimization model
The effectiveness of distributed land use options for urban WWCs can be estimated using on-site depression storage. Depression storage (or initial abstraction), which must be filled prior to the occurrence of runoff, is caused by interception, surface wetting and ponding, and instant evaporation (Huber and Dickinson 1988). Some of the precipitation will be stored in depressions during the wet period, and then the stored water will be depleted by ET during a subsequent dry period. An existing parcel-level land use optimization model based on on-site depression storage (Heaney et al. 2000 Sample et al. 2001) was extended in this study to incorporate the recovery of depression storage capacity using the estimated average monthly ET rate.

Continuous stormwater rainfall-runoff and storage-release simulation model
The process simulation was done using the SS STORM described in Chapter 8. Urban stormwater quality control systems can be evaluated using the storage-release system shown in Figure 13-4. Stormwater flow may be captured and treated by a storage facility or bypassed after its storage capacity is reached.
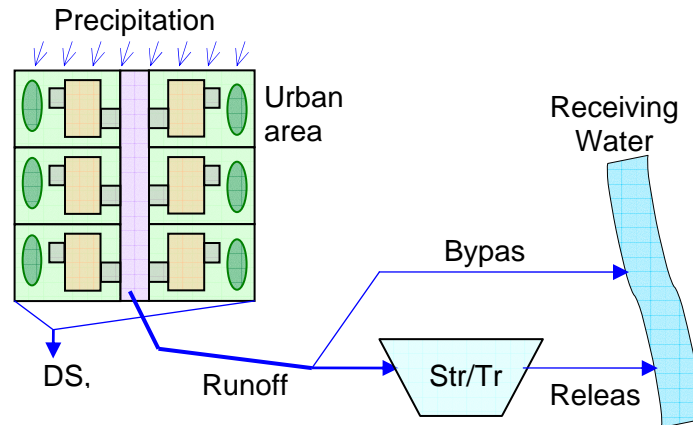
Figure 13-4 Schematic of an on-site control, storage/release system for wet-weather quality control.

Runoff occurs after satisfying available land surface depression storage and deducting infiltration loss. Released surface runoff flows to the storage facility up to its maximum capacity, and then is bypassed to the receiving water. Water in the storage facility is released at a fixed rate. Runoff, storage, release, bypassed stormwater, and capture rate are calculated using Equations 8-4 to 8-12 respectively. Stormwater volume is normalized as the equivalent depth over the entire catchment area. Pollutants will be removed in the storage facility through various physical, chemical and biological processes. Pollutant removal is simulated using a first-order plug-flow model as presented in Equations from 8-13 to 8-15.

Cost functions
Linear cost functions of distributed land use options, which are described by Sample et al. (2001), are adapted in this study. Power functions summarized by Heaney et al. (2002) are applied to estimate the overall cost of storage-release systems. The cost analysis study on urban BMPs, presented by Sample et al. (2003), can be used to select a mathematical representation of cost function for several urban WWC alternatives (See Chapter 9).

Optimization tools
The entire process simulation models were developed using spreadsheet functionalities in this study. A decoupled optimization procedure was designed using spatial and temporal process simulation models and spreadsheet-based optimization tools. An add-in to the basic Excel package called Premium Solver Platform (Frontline Systems 2003), described in Chapter 10 was used for this study. In addition to much more powerful linear and nonlinear programming software, it includes an ES for cases where the problem cannot be solved using classical optimization techniques.

Optimization Model Development
Framework of optimization model
An integrated stormwater management optimization model has been developed using a functional land use simulator and a continuous stormwater process simulation model. The developed optimization model consists of two core system components: simulating functional land development options and modeling continuous rainfall-runoff and storage-release responses. Functional distributed land development options are evaluated using on-site depression storage. Long-term continuous rainfall-runoff and storage-release responses are simulated using one year of hourly precipitation data, on-site depression storages for each land use option, a fixed infiltration rate for pervious area, average monthly ET rates, area-normalized catchment runoff model based on the STORM model (Hydrologic Engineering Center 1977), a fixed stormwater pollutant concentration or an EMC, and a first-order plug-flow (PF) pollutant treatment model. The functional land development evaluation model, based on a detailed spatial database, is able to represent spatial reality for a variety of distributed WWCs in lot level detail. The continuous stormwater

rainfall-runoff and storage-release model simulates rainfall-runoff responses and evaluates overall pollutant removal performance as well as volume-based stormwater capture rate based on a long-term continuous simulation. The framework of the developed spreadsheet optimization model is presented in Figure 13-5.
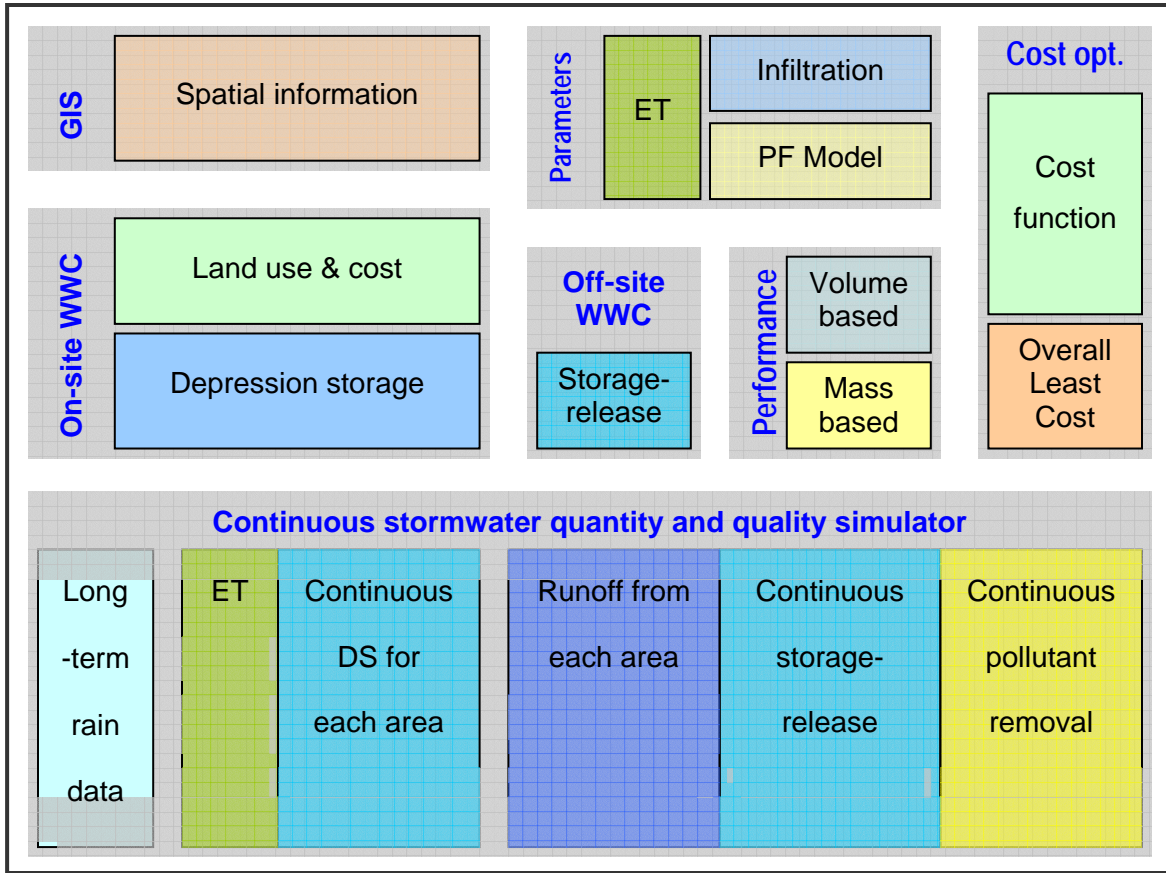
Figure 13-5 Spreadsheet layout of optimization model for integrated urban WWC strategies.

Developed optimization model
This integrated optimization model with a functional land development simulator and a continuous stormwater rainfall-runoff and storage-release simulation model can be summarized as follows:

Minimize $Z = f(Land) + f(Trt)$ (total cost)   **Equation 13-2**

Subject to $\sum_{k=1}^{n} A_i^k = A_i$ (each functional land use)   **Equation 13-3**

$\sum S_i^{onsite} \geq S_{tartget}^{onsite}$ (minimum on-site depression storage)   **Equation 13-4**

$Trt \geq Trt_{t\,\arg et}$ (minimum stormwater volume capture)   **Equation 13-5**

$P \geq P_{t\,\arg et}$ (minimum pollutant treatment)   **Equation 13-6**

where Z = total cost; $f(Land)$ = cost function for land development options; $f(Trt)$ = cost function for storage-treatment systems; A = area of a functional land use segment; S_i^{onsite} = on-site storage; S_{target}^{onsite} = design target of on-site storage; Trt = volume-based treatment rate; $Trt_{t\,\arg et}$ = volume-based target treatment rate; P = pollutant removal rate;

$P_{t\,arg\,et}$ = target pollutant removal rate; k = land use options for each functional sub-areas; and i = functional spatial segments in land use simulation.

To estimate potential pollutant removal performance, the plug-flow model described in Equations 8-13 to 8-15 is applied for each inflow pulse and the overall pollutant removal efficiency is estimated as follows:

$$P = \frac{\sum M_{rv}}{\sum M_{in}}$$

Equation 13-7

$$M_{in} = C_{in}Rff$$

Equation 13-8

where P = mass-based long-term pollutant removal rate; M_{rv} = pollutant mass removal [M]; M_{in} = pollutant mass inflow by runoff [M]; C_{in} = runoff inflow pollutant concentration normalized by catchment area [ML^{-3}·L^2]; and Rff = inflow runoff volume normalized by catchment area [L].

The spreadsheet optimization model is presented in Figure 13-6. It includes spatial representation of the conceptual rainfall-runoff and storage-release system (see the upper left part of Figure 13-6), the entire spreadsheet optimization model, and the spreadsheet optimization tool (see the upper right part of Figure 13-6). The spreadsheet optimization model simulates depression storage and infiltration, runoff from rainfall, an off-site storage-treatment-release system, and system performance based on both runoff volume capture and pollutant removal.

The model shown in Figure 13-6 describes an integrated optimization approach for estimating the overall performance of the off-site stormwater storage-release system and the distributed on-site WWCs implemented in lot-level detail functional sub-areas. Detailed spatial information shown in the upper left part of the spreadsheet in Figure 13-6 was obtained from a functional spatial database. It can represent a single residential lot or an urban catchment. Each functional sub-area within a catchment can be integrated for representing the entire catchment based on their stormwater response characteristics. Thus, a spatial database can be used as is or in an integrated format after aggregating spatial elements based on each functional sub-area.

Using this functional spatial information, the performance of distributed land development options for urban WWCs can be evaluated. In this study, on-site depression storage was used for this evaluation, as shown in the upper left part of Figure 13-6. Monthly average ET rates are used to model the recovery of depression storage. Monitored ET data are presented in the upper center part of Figure 13-6. The long-term continuous stormwater process simulator is integrated with the land use model for simulating rainfall-runoff, storage-release, and pollutant removal phenomena throughout the entire stormwater management system (see the lower part of Figure 13-6).

Fundamental modeling parameters for stormwater quantity and quality simulation, which are infiltration rate (f), long-term precipitation data time increment (Δt), runoff inflow pollutant concentration (C_{in}), and the first-order pollutant reaction constant (k), can be adjusted using appropriate available data (see the upper but slightly right part of Figure 13-6). The performance of the overall stormwater management facilities can be evaluated as an off-site storage system in this model. The long-term continuous storage-release model is shown in the right side of the continuous simulator and storage capacity and release rate is modeled in the upper middle part of Figure 13-6.

Two performance constraints for evaluating off-site stormwater storage facilities are installed in this model: volume capture rate and pollutant removal rate (see the right center part of the spreadsheet in Figure 13-6). On-site small detention storage can also be simulated with a recovery function implemented by evaporation and infiltration (see the center of Figure 13-6). Cost functions for on-site and off-site storage-release systems are added in the model (see the

Figure 13-6 Integrated optimization model for on-site and off-site urban WWCs.

upper right part of Figure 13-6). Based on this cost function for the off-site storage-release system and the on-site distributed land development cost functions (see the upper right part of Figure 13-6), the overall least cost can be

found. The least cost solution is shown in the upper right part of Figure 13-6. This decoupled optimization model in Figure 13-6 uses 15 decision variables and seven design constraints. The model could be solved by the standard NLP Solver but the solution depends on the initial guess. Premium Solver Platform has a multi-start option for NLP optimization that reduces the problem of local optima (Frontline Systems 2003). Alternatively, the ES can be used to find the cost-effective solution in most cases. The applied Solver parameters are shown in the upper right part of Figure 13-6.

Comparison to the existing models
A simple distributed land-use optimization model using on-site depression storage, which was introduced by Heaney et al. (2000) and Sample et al. (2001), is extended to add more meaningful stormwater process dynamics: long-term continuous rainfall-runoff and storage-release dynamic responses. This extended model can be applied not only to analyze distributed land use options for on-site WWCs but also to estimate the overall performance of integrated on-site and off-site urban WWC strategies.

This optimization method does not need to use the three-dimensional production function because process simulation models can be linked directly to the spreadsheet optimization tools without an extra interface. Hence, it is not necessary to perform the time consuming intermediate procedure for developing a graphical representation of system performance and cost relationships. Optimization approaches using the three-dimensional production function are severely limited because only two decision variables and one design constraint can be considered at the same time, i.e., the three axes in a three-dimensional space. In this study, storage volume and release rate were used as two decision variables and pollutant removal performance as one design constraint. However, there are other decision variables and/or design constraints such as different types of storage/treatment and release facilities, the number of outflow water quality violations during a certain period, long-term operation, and maintenance cost. It is impossible to consider all these parameters at the same time using the three-dimensional production function approach. The developed spreadsheet optimization model, however, can directly estimate the performance of a stormwater storage-release system using many decision variables and design constraints.

Graphical representation of storage-treatment performance (Nix and Heaney 1988; Segarra-Garcia and Basha-Rivera 1996; Behera et al. 1999) can also be easily derived using this model. It can be done in a couple of seconds using a two-variable data table creating function in the basic spreadsheet functionalities. The results are presented in Figure 13-7 as a traditional two input variable production function. The isoquants can be derived from the results of long-term continuous process simulation based on a number of combinations of storage-release manipulating strategies. The simulation results are also presented in Figure 13-7 as a number of dots. One dot in Figure 13-7 represents a combination of storage-release strategy. The derived isoquants in Figure 13-7 show typical storage-release relationships. The dots outside of the area of substitution show increasing storage capacity along with increasing release rate to achieve the same level of system performance, which were reported by Flatt and Howard (1978), Goforth et al. (1983), Nix and Heaney (1988), Segarra-Garcia and Basha-Rivera (1996), Behera et al. (1999) and many others. However, this three-variable in two-dimensional representation can only model two decision variables at one time, e.g., storage capacity and release rate. While the new method includes an efficient way to generate the response surface for three dimensional problems, it is no longer necessary to derive the production function separately. The optimizer selects the simulation runs that are most valuable in finding the optimal solution. This sequential search procedure overcomes the limitations of the traditional approaches.

Summary and Conclusions
Methods are needed to jointly evaluate distributed urban WWC alternatives along with traditional downstream storage-based stormwater management systems. An integrated optimization method for overall urban WWC strategies has been described in this chapter. The presented optimization model can be used to determine the most cost-effective strategies at any range of performance for the combination of storage-release systems and distributed on-site WWC alternatives. The model was developed using a decision-support system that includes a functional spatial database, continuous stormwater process simulation models, cost functions for distributed land use options and

a storage-release system, and a spreadsheet optimization tool. The entire optimization procedure – process simulation and optimization – has been implemented in a spreadsheet environment so that it is understandable to the large group of professionals who use spreadsheets routinely.

The developed optimization method does not need to use the three-dimensional production function, which is only good for two decision variables but has been used in most applications for optimizing stormwater storage-release strategies. It is very easy to add more decision variables and design constraints for performing a more sophisticated optimization procedure. This process simulation model can also be used to derive a variety of trade-off relationships among overall stormwater management options. It can be a very useful tool for many areas of stormwater-related decision-makers. Further development will attempt to employ more sophisticated process simulations and enhance the applicability of the optimization model for a variety of generic urban WWC implementations.
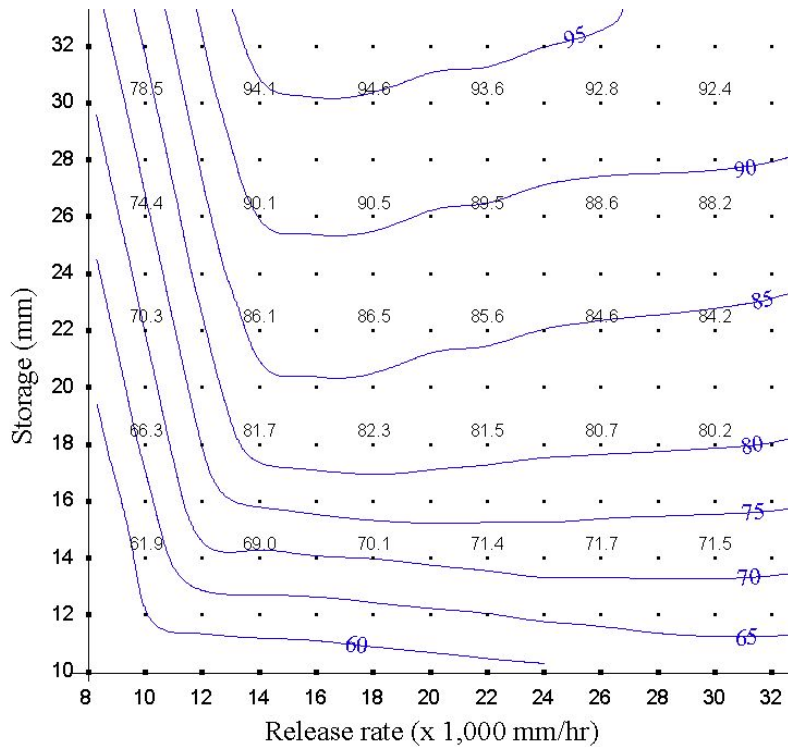
Figure 13-7 Graphical representation of storage-treatment performance.

Chapter 14 Summary and Conclusions

This report is part of a larger project titled "Optimization of Urban Sewer Systems during Wet-weather Periods" (EPA Contract No. 68-C-01-020) to the University of Colorado (CU) with a subcontract to Oregon State University (OSU) dealing with the related issues of simulation and optimization of wet-weather controls. The purpose of this report is to describe optimization methods for finding cost-effective solutions to urban wet-weather flow problems. This report describes a methodology that integrates simulation ("what if" analysis) and optimization ("what's best" analysis) for evaluating which of the myriad of alternative WWCs deserves the title of "best". The optimization analysis integrates process simulation, cost-effectiveness analysis, performance specification, and optimization methods to find this best solution. All analyses are done using a spreadsheet platform.

Following a general review of optimization methods and previous applications to WWC optimization, a series of spreadsheet-based tools are described. An improved method for spatial analysis to get a more accurate representation of land use is described. Then a spreadsheet-based method for analyzing precipitation records to partition them into storm events or to develop intensity-duration-frequency curves is presented. Next, simple methods for estimating infiltration and doing flow routing are described. Influent pollutant loads may be described simply as EMCs or by treatability studies. Next a spreadsheet version of the STORM model for doing continuous simulations is presented. An update on the cost of WWCs is presented next. Then, a primer on optimization methods describes the ease of using these techniques in a spreadsheet environment. Application of these tools to optimize storm sewer design is presented next. Lastly, the optimization of an integrated system that combines land use optimization and a storage-release system is outlined.

The traditional formulation for a constrained optimization problem is to directly embed the process characterization into the optimization model as a constraint. The constraint set, $g(x) \leq, =, or \geq b$, includes the process characterization relationships and performance criteria. If classical mathematical programming techniques are used, then the objective function and constraint set must be well behaved – in the mathematical sense that the relationships are linear, nonlinear but convex, etc. If the problem can be formulated as a coupled system, the optimization methods work quite well and the optimal solution can be found quickly. However, if the objective function and/or the constraint set violate the conditions for solving a classical optimization problem, the more flexible ESs are used. Some expertise is needed to properly match the simulator and the optimizer.

The main findings and contributions of this research project are summarized below:

- The most significant trends of contemporary urbanization are related to automobiles. A case study of detailed spatial analysis shows the critical importance of contributing urban imperviousness, especially directly connected impervious area (DCIA), from transportation-related activities in the right-of-way. Modern tools such as GIS make it much easier to accurately describe the true nature of the imperviousness associated with urbanization.

- Curb and gutter drainage is the major source of DCIA. Alternatives such as swale drainage that minimize the concentration of stormwater can only be accurately evaluated if the nature and importance of DCIA is quantified for proposed developments. Evaluation of decentralized control systems such as LID needs to incorporate explicit recognition of DCIA as a separate source of imperviousness and the role of the right-of-way as a major source.

- Before the availability of GIS and associated databases, spatial analysis typically consisted of highly aggregated characterizations of land uses into major categories such as residential, commercial, and industrial. Also, the study area was divided into relatively few catchments to make the computations easier. This approach was easier to justify when the analysis focused on large, downstream controls. However, much of the current interest is in decentralized controls such as LID. In this case, a large number of smaller WWCs can be used instead of a small number of centralized controls.

- A hierarchy of spreadsheet-based process simulators has been developed as part of this project. These simulators provide flexibility in formulating the optimization problem. Options are presented for land use and spatial analysis, long-term precipitation data analysis, depression storage and infiltration, flow routing, pollutant characteristics, and long-term continuous rainfall-runoff and storage-treatment models.

- Sophisticated state-of-the-art optimization software is available as an Excel add-in. It provides easy to use procedures for doing linear and quadratic programming, GRG nonlinear programming, and evolutionary programming. These optimization methods can be integrated directly with the developed spreadsheet process models described in this report. Thereafter, the entire optimization procedure – process simulation and optimization – can be implemented in a spreadsheet environment so that it is understandable to the large group of professionals who use spreadsheets routinely.

- Long-term precipitation data is one of the most important data sets in urban stormwater analysis. While precipitation data are very critical, it is hard to obtain suitable data sets that have a long period of record and reasonable resolution in time and monitoring depth. Three fundamental precipitation data analysis tools are developed in this project. Developed precipitation analysis model can be an excellent tool for manipulating long-term precipitation records in many WWC applications. The event-based synoptic analysis can be very easily done with a number of IETDs. The disaggregation procedure provides an important tool by generating better resolution of rainfall time series data for a variety of applications, e.g., numerical hydrologic simulation tools such as kinematic wave and diffusion wave simulators. This disaggregation procedure can also be applied to develop site-specific IDF curves with smaller time-step and less than 2-year recurrent frequencies, which are not available at present.

- Site specific characteristics are very important in determining the effectiveness of a WWC system. Local rainfall-runoff data is needed to determine a storage volume necessary to provide sufficient peak attenuation. If site-specific infiltration data are available, infiltration parameters can be calibrated using spreadsheets. Stormwater characterization data such as particle size distribution or settleability data provide useful information to model pollutant removal. Having this site-specific information allows an accurate prediction of the effectiveness of a WWC system. An example of applying site specific reaction rate equation and performance level demonstrated the importance of site specific characteristics when designing a WWC system.

- A wide variety of urban WWC alternatives have been applied to abate stormwater induced water quantity and quality problems. Virtually, every component of urban surface can work as a WWC system: on-site depression storage/infiltration system and off-site storage-release system. Developing a physically sound process simulator is critically important for analyzing WWC alternatives. A continuous simulation model for rainfall-runoff and storage-treatment was developed in this project. This SS STORM model provides an easy way to optimize continuous simulation results. Unlike other earlier works wherein a pre-specified number of

trials were run and then a production function was fit to the output, this model works directly with the simulator to find the best design. It is straightforward to extend this analysis to allow other important parameters such as the rate constant to become decision variables.

- Methods are needed to jointly evaluate distributed urban WWC alternatives along with traditional downstream storage-based stormwater management systems. SS STORM was applied to an integrated optimization method for overall urban WWC strategies. The optimization model can be used to determine the most cost-effective strategies at any range of performance for the combination of storage-release systems and distributed on-site WWC alternatives. Continuous stormwater rainfall-runoff and storage-release simulations are embedded in this model using GIS techniques and long-term precipitation data analysis tools. The optimization model can direct the selection of simulation scenarios in order to find the optimal solution in an efficient manner.

- An integrated approach of applying SS STORM with other WWC analysis components, such as GIS, cost functions, and stormwater flow and pollutant routing techniques, was organized in this project. It does not need to use the three-dimensional production function, which is only good for two decision variables but has been used in most applications for optimizing stormwater storage-release strategies. It is very easy to add more decision variables and design constraints for performing a more sophisticated optimization procedure. This process simulation model can also be used to derive a variety of trade-off relationships among overall stormwater management options. It can be a very useful tool for many areas of stormwater-related decision-makers.

- The developed SS models should be considered as prototypes from this preliminary research project. In order to use them in a general and/or actual application, more effort must be added, such as intense debugging, enhancing user-friendliness, developing appropriate manuals, and conducting an educational program. Further development will attempt to employ more sophisticated process simulations and enhance the applicability of the optimization model for a variety of generic urban WWC implementations.

Chapter 15 Appendix

Functional Spatial Analysis

Prior to the availability of GIS and associated databases, spatial analysis typically consisted of highly aggregated characterizations of land uses into major categories such as residential, commercial, and industrial. Also, the study area was divided into relatively few catchments to make the computations easier. This approach was easier to justify when the analysis focused on large, downstream controls. However, much of the interest is now in decentralized controls such as LID. In this case, a larger number of smaller WWCs can be used instead of a smaller number of centralized controls. With friendly GIS and database systems becoming a reality, a prototype GIS for Happy Hectares was developed to illustrate how these tools can materially improve our ability to optimize WWCs. Lee and Heaney (2003) showed that better spatial resolution can lead to much improved estimates of peak runoff rates.

Major advances in our ability to evaluate urban wet-weather flow systems have come about because high quality GIS and associated database information allow us to evaluate and optimize at various levels of aggregation that would have been infeasible without such tools. A significant effort was made to compile a high quality GIS and associated database for Happy Hectares. This information is summarized below. Accompanying files for ArcExplorer and MS Access are also included.

Description of Happy Hectares
Happy Hectares (HH) is a hypothetical 44.2-hectare study area that is comprised of low to medium density residential land use, apartments, shopping centers, and a school. The University of Colorado research team has used this study area to evaluate water supply, wastewater, and stormwater options. The spatial database was recently expanded to contain more detailed spatial features. This chapter describes the database and several examples that use the database for performing land use analyses.

Happy Hectares, a textbook study area, was adapted from Tchobanoglous (1981). GIS coverage for this case study was developed (Heaney et al. 2000; Sample et al. 2001). The study area was first digitized in AutoCAD, then was edited for geometric consistency, i.e., parallel lines were kept parallel, polygons were joined from separated lines, to make an easier transition to GIS. The existing database is updated parcel by parcel: a building and driveway are newly created for each lot, the public right of way (ROW) is divided by drainage area, and x and y coordinates of most spatial features are added as attribute data. Metric units were used for the entire database. The mix of land uses for the study area is summarized in Table 15-1. The reconstructed GIS coverage of the area is shown in Figure 15-1. The topography of the study area and the layout of the storm sewer system are shown in Figure 15-2. Land use is shown in Figure 15-3 and soils data in Figure 15-4. Based on topography and storm sewer inlets, the entire study area is divided into 81 sub-areas ranging in size from 0.01 to 1.4 hectares.

Figure 15-1 GIS coverage for Happy Hectares.
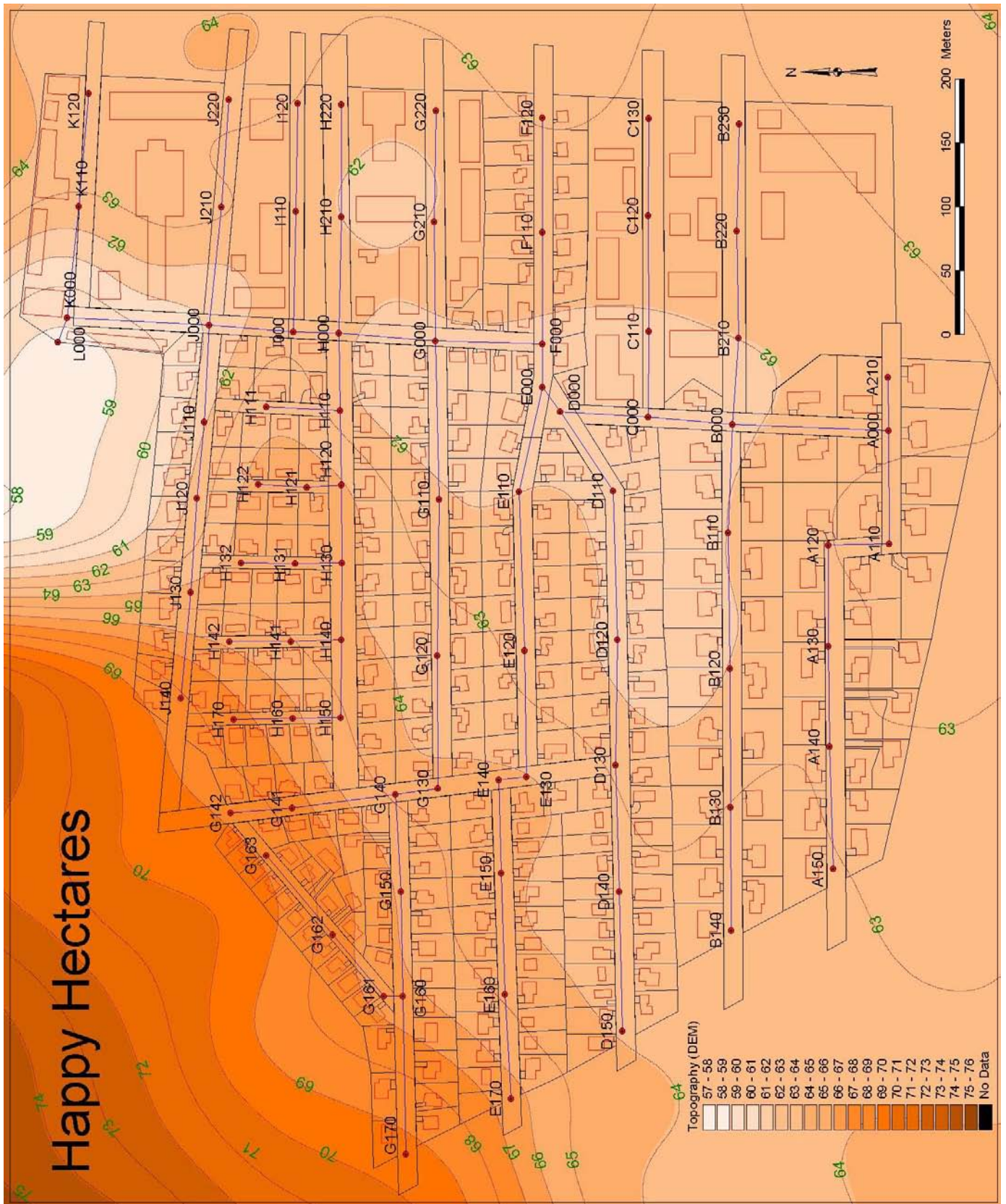
Figure 15-2 Topography and storm sewer system of Happy Hectares.

Figure 15-3 Land use in Happy Hectares.
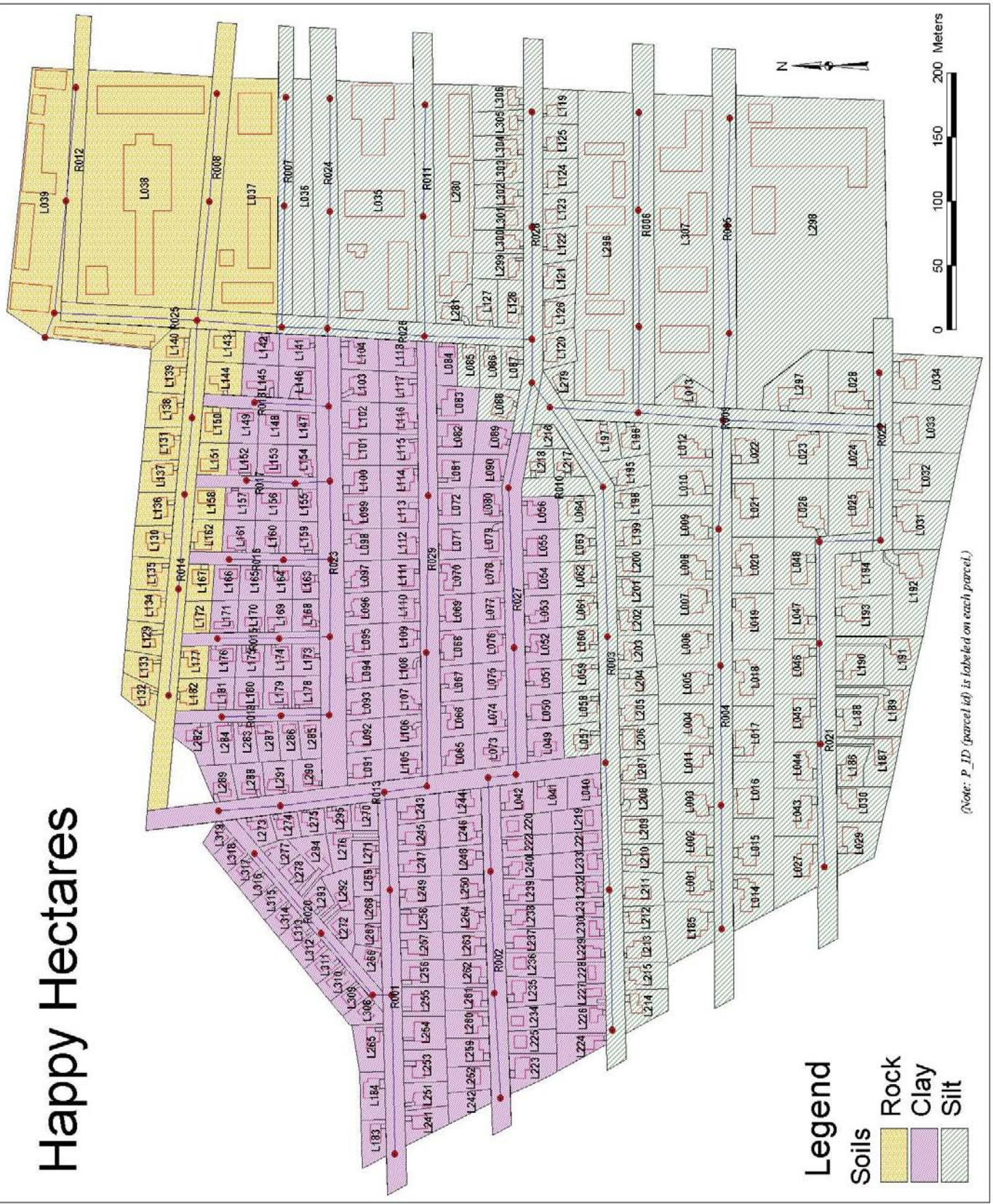
Figure 15-4 Soils in Happy Hectares.

Table 15-1 Mix of land uses in Happy Hectares.

Land use	Area, m^2	Area, ft^2	Area, ac	Units
Apartment	20,743	223,272	5.13	2
Commercial	52,457	564,645	12.96	6
LD Residential	71,294	767,401	17.62	51
MD Residential	168,303	1,811,599	41.59	259
ROW	105,382	1,134,319	26.04	29
School	24,044	258,802	5.94	1
Total	442,222	4,760,038	109.28	348

Spatial database for Happy Hectares
The developed spatial databases are saved as shape files. The spatial data sets are summarized below.

- HHs: Entire Happy Hectares based on sub-drainage areas (433 sub-areas)
- Lots: Data for private lots (330 lots)
- ROW: Data for the public right of way (29 ROW segments)
- Buildings: Building coverage (344 buildings)
- Driveways: Driveway coverage (283 driveways)
- Sewers: Storm sewers (81 sewer pipes)
- Manholes: Storm sewer inlet manholes (82 manholes)
- Contours: One meter topographic contours

These spatial data sets can be used by most of the GIS programs. Free GIS software, named "ArcExplorer 2" can be downloaded from the ESRI website (http://www.esri.com/software/arcexplorer/download2.html). A user's guide for the program is also downloadable from the same website. Basically, it allows the user to display spatial data, build spatial queries, and create maps. The following description is based on ArcExplorer, thus the reader need to download and install the program first.

Main functionalities of ArcExplorer
ArcExplorer is a lightweight GIS data viewer with a straightforward user interface that includes an intuitive menu bar and tool bars. It can easily be used to add themes from existing data sources, control theme characteristics, query spatial properties, print customized maps, zoom in/out, pan, and identify map features. A spatial data set is called as a theme in ArcExplorer. The main functionalities in the ArcExplorer tool bars are summarized in Table 15-2 based on the user guide (ESRI 2002).

Table 15-2 Main tool bars in ArcExplorer.

Button	Name	Description
	New ArcExplorer	Starts a new session of ArcExplorer.
	Open Project	Opens an ArcExplorer project (file with an .aep extension).
	Save Project	Saves an ArcExplorer project.
	Close	Project removes all themes and returns an empty view.
	Add Theme(s)	Adds one or more theme(s) to the view.
	Print	Prints the map view and legend to a preformatted map layout.
	Zoom to Full Extent	Zooms to the extent of all themes.
	Zoom to Active Theme	Zooms to the extent of the active theme.
	Zoom to Previous Extent	Zooms to the last previous extent.
	Zoom In	Zooms in on the position you click or the box you drag on the map view.

	Zoom Out	Zooms out from the position you click or the box you drag on the map view.
	Pan	Pans the map as you drag the mouse across the map view.
	Identify	Lists attributes of features you identify by clicking them in the map view.
	Find	Finds a map feature(s) based on a text string you type in.
	Query	Builder queries the active theme based on a query expression you construct.
	Map Tips	Displays attribute information for features on the map view.
	Measure	Measures distances on the map view. You must first choose measurement units from the detachable menu.
	Theme Properties	Sets the display characteristics of the active theme.

How to use ArcExplorer

ArcExplorer is reasonably straightforward to use. Several examples of the stepwise approaches to display, manipulate and query spatial data sets are summarized below. Also, procedures to create and print maps are summarized.

- Adding spatial data to ArcExplorer
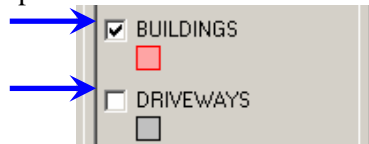 Step 1   Click the Add Theme button to open the Add Themes dialog.
  Step 2   Click on each directory to navigate to the directory where the spatial data are stored.
  Step 3   Click the file.
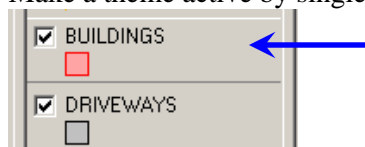 Step 4   Click Add Theme.

- Displaying a theme
  Step 1   Click the check box to the left of each theme's name to make the theme draw in the map view.

  Step 2   Clicking again will turn the theme off.

- Activating a theme (Many operations work only on active themes. When a theme is active, it appears raised in the legend.)
  Step 1   Make a theme active by single clicking on its name in the legend.

- Identifying features with the mouse
  Step 1   In the legend, click the name of the theme you wish to identify to make it active.
  Step 2   Click the Identify tool button .
  Step 3   Click the feature you wish to identify.

- Creating Map Tips (Map Tip is a small popup that displays data for a specified field. Map Tips work on the active theme as you move the cursor over the features on the map view.)
  Step 1   Make the theme you want to display Map Tips for active.
  Step 2   Click the Map Tips tool button to display the Map Tips dialog.
  Step 3   Choose the field to be displayed in the Map Tips and click OK.

Step 4 To disable Map Tips, click the Clear button on the Map Tip Field Selection dialog.

- Using the measurement tool
  Step 1 Specify a measurement unit from the detachable menu under the Measure tool button 📏▼.
  Step 2 On the map view, click and drag to draw a line representing the distance you wish to measure. The segment and total length you measured is displayed in the status panel at the top left of the map view.
  Step 3 To stop measuring and clear the measurements, double-click in the map view. After you double-click, the total length appears in the lower left corner on the status bar.

- Adjusting the Theme Properties
  Step 1 Make a theme active and click the Theme Properties button 🗔 or double-click a theme's name in the legend. The Theme Properties dialog is displayed.
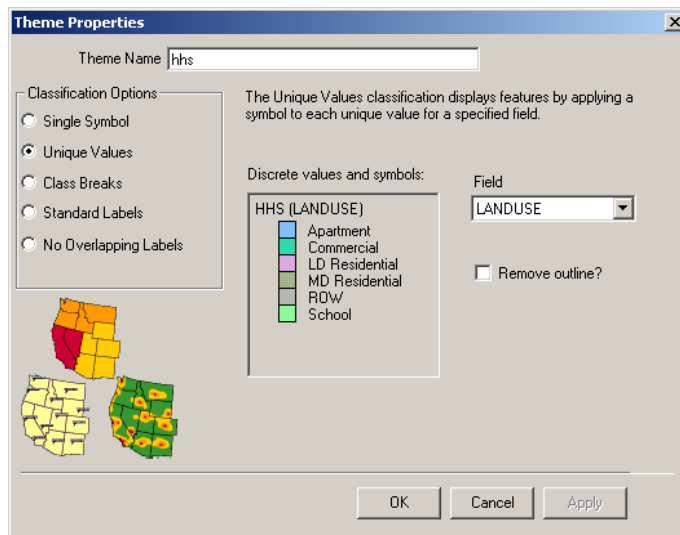
Step 2 To create a Single Symbol map, choose the Single Symbol under the Classification Options.
Step 3 To creating a Unique Values map, choose the Unique Values under the Classification Options.
Step 3a Choose a Field.
Step 3b ArcExplorer automatically assigns random colors to each unique classification.
Step 4 To change a color, click a color box.
Step 5 Click Apply to commit your changes or OK to commit the changes and close the Theme Properties dialog at the same time.

- Working with Query Builder
  Step 1 Click the name of the theme you wish to query to make it active.
  Step 2 Click the Query tool button 🔍.
  Step 3 From the list of fields, choose a field for querying.
  Step 4 Enter a condition to build a query. The Query Builder dialog lets you build the query expression by either clicking on fields, operators, and values or by typing it in.
  Step 5 Click the Execute button. Features that meet the query definition appear in the Query Results panel.
  Step 6 Highlight or zoom the features using the buttons under the window.
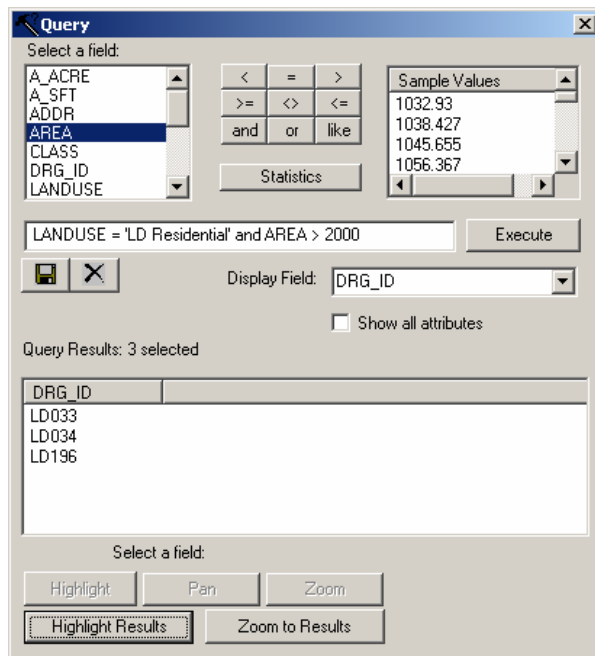
- Generating summary statistics (once you've selected a set of features with the Query Builder, you can also generate summary statistics on a certain field in the database for just those selected records. Statistics are generated for selected features or the full database.)

   Step 1 In the Query Builder, press the Statistics button.
   Step 2 Choose the field for which you want to generate statistics from the list.
   Step 3 Click OK. The summary statistics appear in the Query Results panel.
   Step 4 To save summary statistics to an ASCII file, click the Save Results button above the Query Results panel to create a new file. Name the new file and click Save. The file can be opened in a spreadsheet or word processing program to create a report or conduct further analysis.

- Printing a map

   Step 1 Click the Print tool button .
   Step 2 Enter a title for your map.
   Step 3 Click Print.

- Copying map views for use in other applications (It may need to copy the image of a map view for use in another Windows application, like a word processor or a drawing program.)
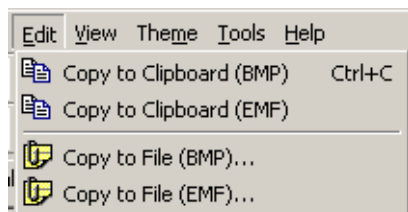
   Step 1 From the Edit menu, or right-click in the map view, click the Copy to Clipboard and Copy to File commands.
   Step 2a Choose Copy to Clipboard if you plan to go into another Windows program and paste the image in directly.
   Step 2b Choose Copy to File if you want to create a file to use anytime in the future. If you do this, ArcExplorer prompts you to name the new file.

Step 3a Choose the BMP (Windows bit map) option for raster data, images.
Step 3b Choose the EMF (Enhanced Metafile) option if the map has only vector data, lines, areas, or points.

- Saving the work in ArcExplorer

  Step 1 Choose Save Project or Save As from the File menu, or click the Save tool button 🖫.
  Step 2 If necessary, name your ArcExplorer project.
  Step 3 Click OK.

Using ArcExplorer, the developed spatial database can be accessed directly using the saved project file (HHs.aep).

Relational database for Happy Hectares

A relational database was developed for representing the attribute information for Happy Hectares based on the spatial database. Some large lots, such as commercial and school, and larger right of way are sub-divided based on drainage inlets. Thus, some parcels are divided by a couple of sub-drainage areas with a fraction of the original size. A schematic description of the relational database for Happy Hectares (HHs.mdb) is shown in Figure 15-5. Data tables (Dat_Xxxx) are linked by two identification keys, drainage area ID (Drg_ID) and manhole node name (Node). Hydraulic attributes of private lots and design properties of the right of way are managed separately as user defined tables (usd_Xxx).
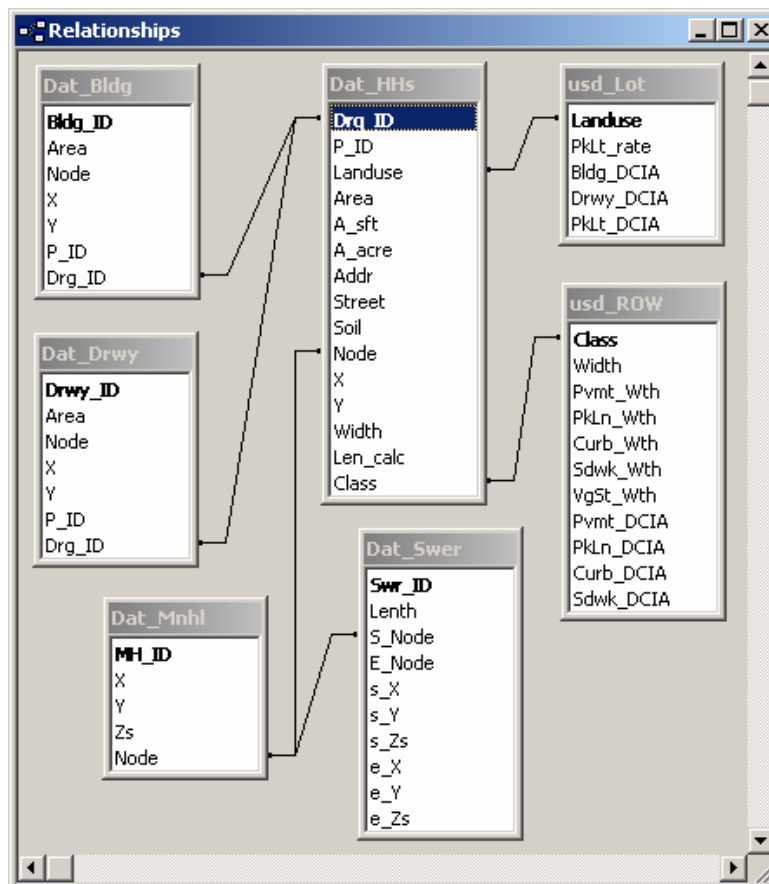

Figure 15-5 Schematic structure of relational database for Happy Hectares.

General descriptions of each data table are summarized below.

- Main Data Tables (Dat_Xxxx)
  Dat_HHs: General descriptive information of the entire parcels
  Dat_Bldg: Description of building rooftops
  Dat_Drwy: Description of driveways
  Dat_Mnhl: Description of manholes
  Dat_Swer: Description of storm sewer pipes

- User Defined Data Tables (usd_Xxx)
  usd_Lot: Attributes of private lots
  usd_ROW: Design properties of the right of way=

Land cover in private land – Lots
The private land is divided into individually owned lots. The total area of private lots is 33.7 hectares and consists of 7.0 hectares of building rooftops, 4.7 hectares of parking lots, 0.6 hectares of driveways (i.e. 12.3 hectares of impervious area) and 21.3 hectares of pervious area. The description of private land is summarized in Table 15-3. Percentage of parking lots and hydraulic connectivity of each impervious area are assumed based on land use categories. These assumptions are shown in Table 15-4. In the relational database, the assumptions are saved as a separate user defined table named "usd_Lot" and can be varied in different scenarios. Whenever the assumptions in "usd_Lot" are changed, every related query result will be updated automatically.

Table 15-3 Land cover in the 319 private lots (m^2).

Land Use	Area	Building	Driveway	Parking Lot	TIA	DCIA	Pervious Area
Apartment	20,743	6,471	0	9,990	16,461	11,931	4,281
Commercial	52,457	16,270	0	32,568	48,839	48,839	3,619
LD Residential	71,294	12,712	2,119	0	14,830	2,330	56,463
MD Residential	168,303	30,681	4,153	0	34,834	5,777	133,469
School	24,044	3,839	0	4,041	7,880	4,809	16,164
Total	336,840	69,973	6,272	46,599	122,844	73,686	213,996

Table 15-4 Percents of parking lots and hydraulic connectivity for private lots.

Land Use	PkLt_rate	Bldg_DCIA	Drwy_DCIA	PkLt_DCIA
Apartment	70%	30%	0%	100%
Commercial	90%	100%	0%	100%
LD Residential	0%	5%	80%	100%
MD Residential	0%	8%	80%	100%
School	20%	20%	0%	100%

Land cover in public land – right of way (ROW)
General design characteristics of the right of way are shown in Table 15-5. These characteristics are saved as a separate user defined table named "usd_ROW" in the relational database. These properties can be changed in different development scenarios. Based on the properties, land cover of the right of way is analyzed. Half of the sidewalk is assumed to be DCIA and the other impervious areas are considered directly connected to the drainage system. Whenever the assumptions in "usd_ROW" are changed, every related query result will be updated automatically.

Table 15-5 Right of way design widths and lengths in Happy Hectares (m).

Width	Pavement	Parking Lane	Curb	Sidewalk	Vegetated Strip	Length
10	5.4	0	0.9	1.8	1.9	684.3
11.6	5.5	0	1.2	2.5	2.4	447.8
15	5.8	2.4	1.2	2.5	3.1	4,825.3
16.6	6.8	3	1.2	2.5	3.1	279.4
18.3	6.8	5	1.2	2.5	2.8	278.4
20	7	5.2	1.2	2.5	4.1	574.9

The right of way is 10 to 20 meters wide and a total of 7,090 meters long. The narrower right of way represents the typical neighborhood street, sidewalk, and vegetation strip. The right of way comprises 10.5 hectares and consists of pavement for traffic lanes (4.2 hectares), parking area (1.7 hectares), curb and gutter (0.8 hectares), sidewalks (1.7 hectares), and landscaping vegetation strip (2.1 hectares) based on the design properties in Table 15-5. The summary of land cover in the right of way is shown in Table 15-6.

Table 15-6 Land cover in the right of way (m^2).

Width	Area	Pavement	Parking Lane	Curb	Sidewalk	Vegetation Strip	TIA	DCIA	Pervious Area
10	6,843	3,695	0	616	1,232	1,300	5,543	4,927	1,300
11.6	5,194	2,463	0	537	1,120	1,075	4,120	3,560	1,075
14.9	34,127	13,284	5,497	2,748	5,726	6,871	27,255	24,392	6,871
15	38,023	14,702	6,084	3,042	6,337	7,858	30,165	26,997	7,858
16.6	4,638	1,900	838	335	698	866	3,771	3,422	866
18.3	5,094	1,893	1,392	334	696	779	4,315	3,967	779
19.9	6,982	2,456	1,824	421	877	1,403	5,578	5,140	1,403
20	4,481	1,568	1,165	269	560	919	3,562	3,282	919
Total	105,382	41,961	16,800	8,303	17,246	21,072	84,310	75,687	21,072

Total land use and land cover

The total land use in Happy Hectares consists of 33.7 hectares in private lots and 10.5 hectares in public right of way, or a total area of 44.2 hectares. The area is covered by 20.7 hectares of TIA, and 72% of this area is DCIA. A general description of land cover of the Happy Hectares is shown in Table 15-7 based on land use categories.

Table 15-7 Land cover in Happy Hectares (m^2).

Land use	Area	Building	Driveway	Parking	Other Imp	TIA	DCIA	Perv. Area
Apartment	20,743	6,471	0	9,990	0	16,461	11,931	4,281
Commercial	52,457	16,270	0	32,568	0	48,839	48,839	3,619
LD Residential	71,294	12,712	2,119	0	0	14,830	2,330	56,463
MD Residential	168,303	30,681	4,153	0	0	34,834	5,777	133,469
ROW	105,382	0	0	16,800	67,510	84,310	75,687	21,072
School	24,044	3,839	0	4,041	0	7,880	4,809	16,164
Total	442,222	69,973	6,272	63,400	67,510	207,154	149,373	235,068

Database query to obtain information tables

Structured Query Language (SQL) in MS-Access Query is used to create information tables. SQL is an ANSI (American National Standards Institute) standard language for accessing databases and the results of a query are

returned in a tabular form. The use of SQL to retrieve the database is shown below.

    SELECT required information (columns)
    FROM   data table(s)
    WHERE   selection criteria
    GROUP BY   grouping criteria

Most of the required basic queries are already built into the relational database. The query results can be used as is or simply copied from a query table to another application program, such as MS-Excel. Two flat data tables, based on drainage areas (fdt_HHs) and parcels (fdt_Parcels), can be applied for any other analyses as base information. Two of subset data tables for private lots (sdt_Lots) and right of way (sdt_ROW) can also be used for any purpose. Built-in queries are shown in Figure 15-6 and summarized below.

- fdt_HHs: General descriptive information for the entire area based on sub-drainage areas
- fdt_Parcels: General descriptive information for the entire area based on parcels
- sdt_Lots: General descriptive information for private lots based on sub-drainage areas
- sdt_ROW: General descriptive information for the right of way area based on sub-drainage areas
- qry_Bldg: Aggregated information of buildings for every lot
- tbl_Landuse: Basic summary of land use for the entire area (Table 15-1)
- tbl_Landuse_Lots: Summary of land use for private lots (Table 15-3)
- tbl_ROW: Design characteristics of the right of way (Table 15-5)
- tbl_Landuse_ROW: Summary of land use for the right of way (Table 15-6)
- tbl_Landcover: Summary of land use for the entire area (Table 15-7)
- SubC_###: Land use analysis for drainage study (Table 15-8)

How to use built-in queries
Using built-in queries is straightforward. The query result automatically pops up as a tabular form whenever any of the query icons in Figure 15-6 is double clicked. An example of the stepwise approach is summarized below.

- To obtain "Land cover in Happy Hectares" (Table 15-7) and copy it to an MS-Excel spreadsheet
    Step 1   Open "HHs.mdb" in MS-Access
    Step 2   Activate or click "Queries" under "Objects" in the left side of the "HHs: Database" window like Figure 15-6
    Step 3   Double click "tbl_Landcover" query icon
    Step 4   Press Ctrl-A to select all data
    Step 5   Press Ctrl-C to copy them to clipboard
    Step 6   Activate or open a worksheet in MS-Excel
    Step 7   Locate a destination cell in the worksheet
    Step 8   Select "Paste Special" under "Edit" menu or after clicking the right button of your mouse
    Step 9   Choose one of "Unicode Text", "Text", or "Csv" for "Paste As:", and then click the "OK" button in the right side of that window
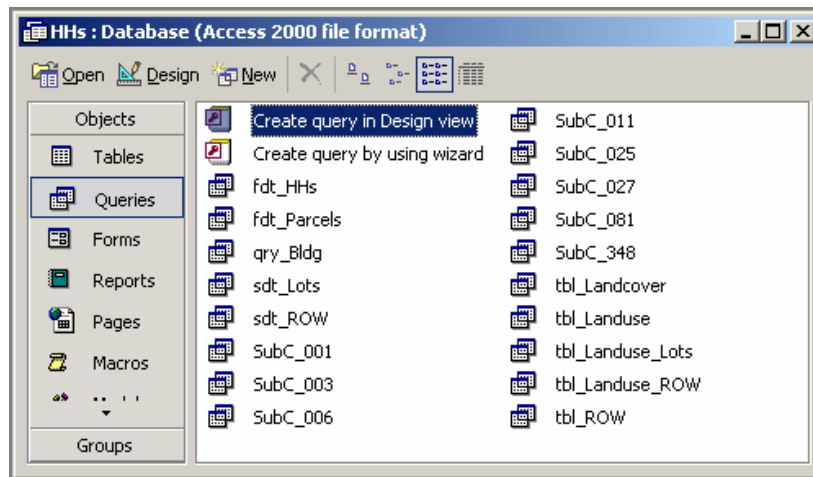
Figure 15-6 Built-in queries in the rational database.

All of the other built-in queries (and also data tables) can be used through the same procedure. Once any required data sets are copied to the spreadsheet, a variety of mathematical functionalities in the spreadsheet can be applied for further numerical manipulation. For instance, the "Total" of every table shown in this report is calculated in the spreadsheet after doing "Copy" and "Paste Special" of required data sets from MS-Access to MS-Excel.

Further analyses or data queries can be done by building queries using SQL in the relational database or manipulating the flat data table (like fdt_HHs) in the spreadsheet.

Subcatchment delineation for stormwater drainage modeling

For stormwater drainage analysis, the entire Happy Hectares can be modeled as one subcatchment, 11 subcatchments based on the main sewer trunks, 81 subcatchments based on all drainage inlets, and so on. Some of the subcatchment delineations for stormwater drainage study are already built in the relational database as queries (See Figure 15-6). These built-in queries can be used by the same procedure shown in previous section. Several queries for subcatchment delineation are listed in Table 15-8.

Table 15-8 Built-in queries for land subcatchment delineation.

Query	Subcatchments	Description
SubC_001	1	the entire area
SubC_003	3	based on soil properties
SubC_006	6	based on land use categories
SubC_011	11	based on the main sewer trunks
SubC_025	25	based on streets name
SubC_027	27	based on sewer branches
SubC_081	81	based on all storm inlets
SubC_348	348	based on all parcels

The query result brings total area (Area), TIA, DCIA, and PA for every subcatchment of the entire Happy Hectares. One example is shown in Table 15-9 from "SubC_011".

Some of the built-in land use analyses may not be useful in real applications and some of the additional analyses may be required for a certain purpose. Queries for different patterns of land use analysis can be added or deleted easily if required.

Table 15-9 Land use analysis based on the main sewer trunks (m^2).

Node	Area	TIA	DCIA	Perv. A
A000	45,012	14,144	7,047	30,868
B000	70,968	25,676	15,645	45,293
C000	25,748	19,244	14,657	6,505
D000	44,657	16,844	10,240	27,812
E000	39,759	13,484	7,748	26,274
F000	16,319	5,344	3,129	10,975
G000	71,092	30,472	20,281	40,620
H000	59,685	30,736	24,018	28,948
I000	6,713	5,226	4,775	1,486
J000	32,400	18,707	14,967	13,693
K000	29,871	27,277	26,866	2,594

Metafile for the database

Main Data Tables (Dat_Xxxx) in HHs.mdb

- Dat_HHs   *General descriptive information of the entire parcels*
    - Drg_ID   *ID for a sub-drainage area*
    - P_ID   *ID for a parcel*
    - Landuse   *Land use category*
    - Area   *Area in square meter*
    - A_sft   *Area in square feet*
    - A_acre   *Area in acre*
    - Addr   *Street number in address*
    - Street   *Street name in address*
    - Soil   *Soil category*
    - Node   *Manhole node name for drainage*
    - X   *X coordinate in meters*
    - Y   *Y coordinate in meters*
    - Width   *Width of the right of way in meters*
    - Len_calc   *Length of the right of way in meters*
    - Class   *ROW class based on its width*

- Dat_Bldg   *Description of building rooftops*
    - Bldg_ID   *ID for a building*
    - Area   *Area of a building in square meters*
    - Node   *Manhole node name for drainage*
    - X   *X coordinate of a building in meters*
    - Y   *Y coordinate of a building in meters*
    - P_ID   *ID for a parcel*
    - Drg_ID   *ID for a sub-drainage area*

- Dat_Drwy   *Description of driveways*
    - Drwy_ID   *ID for a driveway*
    - Area   *Area of a driveway in square meters*
    - Node   *Manhole node name for drainage*
    - X   *X coordinate of a driveway in meters*
    - Y   *Y coordinate of a driveway in meters*
    - P_ID   *ID for a parcel*

Drg_ID   *ID for a sub-drainage area*

- Dat_Mnhl   *Description of manholes*
  MH_ID   *ID for a manhole*
  X   *X coordinate of a manhole in meters*
  Y   *Y coordinate of a manhole in meters*
  Zs   *Z coordinate of a manhole in meters (surface)*
  Node   *Manhole node name*

- Dat_Swer   *Description of storm sewer pipes*
  Swr_ID   *ID for a storm sewer pipe*
  Length   *Length of a storm sewer pipe in meters*
  S_Node   *Starting manhole node name*
  E_Node   *Ending manhole node name*
  s_X   *X coordinate of starting node in meters*
  s_Y   *Y coordinate of starting node in meters*
  s_Zs   *Z coordinate of starting node in meters (surface)*
  e_X   *X coordinate of ending node in meters*
  e_Y   *Y coordinate of ending node in meters*
  e_Zs   *Z coordinate of ending node in meters (surface)*

User Defined Data Tables (usd_Xxx) in HHs.mdb
- usd_Lot   Assumption of private lots
  Landuse   *Land use category*
  PkLt_rate *Rate of parking lot area except building and driveway*
  Bldg_DCIA *DCIA rate for buildings*
  Drwy_DCIA *DCIA rate for driveways*
  PkLt_DCIA *DCIA rate for parking lots*

- usd_ROW   *Design properties of the right of way*
  Class   *ROW class based on its width*
  Width   *Width of the right of way in meters*
  Pvmt_Wth   *Width of traffic lane pavement in meters*
  PkLn_Wth   *Width of parking lane in meters*
  Curb_Wth   *Width of curb and gutter in meters*
  Sdwk_Wth   *Width of sidewalk in meters*
  VgSt_Wth   *Width of landscaping vegetation strip in meters*
  Pvmt_DCIA *DCIA rate for traffic lane pavement area*
  PkLn_DCIA *DCIA rate for parking lane area*
  Curb_DCIA *DCIA rate for curb and gutter area*
  Sdwk_DCIA *DCIA rate for sidewalk area*

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