Optimization Model for the Design of Bioretention Basins with Dry Wells

by

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ABSTRACT

Bioretention basins are a common stormwater best management practice (BMP) used to mitigate the hydrologic consequences of urbanization. Dry wells, also known as vadose-zone wells, have been used extensively in bioretention basins in Maricopa County, Arizona to decrease total drain time and recharge groundwater. A mixed integer nonlinear programming (MINLP) model has been developed for the minimum cost design of bioretention basins with dry wells.

The model developed simultaneously determines the peak stormwater inflow from watershed parameters and optimizes the size of the basin and the number and depth of dry wells based on infiltration, evapotranspiration (ET), and dry well characteristics and cost inputs. The modified rational method is used for the design storm hydrograph, and the Green-Ampt method is used for infiltration. ET rates are calculated using the Penman Monteith method or the Hargreaves-Samani method. The dry well flow rate is determined using an equation developed for reverse auger-hole flow.

The first phase of development of the model is to expand a nonlinear programming (NLP) for the optimal design of infiltration basins for use with bioretention basins. Next a single dry well is added to the NLP bioretention basin optimization model. Finally the number of dry wells in the basin is modeled as an integer variable creating a MINLP problem. The NLP models and MINLP model are solved using the General Algebraic Modeling System (GAMS). Two example applications demonstrate the efficiency and practicality of the model.

ACKNOWLEDGMENTS

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1. INTRODUCTION

1.1 Introduction to Bioretention and Dry Wells

Stormwater management has undergone a dramatic transformation throughout the world in recent decades, as the hydrologic effects of urbanization have become better understood. Urban development is associated with an increase in the volume of stormwater runoff, increased peak discharges, flashier hydrographs, decreased infiltration and evapotranspiration, and water quality degradation.

To mitigate the adverse hydrological effects of urbanization, various methods have been developed to return urban watersheds to a more natural state. These methods are collectively termed low impact development (LID) or "green" infrastructure. LID is defined by the U.S. Environmental Protection Agency (US EPA) as "systems and practices that use or mimic natural processes that result in the infiltration, evapotranspiration or use of stormwater in order to protect water quality and associated aquatic habitat." There is a wide range of stormwater best management practices (BMPs) that have been developed, and vary dramatically across regions, from engineered wetlands to permeable pavements.

A promising at-source stormwater BMP is bioretention (Roy-Poirier et al., 2010). Bioretention refers to systems that are designed to provide flood control and water quality treatment through a variety of natural processes including evapotranspiration, infiltration, biological uptake, microbial decomposition, etc.
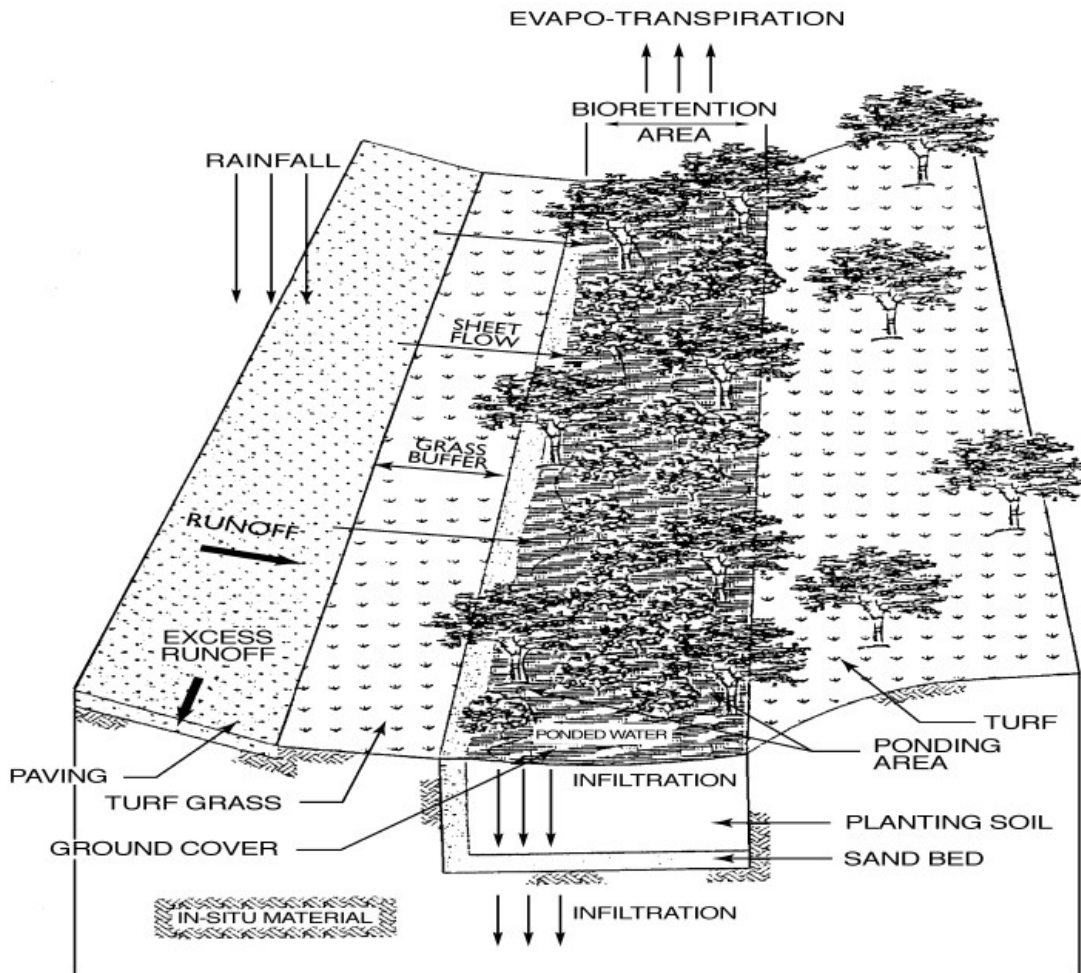
Figure 1. A Conventional Bioretention Basin. Source: (Prince George's County Department of Environmental Resources, 1993)

A bioretention system typically consists of a water storage area, vegetation, mulch, soil filter media, and in some cases an underdrain to decrease drainage time (Kim et al., 2012). A schematic of a conventional bioretention basin is shown in Figure 1. This is the original bioretention basin design proposed by Prince George's County, MD, Department of Environmental Resources (PGDER) when bioretention was first developed in the early 1990's and is still used in the US EPA's bioretention fact sheet (U.S. Environmental Protection Agency, 1999). Bioretention is an evolving stormwater technology and designs vary widely based on geography, application, local standards, etc.

A unique approach taken in Arizona to bioretention basin design is the use of dry wells located in the floor of the basins to directly inject stormwater runoff into the substrate. A dry well is a bored hole drilled into the unsaturated vadose zone for the purpose of infiltrating stormwater at depth. Dry wells rather than underdrains have been used extensively in retention and bioretention basins in Arizona, specifically in Maricopa County. The Arizona Department of Environmental Quality (ADEQ) had registered 40,586 drywells by the end of 2008, 96% of which were installed in Maricopa County. It is expected that this number is much higher today, as drywell registrations with ADEQ exceed 3,000 per year (Graf, 2010).

Figure 2 shows a schematic for a typical modern dry well. The pretreatment interceptor isn't always provided; many designs have the inlet grate flowing directly into the settling chamber in the dry well.
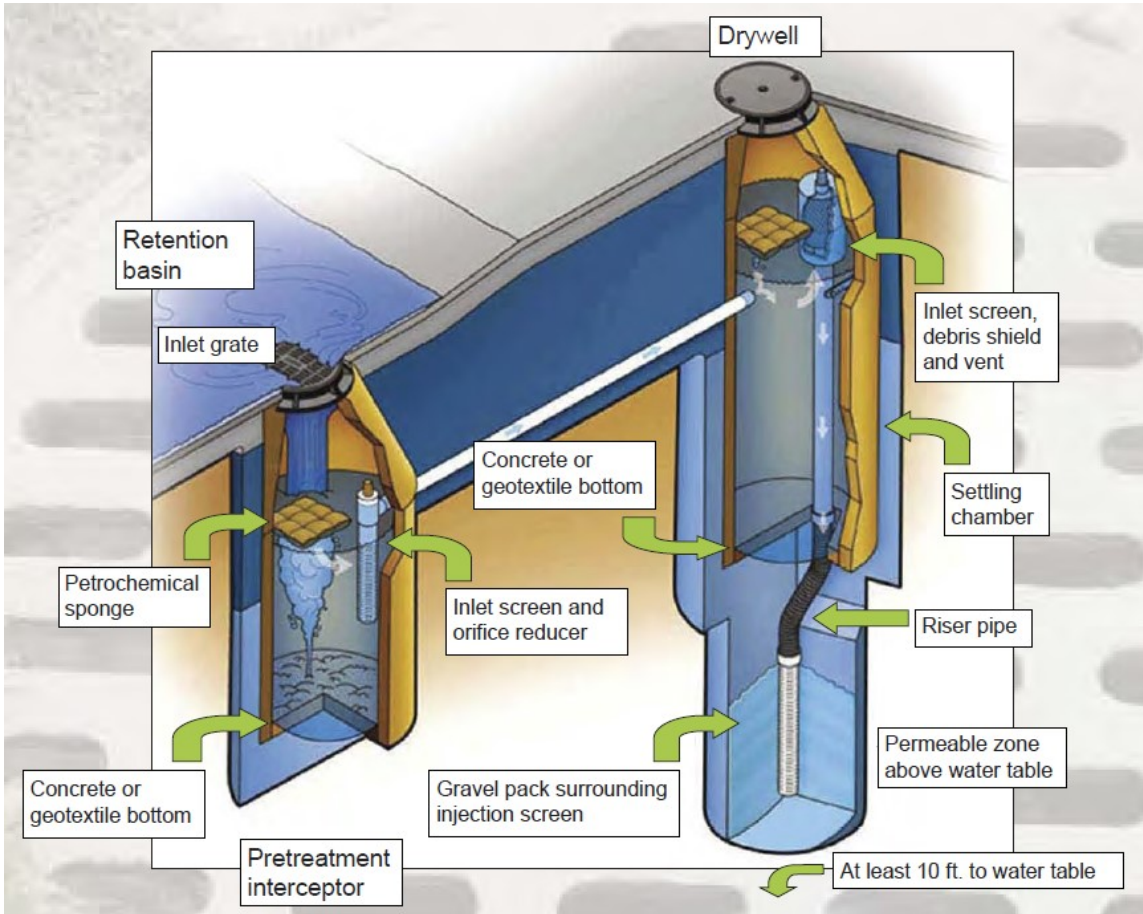
Figure 2. Configuration of a Typical Modern Dry Well in Maricopa County.
Source: www.torrentresources.com

Bioretention basins and dry wells are relatively new stormwater management technologies, and there is a lack of design methods for these BMPs. Most design procedures are overly simplistic, many times relying solely on regional drainage codes. There is a need for methods to assist designers in site specific optimal design recommendations.

1.2 Research Objective

The objective of this research is to develop a model to determine the optimal (least cost) design of bioretention basins and dry wells for the management of stormwater, given site specific parameters. The model was tailored for use in optimizing local design practices in Arizona, including the use of dry wells, but can be easily modified by the user to fit other design scenarios. The optimization model is solved using the General Algebraic Modeling System (GAMS).

Currently, design procedures for bioretention basins are simplistic and any optimization usually involves iterative trial-and-error procedures. In the Phoenix area, bioretention basin design is governed by the minimum requirements put forth in the Drainage Policies and Standards for Maricopa County, Arizona. The Flood Contol District of Maricopa County (FCDMC) requires that all new developments have a retention volume capable of retaining the 100 year, 2 hour duration storm within its boundaries, and that the basin drains within 36 hours (Flood Control District of Maricopa County, 2007). Designers determine the number of dry wells needed to meet the drain time requirement using an assumption of the dry well flow rate. The design dry well disposal flow rate is assumed to be 0.1 cfs unless a percolation test is performed on a

completed well at the site (Flood Control District of Maricopa County, 2007). Many times the basin must be redesigned once a higher flow rate is demonstrated on a completed well, at which point construction is already underway.

Another method for the determination of the number of dry wells needed in a basin is the ADEQ design recommendations: one dry well per 6,000 cubic feet of drainage volume draining paved areas, and one dry well per 15,000 cubic feet of drainage volume draining landscaped areas (Arizona Department of Environmental Quality, 2009). These simplistic measures do not take into account any site specific information beyond the estimated runoff volume. There is a need for an efficient method to determine the size of the basin, amount of dry wells, and dry well depths to meet the stormwater requirements at minimal cost.

The approach taken in this research is to create a nonlinear mathematical model to describe the inflows and outflows from the basin, compute the storm duration that will result in the maximum storage volume required, and design the optimum basin to provide the required stormwater management while taking into account associated costs, eliminating the need for trial-and-error.

1.3 Methodology and Phases of Research

The optimization model developed here is an expansion of the nonlinear programming (NLP) optimization model for the design of infiltration basins developed by Stafford et al. (2015). To expand the model for use with bioretention basins and dry wells, model development took place in four phases:

(1) Expand the existing infiltration basin optimization model to include bioretention.

(2) Addition of one dry well to the bioretention model.

(3) Model the number of dry wells as an integer variable, allowing multiple dry wells in the bioretention model.

(4) Application of the models developed.

Phase 1 - Bioretention

The primary difference between an infiltration basin and a bioretention basin is the addition of vegetation. The addition of an evapotranspiration (ET) outflow was the primary component in extending the infiltration basin model developed by Stafford et al. (2015) for use with bioretention basins. Two approaches to the estimation of the evapotranspiration rate are included in the model, one more data intensive than the other. Ponding depth in the basin was not accounted for in the original infiltration model, but is included in the present model. The calculation of the total drain time of the basin and a constraint limiting the drain time to less than 36 hours is added, the maximum allowed by FCDMC (Flood Control District of Maricopa County, 2007). The objective function is modified to reflect the additional costs of vegetation.

Phase 2 – Single Dry Well

Incorporating a dry well into the optimization model requires an analytical relationship between site specific data and the dry well flow rate. A literature review revealed a lack of research on the hydraulics of dry wells. This is presumably due to the limited geographical extent of the use of dry wells. The most applicable analytical relationship for the estimation of the flow rate from a dry well is used by the Washington

Department of Transportation and is based upon work done by the U.S. Bureau of Reclamation (USBR) in the early 1950's. This relationship is added to the model, and the objective function is modified to reflect the costs of the dry well.

Phase 3 – Variable Number of Dry Wells

Allowing the model to determine the optimal number of dry wells requires the use of mixed-integer nonlinear programming (MINLP) in order to model the number of dry wells as an integer variable. Modeling more than one dry well also requires knowledge of the hydraulic interaction of multiple dry wells. Due to the lack of research on dry wells, no analytical relationship was found describing the hydraulic effects of one dry well on another. The hydraulic interaction of multiple dry wells is avoided in the model by a user specified minimum spacing. This minimum spacing defines the upper limit for the number of dry wells in the basin, an integer variable.

Phase 4 – Application of the Models Developed

The models developed are solved for example applications using the General Algebraic Modeling System (GAMS). GAMS is a modeling system for mathematical programming and optimization that is capable of solving linear, nonlinear, and integer nonlinear programming. GAMS has been used to solved many problems in water and hydrology. Mays and Tung (1992) used GAMS to formulate many water resources optimization problems ranging from pipeline design to reservoir operation to groundwater management. GAMS is capable of solving nonlinear programming (NLP) problems (infiltration, bioretention, and bioretention with single dry well models) as well

as mixed-integer nonlinear programming (MINLP) problems (number of dry wells as an integer variable).

The models developed are applied to two example developments on the Arizona State University (ASU) campus to illustrate the optimization application. The first example is an existing development with a drainage plan that incorporates a bioretention basin. All phases of the model are run for the first example application to compare the results. The second example is a hypothetical larger development at the same location. The purpose of this example is to apply the final MINLP model developed to a larger watershed. The soil and ET parameters are varied to investigate how the model responds.

2. STORMWATER MANAGEMENT

2.1 Hydrologic Effects of Urbanization

Urban development is associated with an increase in impervious surfaces such as roadways, roofs, and sidewalks. An increase in imperviousness increases the volume of water available for runoff due to a reduction in infiltration. Traditionally, stormwater is routed from paved surfaces through storm sewers to concrete lined channels ("grey" infrastructure) designed with maximum hydraulic efficiency to convey water out of the urban area. Properly designed, this can effectively minimize flooding of urban areas during storm events, but also removes water from the local watershed that otherwise would recharge groundwater or irrigate vegetation.

A drastic example of the hydrologic effects of urbanization and "grey" stormwater infrastructure can be found in Los Angeles. Runoff to the ocean consisted of about 5% of the total precipitation in the Los Angeles River basin in the 1920's. After intense urbanization, a dramatic increase in paved surface area, and increased hydraulic efficiency associated with artificial channels and traditional storm drainage collection systems, the runoff to the ocean increased to about 50% of the total precipitation (Stephens et al., 2012). Under predevelopment conditions, this increased volume of runoff would have infiltrated to recharge the groundwater, naturally irrigated vegetation, and provided evaporative cooling.
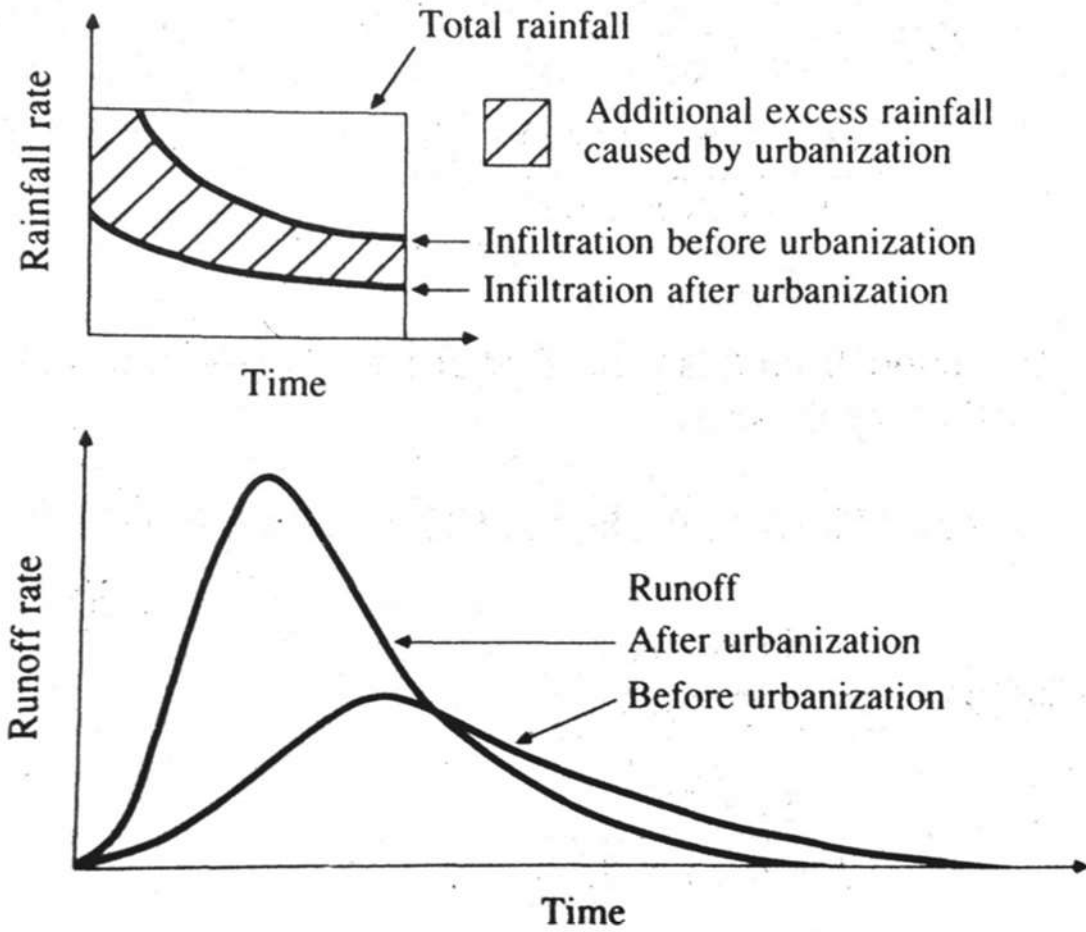
Figure 3. The Effect of Urbanization on the Storm Runoff Hydrograph. Source: (Chow et al., 1988)

An additional adverse effect of traditional stormwater management is a decrease in water quality. Increased urbanization has caused stormwater to become a significant source of water pollution, as all of the pollutants in the urban environment are efficiently transported with no treatment to a receiving body downstream. The US EPA has estimated that 30% of the known water pollution in the United States is a result of stormwater runoff.

2.2 BMPs for Stormwater Management

In order to manage stormwater to minimize the negative hydrologic effects of urbanization in an increasingly urban world, many stormwater best management practices (BMPs) have been developed. The development of BMPs in the United States began in earnest after 1990 when the need to control stormwater quality was legally recognized by the US EPA. Until the 1990's, stormwater management largely focused on flood protection.

Stormwater BMPs include wet ponds, green roofs, stormwater wetlands, porous pavements, grass swales, sand filters, dry detention, etc. Stormwater BMPs generally rely on storage, infiltration, and evapotranspiration to reduce the frequency and intensity of flood events and associated pollutant loads (Lee et al., 2010).

Many studies have been done on a variety of BMPs, providing a wide yet inconsistent set of data. There have been stormwater BMP manuals developed by cities, counties, and states across the U.S., all with varying recommendations and complexity. Attempting to review and summarize the information gathered from the various studies and design manuals reveals inconsistent study methods and lack of associated design

information (Strecker et al., 2001). There is no authoritative stormwater BMP manual in Arizona.

In 2012, the US EPA partnered with the City of Phoenix to create a report titled "Green Infrastructure Barriers and Opportunities in Phoenix, Arizona" (Mittman et al., 2013). Green infrastructure techniques identified for the Phoenix area include bioretention basins and bioswales consisting of depressed landscaped areas, simple surface treatments, mulching, and drought tolerant planting. A lack of an LID design manual for the area was identified as a limiting factor in the extent of green infrastructure practices. BMP manuals have been developed by The Los Angeles County Department of Public Works and the City of San Diego, which are generally applicable to the semiarid climate of Maricopa County, though neither manual covers the use of dry wells.

Detention, retention, and bioretention basins have become widespread in Arizona in part because of the requirements for onsite retention of stormwater. Table 1 presents a summary of the stormwater retention / detention requirements of several Arizona jurisdictions. One reason for the widespread use of dry wells in Arizona is the need for retention basins to meet the 36 hour drain time requirement.

Table 1. Stormwater Retention Requirements of Several Arizona Jurisdictions. Source: (Arizona Department of Environmental Quality, 2009)

CITY	BASE STORM EVENT	MAX RETENTION TIME	DRYWELL DEPTH	MAX DRAINAGE VOL / DRYWELL	OTHER REQUIREMENTS
Apache Jct.	10–year 24 hour storm	36 hours	150 ft	no maximum drainage volume	none
Chandler	100–year, 2 hour storm	36 hours	10 feet into permeable layer	—	—
Gilbert	50–year, 24 hour storm	36 hours	10 feet into permeable layer	43,500ft^3	• Minimum settling basin depth of 19 feet • Standard: MaxWell IV or approved equivalent • see city codes for more
Glendale	100% detention / retention of a 100–year, 2-hour event	36 hours per Maricopa County Health Dept. (pest control reqm't)	10 feet into a permeable layer	Depends on DW capacity.	Contractor submit Drywell Notice to ADEQ with copy to the City
Mesa	50–year, 24 hour	36 hours	10 feet into permeable layer; depth <75 ft.	9300 ft^3	Separate silting chamber in retention areas with more than 3 drywells

CITY	BASE STORM EVENT	MAX RETENTION TIME	DRYWELL DEPTH	MAX DRAINAGE VOL / DRYWELL	OTHER REQUIREMENTS
Phoenix	100–year, 2–hour	36 hours	10 feet into a permeable layer	Not to exceed 0.1 cfs per well unless a greater rate can be supported by a detailed, certified soils report.	• Design must conform with ADEQ guidelines • The City inspector must be present before backfill or well pipes are placed within any drywell bore holes. • Operator responsible for cleaning and maintenance of each structure to assure proper working order. • Regular maintenance schedule shall not exceed 3 years.
Scottsdale	100–year, 2–hour	36 hours	—	—	• discharge – stored stormwater to the underground must be in accordance with the approved groundwater master plan and approved by the appropriate city officials. • must secure proper state and federal permits that allow stormwater discharge to the underground
Tempe	100–year, 1–hour	36 hours	—	—	—
Tucson		18 hours	10 feet into permeable layer	—	—

2.3 Infiltration Basins

An infiltration basin is a stormwater BMP where the primary method of water quality treatment and volume reduction is through infiltration. Infiltration basins are a type of retention basin, but are located in areas of high permeability soils so that the stored stormwater infiltrates into the ground rather than being stored permanently. Infiltration basins may have an emergency spillway structure to direct excess stormwater, but they are sized in order to infiltrate the design storm through the soil of the basin rather than discharge runoff through an outlet. Infiltration basins treat stormwater collected locally, or transported through storm sewers like the design shown in Figure 4. A primary benefit of infiltration basins over other stormwater BMPs is the groundwater recharge resulting from the increased infiltration.

Infiltration basins generally work best in areas with high permeability soils and a sufficiently deep groundwater table or bedrock elevation. The LA County BMP manual requires a minimum soil infiltration rate of 0.5 inches/hour and at least 10 feet of groundwater separation (County of Los Angeles Department of Public Works, 2010). Infiltration basins where site parameters allow for sufficient surface infiltration are generally preferred over systems with dry wells or other recharge methods because they offer the best opportunity for clogging control and the best soil-aquifer treatment of the infiltrating stormwater (Bouwer, 2002).
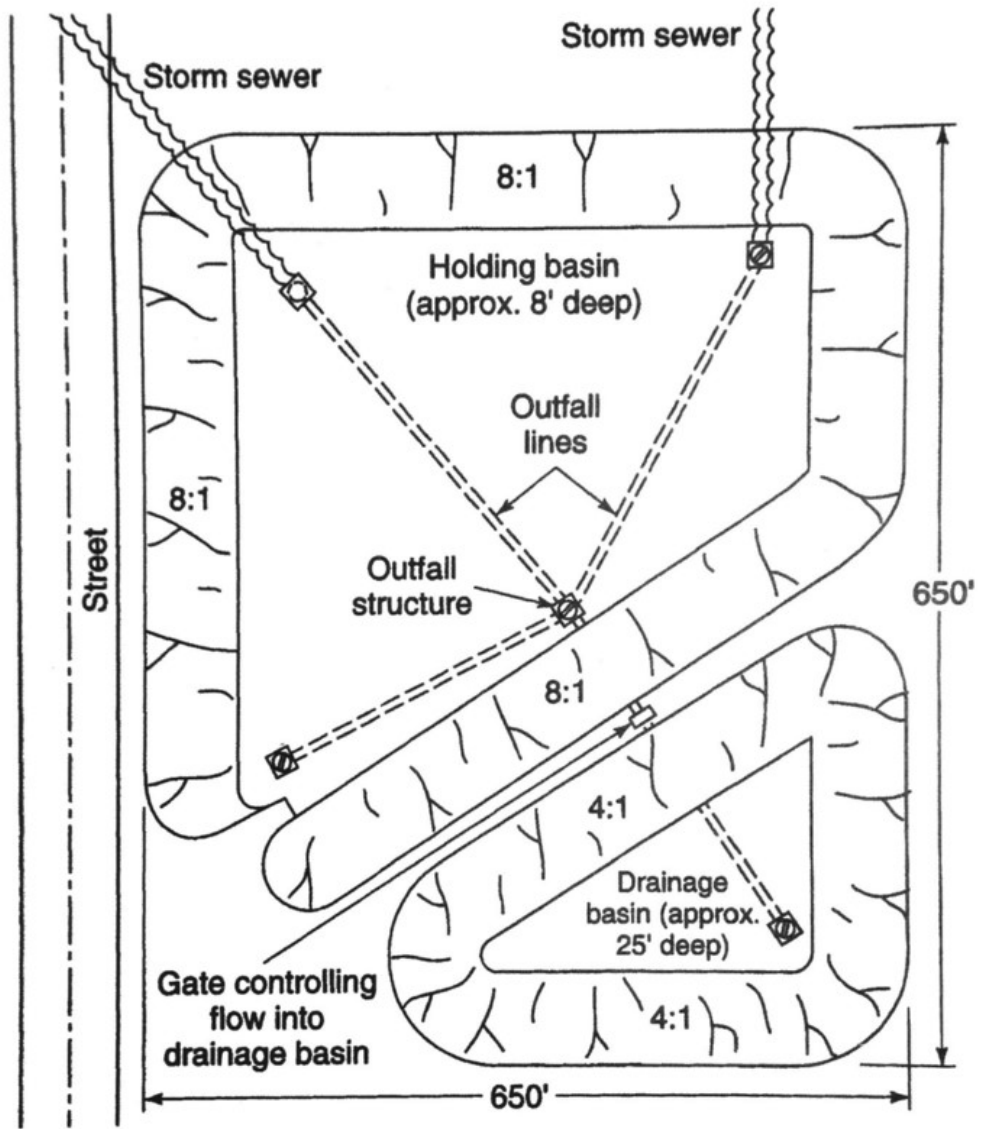
Figure 4. Typical Infiltration Basin Design. Source: (American Iron and Steel Institute, 1995)

2.4 Bioretention

Bioretention is one of the most frequently used stormwater management tools in urbanized watersheds (Davis et al., 2009). Bioretention basins can be as simple as a retention basin that is vegetated, usually with several types of vegetation including herbaceous groundcover species, shrubs, and trees. Bioretention basins are also referred to as "rain gardens" or "bioinfiltration basins". The concept of bioretention systems for stormwater treatment was inspired by similar natural systems to treat sewage effluents (Roy-Poirier et al., 2010).

Bioretention basins have many benefits in addition to those provided by a retention basin without vegetation, including increased water removal through transpiration, increased pollutant removal, and improvements in the livability of urban areas. Vegetation can also increase infiltration in a basin with root systems (Roy-Poirier et al., 2010). As these stormwater management facilities are located in dense urban areas, the additional benefits of shading, evaporative cooling, and aesthetics are welcome. Bioretention basins have been shown to successfully reduce pollutants such as suspended solids, nutrients, hydrocarbons, and heavy metals primarily through sedimentation, filtration, sorption, plant uptake and storage, and microbial decomposition (Davis et al., 2009).

The design of bioretention basins vary greatly across the country. Current design guidelines lack consistency across regions, and there is a lack of knowledge on the

Figure 5. A Bioretention Basin Located on the Arizona State University Campus. Photo: Mason Lacy

performance of bioretention systems in arid or semiarid climates (C. Houdeshel et al., 2012; Roy-Poirier et al., 2010). Bioretention in arid and semi-arid climates presents both opportunities and challenges.

Mulch is rarely used in xeric climates as it becomes sun-faded, then must be disposed of and replaced because the dry conditions do not provide an environment that promotes decomposition (C. Houdeshel et al., 2012). Gravel is used in place of mulch in many designs. Selecting plants for use in arid regions requires greater consideration due to limited water (C. Houdeshel and Pomeroy, 2010). Another important difference is the relative importance of infiltration and groundwater recharge in arid regions as opposed to more mesic regions. Many bioretention basins in Arizona incorporate drywells in an attempt to increase recharge.

In order to satisfy the LID goals of bioretention, the vegetation chosen for basins should ideally be able to sustain itself without irrigation. A primary goal of green infrastructure is to limit the use of irrigation in landscaping (Davis et al., 2009). This can be a challenge in water limited regions such as the arid southwest. Few non-native vegetation species can survive in the dry conditions as well as inundation during a storm. This is one reason native plant species are common in successful bioretention systems in xeric climates. Native, non-invasive plants are also preferable as seeds and plant materials may be transported into natural receiving waters and environments ideal for natural recruitment (C. Houdeshel and Pomeroy, 2010). The American Society of Civil Engineers (ASCE) plant suggestions for use in bioretention in Phoenix are shown in Table 2.

Table 2. Suggested Plants for Use in Bioretention in Phoenix, AZ. Source: (C. Houdeshel and Pomeroy, 2010)

Scientific Name	Common Name	Favorable Traits	Successional Stage	Photosynthetic Pathways
Encelia farinosa	Brittlebush	High C:N ratio,high growth rate, drought tolerant, yellow flowers.	Late	C3
Larrea tridentata	Creosote Bush	Evergreen, extensive and shallow roots, high C:N ratio, tolerates inundation, yellow flowers.	Late	C3
Schizachyrium scoparium	Little Bluestem	Adapted to wide range of soils, high drought tolerance, fair flood tolerance, medium C:N ratio.	Late	C4
Spaeralcea ambigua	Desert Globmallow	High C:N ratio, biennial, high seedling vigor, perennial, drought tolerant.	Transitional	Unknown
Panicum virgatum	Switchgrass	Tolerates flooding, widely adapted, drought tolerant, medium C:N ratio, early spring growth.	Transitional	C4
Distichlis stricta	Saltgrass	High C:N ratio, rhizomatous, drought tolerant, salt tolerant.	Transitional	C4
Bouteloua repens	Slender Gramagrass	Perennial, widely adapted to many soil types, high C:N ratio, tolerates inundation.	Early	C4
Escholzia glyptosperma	Desert Poppy	Annual, variable seed germination rate, inexpensive, extensive tap root compared to most annuals.	Early	C3

Figure 6. A Non-Native South American Mesquite Uprooted in a Storm. Photo: Mason Lacy

An example of the superior performance of native vegetation over foreign vegetation can be seen in mesquite trees used in Arizona. South American mesquite species are commonly used as landscape trees in place of native velvet mesquite due to their ability to grow faster and create dense shade canopies. However, these South American mesquites require regular irrigation, produce shallow roots that can damage nearby hardscape and become vulnerable to uprooting in storms, produce flowers that are not as attractive to native bees and birds, and hybridize with native species (MacAdam, 2012).

2.5 Dry Wells

In arid environments with large water demands, water supply can rely heavily on groundwater. Often this results in over withdrawal from groundwater aquifers, depleting vital resources. Recharging depleted groundwater reserves is often a primary water management goal in arid regions. Changes in stormwater management have been seen as a way to increase natural recharge, and many unique techniques have been developed to promote groundwater recharge. In Maricopa County that has meant dry wells.

The Arizona Department of Environmental Quality (ADEQ) defines a dry well as "a bored, drilled, or driven shaft or hole with a depth that is greater than its width and that is designed and constructed specifically for the disposal of stormwater" (Arizona Department of Environmental Quality, 2009). A dry well can be as simple as an excavated pit or a pre-fabricated storage chamber. Original dry well designs consisted of a pit filled with gravel, riprap, rubble or other debris (Poribesh, Accessed 2016).
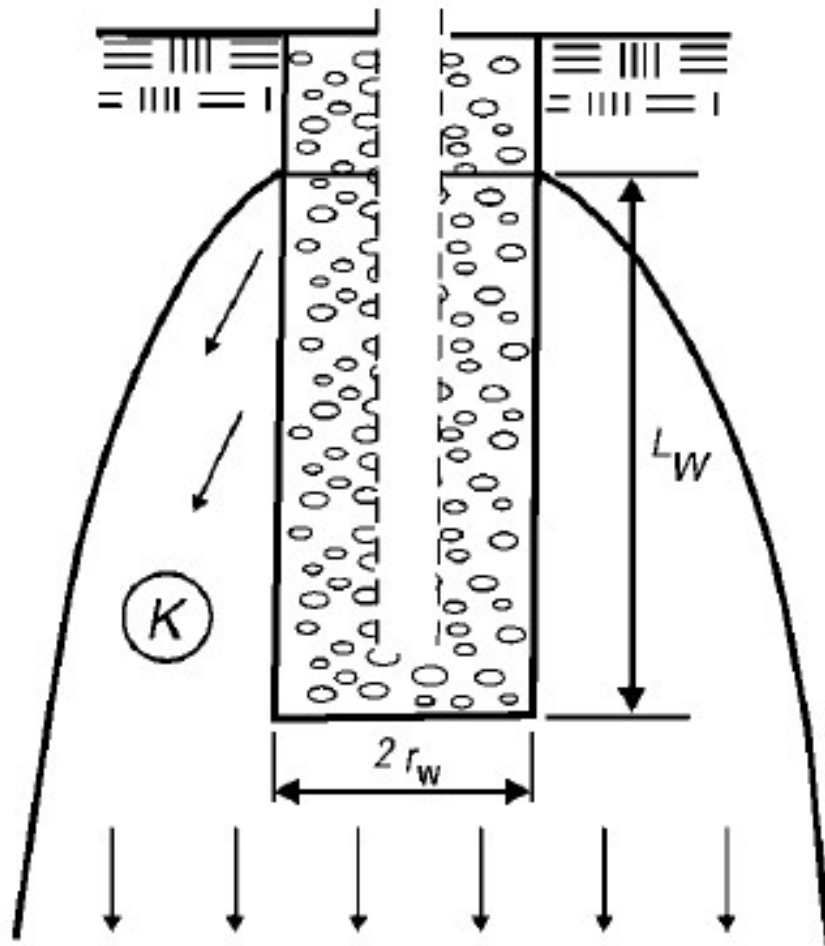
Figure 7. Section of a Simple Dry Well with Sand or Gravel Gill and Perforated Supply Pipe. The arrows represent downward flow in the wetted zone with hydraulic conductivity K. Source: (Bouwer, 2002)

Figure 7 shows a section view of a basic vadose-zone recharge well (another term for a dry well). The Washington State Department of Transportation (WSDOT) uses dry wells consisting of pre-cast concrete cylinders with drain holes. A schematic of a typical dry well configuration used by WSDOT is shown in Figure 8.

A concern with dry wells is that if polluted stormwater is directly injected into the groundwater, the aquifer can be rendered useless as a drinking water source (Arizona Department of Environmental Quality, 2009). Another disadvantage of dry wells is that eventually their infiltrating surface will clog up because of accumulation of suspended soil soils and/or biomass. They cannot be pumped for "backwashing" the clogging layer because they are located in the vadose zone (Bouwer, 2002).

To minimize these consequences, dry well regulation in Arizona requires pretreatment of stormwater prior to injection into the substrate. ADEQ maintains a list of recommended flow control and pre-treatment technologies for dry wells. The flow control technologies include manual or automatic normally closed valves, a raised dry well inlet within the retention basin, magnetic mat or cap, primary sump, interceptor, or settling chamber. Pre-treatment technologies include combined settling chamber and oil/water separator, filtration / adsorption, passive skimmers, or catch basin inlet filters (Arizona Department of Environmental Quality, 2009). Modern dry well construction in Arizona usually includes a settling chamber and an absorbent sponge to provide removal of pavement oils. The standard dry well system typically has a minimum effective settling capacity of 1,000 gallons per chamber, resulting in a settling chamber 16 feet deep for a four foot ID chamber (Arizona Department of Environmental Quality, 2009).
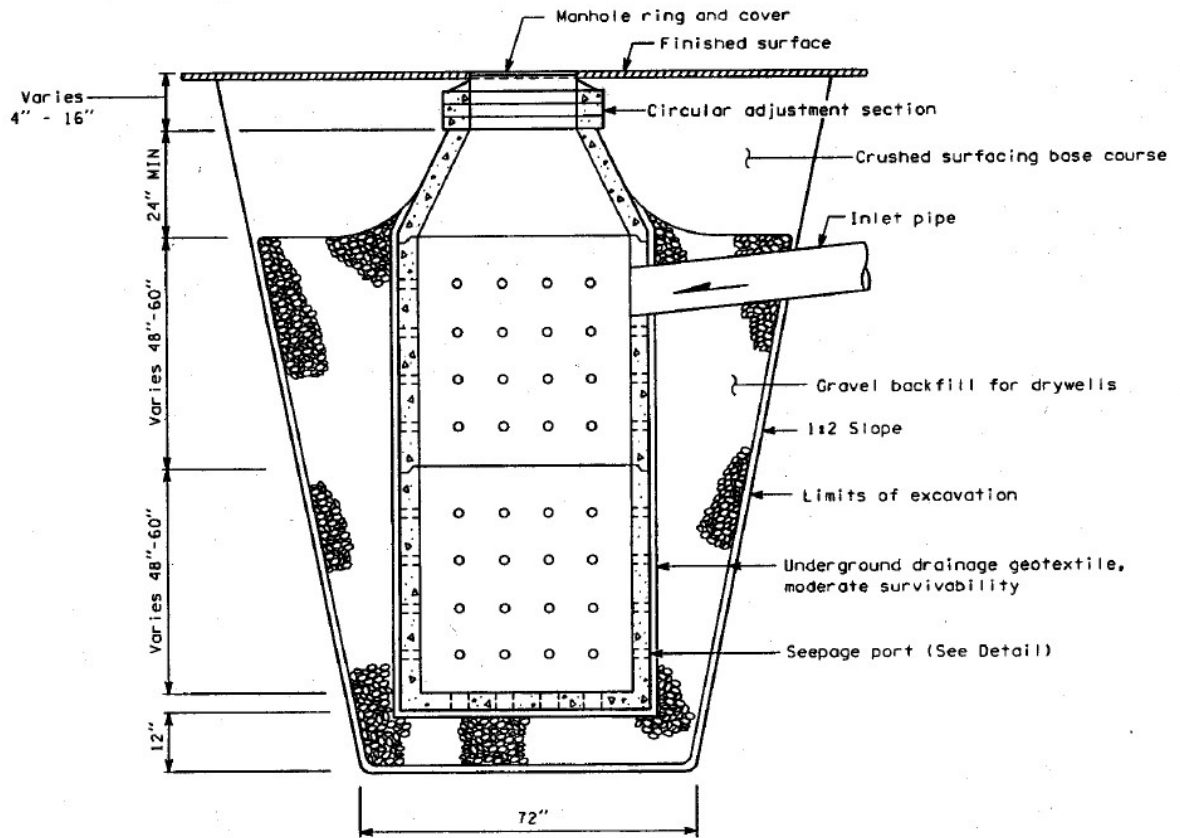
Figure 8. Plans for a Pre-cast Concrete Dry Well Similar to Those Used by WSDOT.
Source: (Massmann, 2004)

ADEQ recommends a hydrophobic petrochemical absorbent with a minimum capacity of 128 ounces. A design detail for a typical dry well installed in Arizona is shown in Figure 9. Dry wells are typically located in the floor of a retention or bioretention basin to accept stormwater runoff. ADEQ recommends that dry well surface grates are raised at least three inches above the bottom of landscaped retention basins, see Figure 10.

The depth of the dry well is required to be a minimum of 10ft above the groundwater table in order to provide additional filtration of the stormwater before it enters the aquifer (Flood Control District of Maricopa County, 2007). Dry wells in Maricopa County range from 19ft deep to depths greater than 120ft, with dry wells as deep as 180ft being constructed (Graf, 2010). The depths of dry wells registered with ADEQ are shown in Figure 11.

The majority of dry wells in the United States have been installed in Arizona, however Washington has more than 26,000 registered dry wells and Oregon has over 27,000 (Graf, 2010). The number of dry wells registered by the Arizona Department of Environmental Quality (ADEQ) is shown in Figure 11.

Proper maintenance is extremely important to the functioning of dry wells. ADEQ recommends inspections of dry wells at least annually, or if water remains standing in the retention basin for longer than 36 hours (Arizona Department of Environmental Quality, 2009). Maintenance should include removal of all sediment, cleaning of all filters and screens and replacement of chemical absorbents. Failure to properly maintain a dry well can result in premature clogging of the infiltration media and/or pollutants entering the groundwater.

ITEM NUMBERS

1. **Manhole Cone** - Modified Flat Bottom.

2. **Moisture Membrane** - 6 Mil. Plastic. Applies only when native material is used for backfill. Place membrane securely against eccentric cone and hole sidewall.

3. **Bolted Ring & Grate** - Diameter as shown. Clean cast iron with wording "Storm Water Only" in raised letters. **Bolted in 2 locations** and secured to cone with mortar. Rim elevation ±0.02' of plans.

4. **Graded Basin or Paving** (by Others).

5. **Compacted Base Material** - 1-Sack Slurry except in landscaped installtions with no pipe connections.

6. **PureFlo® Debris Shield** - Rolled 16 ga. steel X 24" length with vented anti-siphon and Internal .265" Max. SWO flattened expanded steel screen X 12" length. **Fusion bonded epoxy coated.**

7. **Pre-cast Liner** - 4000 PSI concrete 48" ID. X 54" OD. **Center in hole** and align sections **to maximize bearing surface.**

8. **Min. 6' Ø Drilled Shaft.**

9. **Support Bracket** - Formed 12 Ga. steel. **Fusion bonded epoxy coated.**

10. **Overflow Pipe** - Sch. 40 PVC mated to drainage pipe at base seal.

11. **Drainage Pipe** - ADS highway grade with TRI-A coupler. **Suspend pipe** during backfill operations to prevent buckling or breakage. Diameter as noted.

12. **Base Seal** - Geotextile or concrete slurry.

13. **Rock** - Washed, sized between 3/8" and 1-1/2" to **best complement soil conditions.**

14. **FloFast® Drainage Screen** - Sch. 40 PVC 0.120" slotted well screen with 32 slots per row/ft. Diameter varies 120" overall length with TRI-B coupler.

15. **Min. 4' Ø Shaft** - Drilled to **maintain permeability** of drainage soils.

16. **Fabric Seal** - U.V. resistant geotextile - **to be removed by customer** at project completion.

17. **Absorbent** – Hydrophobic Petrochemical Sponge. Min. to 128 oz. capacity.

18. **Freeboard Depth** Varies with inlet pipe elevation. Increase settling chamber depth as needed to maintain all inlet pipe elevations above overflow pipe inlet.

19. **Optional Inlet Pipe** (Maximum 4", by Others). Extend moisture membrane and compacted base material or 1 sack slurry backfill below pipe invert.

Figure 9. Dry Well Design Detail. Source: www.torrentresources.com

Figure 10. Inlet to a Dry Well Located in the Bioretention Basin Shown in Figure 5. Photo: Mason Lacy

Figure 11. Number of Registered Dry Wells in Arizona, Above, and Their Depths, Below. Source: (Graf, 2010)

3. PREVIOUS OPTIMIZATION MODELS FOR STORMWATER MANAGEMENT

3.1 Storm Sewer Systems

Storm sewer systems were the first stormwater management systems to be optimized. Optimization models for storm sewer system design were first developed in the mid-1960's (Deininger, 1966; Holland, 1966). Linear programming (LP) and non-linear programming (NLP) were introduced in these early models. These optimization procedures had many limitations, for example the inability deal with integer values such as discrete pipe diameters. These models were developed based on a given sewer system layout.

Later models took advantage of new optimization techniques such as dynamic programming (DP), discrete differential dynamic programming (DDDP), and evolutionary algorithms. DP and DDDP provided many benefits, including the ability to model discrete pipe diameters (Mays et al., 1976; Merritt and Bogan, 1973; Tang et al., 1975). With the advances in optimization procedures, model developers were able to create models considering the optimal layout of storm sewer systems as well as the optimal pipe design (Argaman et al., 1973; Mays et al., 1976).

Genetic algorithms have been popular for sewer system optimization since an evolutionary algorithm was first applied to the problem by Cembrowicz and Krauter (1987). As optimization techniques continue to evolve, new ways to approach the problem evolve as well. Afshar (2010) applied an ant colony optimization algorithm to the optimal storm sewer system problem. Karovic and Mays (2014) developed an optimization model for the design of storm sewers for given layouts using simulated annealing. Steele et al. (2016) developed a model combining the layout problem and

optimal pipe design. The procedure solves the mixed-integer nonlinear programming (MINLP) problem using GAMS and the simulated annealing procedure for design developed by Karovic and Mays (2014).

3.2 Detention and Retention Networks

Detention and retention basins are both areas where excess stormwater is stored, but differ in that detention basins slowly drain, usually through an outlet at the base of the basin, while retention basins store the water indefinitely until it has infiltrated, evaporated or transpired (Harris County Flood Control District, Accessed 2016). In locations with heavy rainfall and low infiltration rates, water may remain in a retention basin permanently.

Detention basin networks have been the subject of several optimization models using dynamic programming, as well as genetic algorithms (Bennett and Mays, 1985; Mays and Bedient, 1982; Yeh and Labadie, 1997). More recently Oxley and Mays (2014) developed an optimization model for detention basin systems using a simulated annealing procedure and interfacing with the U.S. Army Corps of Engineer's Hydrologic Engineering Center – Hydrologic Modeling System (HEC-HMS). This model optimized the size and location of detention systems, including outlet structures.

Retention basins have been considered as a network less often than detention basins, resulting in less optimization models for retention systems. In the case of a complex watershed with multiple sub-watersheds, sizing basins to only retain flow from their contributing subwatershed may result in a suboptimal solution in terms of flood control and cost. Travis and Mays (2008) developed a discrete dynamic programming

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technique to determine the optimal location and geometry of a system of retention basins within a watershed. The primary means of disposal of stormwater in the retention basins in the model is through infiltration.

3.3 Infiltration Basins

There have been design aids and methods developed for infiltration basins, though optimization models for infiltration basins considered individually are rare. Akan (2002) developed a design aid for stormwater infiltration structures, though there was no optimization component. Perez-Pedini et al. (2005) developed a procedure including a hydrological model of an urban watershed and a genetic algorithm to determine the optimal location for infiltration facilities. This model did not actually design the infiltration facilities.

Stafford et al. (2015) developed a model for the purpose of optimizing the design of infiltration basins. The non-linear programming (NLP) model uses the modified rational method to develop a hydrograph for the design storm event and the Green-Ampt infiltration method to estimate infiltration. This model was extended for the design of bioretention basins with dry wells in this study.

The objective of the optimization model is to determine the infiltration basin design with minimum cost. The objective function takes into account land cost, cost of excavation, and cost of hydroseeding the basin as expressed below:

$$\text{Minimize } Z = U_E(LWd) + U_s(A_b) + U_L(LW) \tag{1}$$

where L, W, and d are, respectively, the length, width, and depth of the infiltration basin

(ft), A_b is the area of the basin (ft^2), U_E is the unit cost of excavation ($/ft^3), U_S is the unit

cost of seeding the basin ($/ft^2), and U_L is the unit cost of land ($/ft^2).

The objective function is subject to the following constraints:

(a) Relationship defining basin area and volume based on size, shape and depth

(b) Rainfall-intensity-duration relationship for the design location

(c) Time of concentration

(d) The modified rational method determining the design hydrograph

(e) Relationship defining the critical storm duration to maximize the detention

   volume

(f) Infiltration volume as a function of time (Green-Ampt method)

(g) Infiltration rate as a function of time (Green-Ampt method)

(h) Volume of storage required to store the storm runoff not infiltrated at the

   critical detention time

(i) Relationships defining the effects of groundwater mounding and a limit on the

   maximum mound height

The rainfall intensity-duration-frequency (IDF) relationship defines the rainfall

intensity, and can be established for a drainage area. The expression for the IDF

relationship is defined as:

$$i = \frac{a}{(T_d+b)^n} \tag{2}$$

where i is the intensity of rainfall (in/hr), a, b, and n are regional scalars, and T_d is the

storm duration (min). The time of concentration is computed using the SCS lag equation

$$t_c = \frac{100L^{0.8}\left[\left(\frac{1000}{CN}\right)-9\right]^{0.7}}{19000S^{0.5}} \tag{3}$$

where t_c is the time of concentration (min), L is the hydraulic length of the watershed (ft), CN is the SCS runoff curve number, and S is the average watershed slope. The inflow runoff volume into the basin is defined using the modified rational method hydrograph. A general representation of hydrographs produced using the modified rational method is shown in Figure 12. The equation for the stormwater inflow is

$$V_{in} = 60Q_P(0.5)[(T_d - t_c) + (T_d + t_c)] \qquad (4)$$

where V_{in} is the stormwater inflow volume (ft^3), Q_P is the peak flow (cfs), T_d is the time of duration (min), and t_c is the time of concentration (min) as defined in Equation 3. The outflow from the basin is defined using the Green-Ampt infiltration method.

Green and Ampt (1911) proposed an infiltration approach which simplifies the infiltration process, defining the wetting front as a sharp boundary dividing the saturated soil above from the soil with initial moisture content θ_i. Figure 13 shows a depiction of the Green-Ampt infiltration model, where the vertical axis is the distance from the soil surface and the horizontal axis is the moisture content of the soil.

The Green and Ampt equation for total volume infiltrated is:

$$F(t) = Kt + \psi\Delta\theta\ln\left(1 + \frac{F(t)}{\psi\Delta\theta}\right) \qquad (5)$$

where F(t) is the cumulative volume of water infiltrated (in) at time t (hr), K is the hydraulic conductivity (in/hr), ψ is the wetting front soil suction head (in), and $\Delta\theta$ is the change in moisture content defined as $\Delta\theta = \eta - \theta_i$ where η is the porosity and θ_i is the initial soil moisture content. The infiltration rate is expressed as:

$$f = K\left(1 + \frac{\psi\Delta\theta}{F}\right) \qquad (6)$$

where f is the infiltration rate (in/hr), and F is the cumulative infiltration defined in Equation 5. The outflow volume at the time of duration is defined using the Green-Ampt infiltration volume:

$$V_{out} = A_b F(T_d/60)/12 \tag{7}$$

where V_{out} is the volume infiltrated (ft^3) at time T_d (min), A_b is the area of the basin (ft^2), and F is the cumulative infiltration (in) as defined by Equation 5. The detention volume required for the infiltration basin is given by the difference between the inflow volume and the outflow volume:

$$V_d = V_{in} - V_{out} \tag{8}$$

where V_d is the detention volume required, and V_{in} and V_{out} are the inflow volume and outflow volume defined, respectively, by Equations 4 and 7.

Differentiating Equation 8 for detention volume V_d with respect to the duration T_d and setting the first derivate equal to zero results in an expression for the duration time resulting in the maximum required detention volume:

$$60Q_P A(-an)\frac{T_m}{(T_m+b)^{n+1}} + 60Q_P \left(\frac{a}{(T_m+b)^n}\right) A - A_b f(T_m) = 0 \tag{9}$$

where T_m is the duration time (min) resulting in the maximum detention volume, Q_P is the peak inflow to the basin (cfs), A is the watershed area (acres), and a, b, and n are unit-less parameters defining the IDF relationship. The required basin volume V_{req} is then computed with Equations 4, 7, and 8:

$$V_{req} = 60Q_P(0.5)[(T_m - t_c) + (T_m + t_c)] - A_b F(\tfrac{T_m}{60})/12 \tag{10}$$

The NLP problem is solved using GAMS.
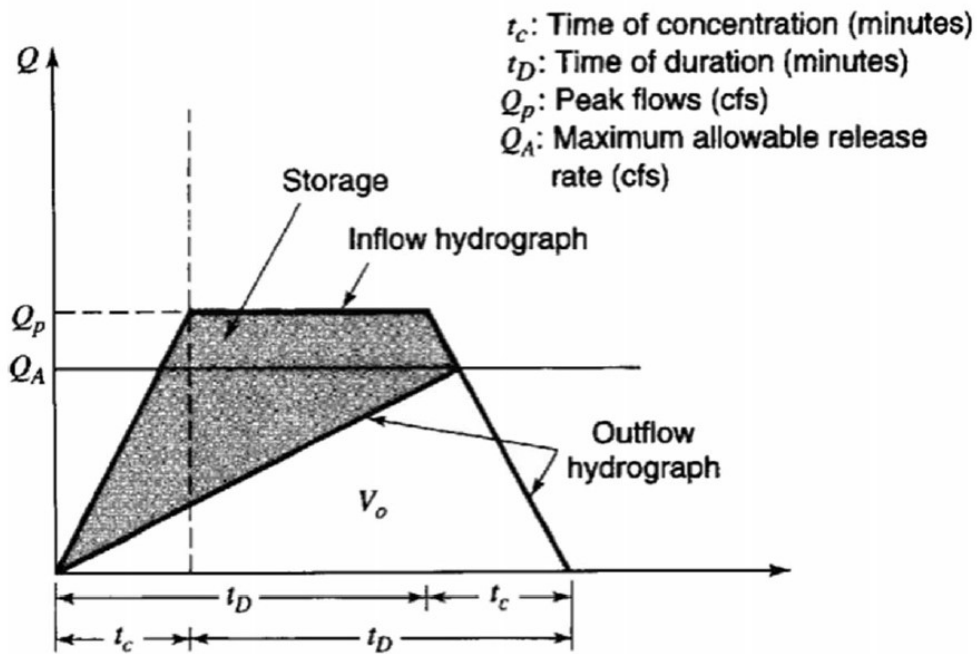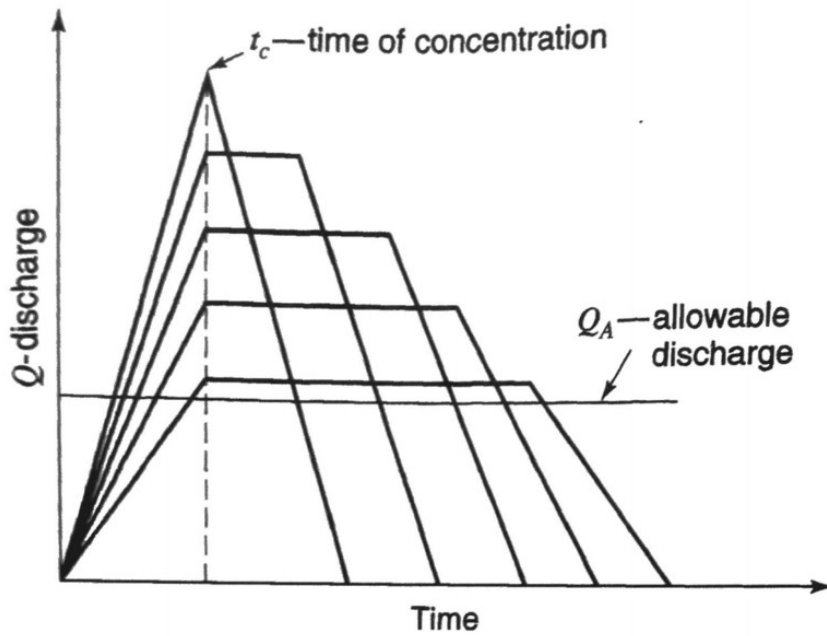
Figure 12. Modified Rational Method Hydrographs. Above: Hydrographs for Different Durations. Below: Required Storage Volume. Source: (Mays, 2005)
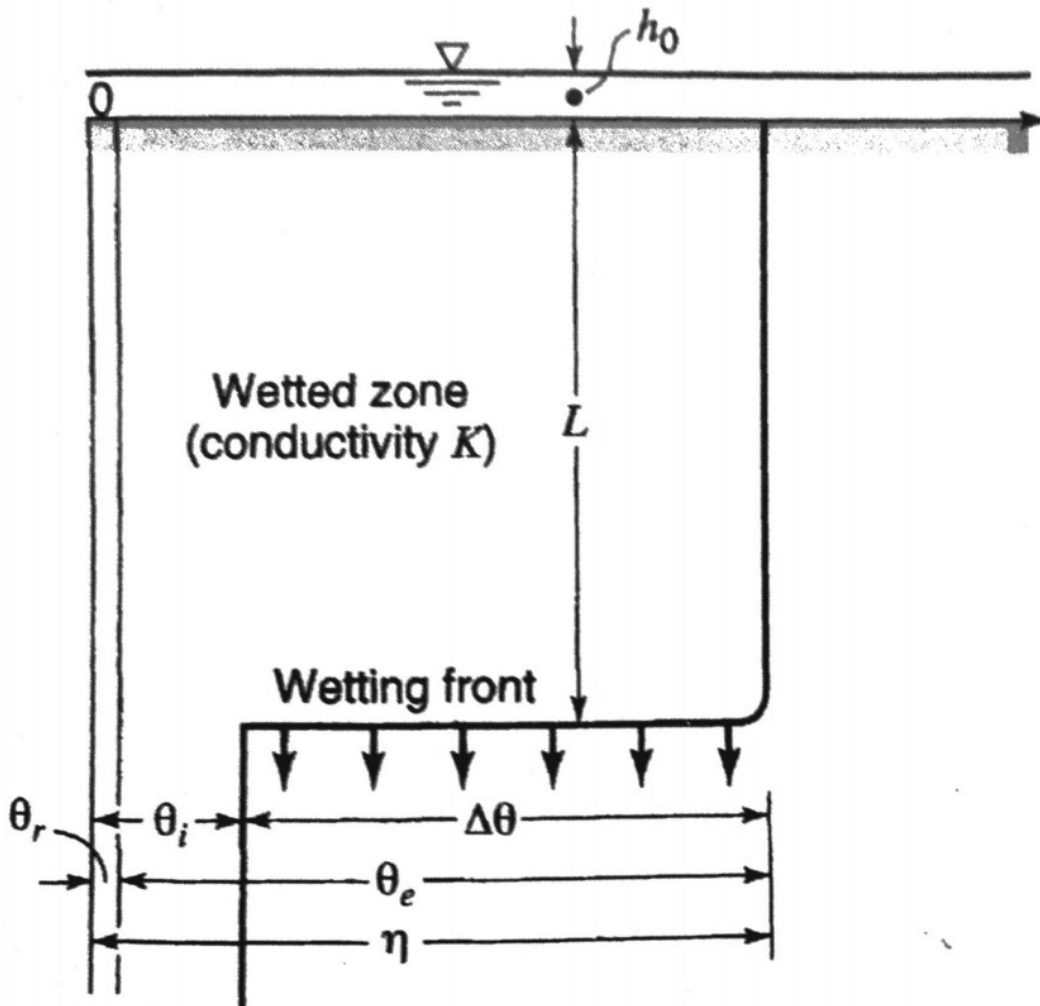
Figure 13. Graphical Depiction of the Green-Ampt Infiltration Model. Source: (Chow et al., 1988)

3.4 Vegetative Filter Strips

A vegetative filter strip (VFS) is a strip of soil and vegetation (usually grass) located along the length of a paved surface for the purpose of stormwater management and treatment. They are commonly used along roadways, parking lots, and also in agricultural areas to treat runoff. A schematic of a typical VFS system for an agricultural area is shown in Figure 14. There have been several physical and mathematical models developed for VFS systems for stormwater management, though there is a lack of design optimization models. One exception is the optimization model developed by Khatavkar (2015). The NLP model includes optimization for stormwater management as well as sediment control. For the formulation of the model, the VFS system was assumed to be located along an impervious surface such as a parking lot or road. The objective of the model is to determine the minimum length of VFS that would provide adequate infiltration while using the minimum area of land. The kinematic wave equation is used for modeling the overland sheet flow and the discharge per unit width is defined using Manning's equation. The infiltration is modeled using the Green-Ampt method. The equations are solved using a finite difference scheme. Khatavkar (2015) solves the model using GAMS and Microsoft Excel.
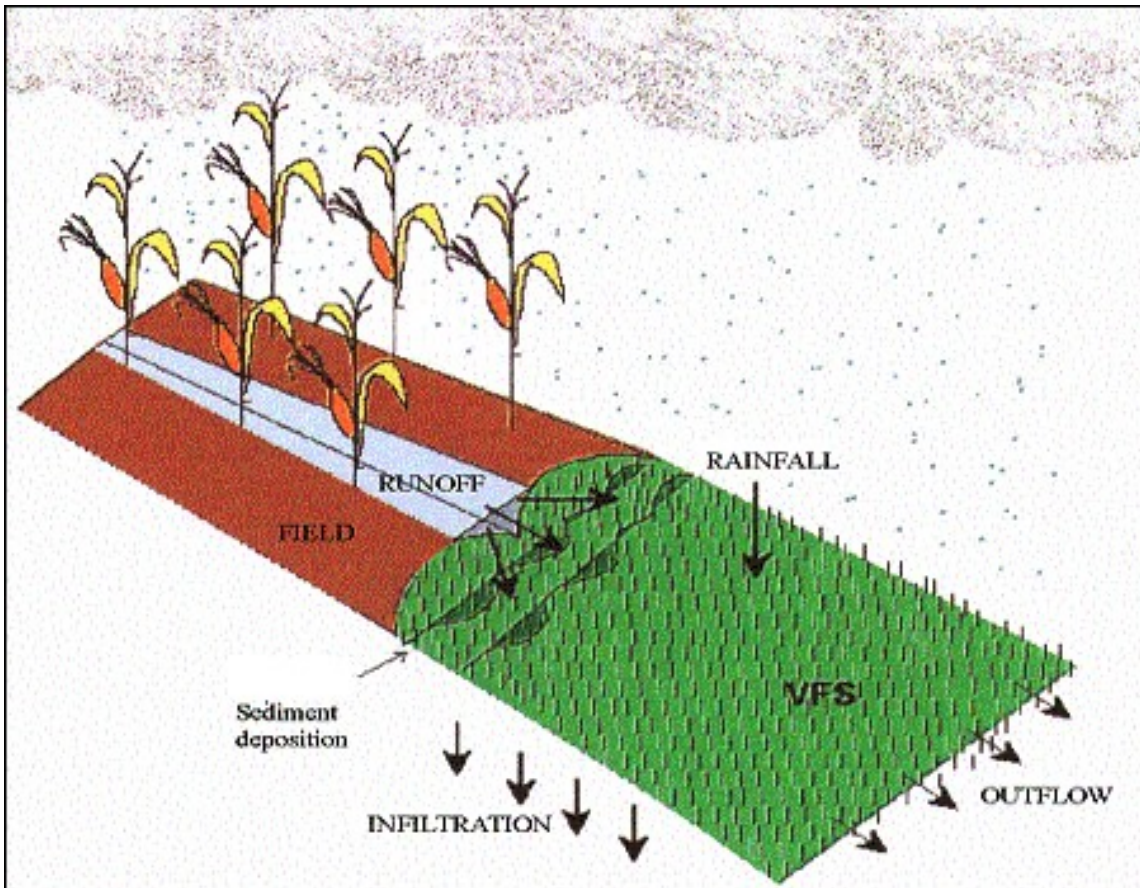
Figure 14. A Vegetative Filter Strip for Agriculture. Source: University of Florida
(http://abe.ufl.edu/carpena/vfsmod/)

4. MODEL DEVELOPMENT

The objective of the optimization model is to design the minimum cost basin with adequate storage volume and outflow capacity such that any stormwater inflows are temporarily stored and removed. The outflows for the three phases of the model differ as follows:

- Phase 1 model (bioretention), outflows are through infiltration and evapotranspiration.

- Phase 2 model (bioretention and single dry well), outflows are through infiltration, evapotranspiration and a single dry well.

- Phase 3 model (bioretention and variable number of dry wells), outflows are through infiltration, evapotranspiration and, if the number of dry wells is greater than zero, one or more dry wells.

The objective function for the Phase 1 model (bioretention) is subject to the following constraints:

(a) Relationship defining basin area and volume based on size, shape and depth.

(b) Rainfall-intensity-duration relationship for the design location.

(c) Time of concentration.

(d) The modified rational method determining the design hydrograph.

(e) Relationship defining the critical storm duration to maximize the detention volume based on infiltration and evapotranspiration outflows.

(f) Infiltration volume as a function of time (Green-Ampt method).

(g) Infiltration rate as a function of time (Green-Ampt method).

(h) Evapotranspiration rate (Penman-Monteith method or Hargreaves-Samani method).

(i) Volume of storage required to store the storm runoff not infiltrated or evapotranspired at the critical detention time.

(j) Total basin drain time must be less than 36 hours.

The objective function for the Phase 2 model (bioretention and single dry well) is subject to the same constraints as above, with the following additions / revisions:

(k) Dry well flow rate relationship based on soil properties and dry well depth.

(l) Maximum dry well depth is 10 ft above aquifer depth.

- The determination of critical storm duration, Constraint (e), is revised to include dry well outflow in determination of critical storm duration in addition to infiltration and evapotranspiration.

- The calculation of storage volume of required, Constraint (i), is revised to include dry well flow rate.

The objective function for the Phase 3 model (bioretention and variable number of dry wells) is subject to the same constraints as the Phase 2 model, except that dry well outflows are multiplied by the integer variable of number of dry wells. Dependent on the value of the integer variable, dry well outflows could be zero, from one dry well, or multiple dry wells. The following constraint is added to the model as an upper limit to the number of dry wells in the basin:

(m) Integer variable number of dry wells is less than maximum number of dry wells given by minimum spacing required to prevent hydraulic interference.

4.1 Objective Function

Phase 1 Bioretention

      The objective function used in the infiltration basin model (Equation 1) takes into account the cost of land, excavation, and hydroseeding (Stafford et al., 2015). This objective function was modified for the Phase 1 model to take into account the cost of vegetation, rather than the cost of hydroseeding. The land costs and cost of excavation remain. Three different types of vegetation species is the minimum recommended in many stormwater BMP manuals, as different species provide differing benefits, increased resilience, and resistance to diseases. A tree species, shrub species, and a grass species are included in the model. The modified objective function taking into account vegetation costs is shown below:

$$\text{Minimize } Z = U_e(V_b) + \left(U_t S_t^{-2} + U_{sh} S_{sh}^{-2} + U_g S_g^{-2}\right)A_v + U_l(A_b) \tag{11}$$

where Z is the total cost ($), U_e is the unit cost of excavation ($/ft^3), V_b is the total volume of the basin including freeboard (ft^3), U_t, U_{sh}, and U_g are the unit cost for each tree, shrub, and grass ($), S_t, S_{sh}, and S_g are the spacing of trees, shrubs, and grasses (ft), A_v is the vegetated area of the basin (ft^2), U_l is the unit cost of land ($/ft^2), and A_b is the total area of the basin (ft^2).

Phase 2 Bioretention and Single Dry Well

      Dry well costs reflecting the addition of the dry well are added to the objective function. Dry well costs include one-time costs per dry well (settling chamber cost, mobilization, etc.), and drilling costs associated with the depth of the dry well.

$$\text{Min } Z = U_e(V_b) + \left(U_t S_t^{-2} + U_{sh} S_{sh}^{-2} + U_g S_g^{-2}\right)A_v + U_l(A_b) + N_{dw}(U_{dw} + U_{drill}H_{dw}) \tag{12}$$

where N_{dw} is the number of dry wells in the basin (one), U_{dw} is the unit cost of each dry well ($), U_{drill} is the unit cost of drilling the dry well ($/ft), and H_{dw} is the depth of the dry well (ft).

Phase 3 Bioretention and Variable Number of Dry Wells

The objective function is the same as Phase 2, except that N_{dw}, the number of dry wells, is an integer variable that can be zero, one, or multiple up to the limit allowed to prevent hydraulic interference.

4.2 Infiltration

The Green-Ampt infiltration model used by Stafford et al. (2015) (Equations 5 and 6) was modified to take into account ponding depth. The ponding depth changes as a function of time, and is also a function of the critical duration of the design storm event. For the purposes of this model, the average ponding depth during the critical event was used:

$$h_o = (12d)/2 \tag{13}$$

where h_o is the average ponding depth (in) and d is the maximum depth of ponding in the basin during the critical storm event (ft). The implicit solution to the cumulative infiltration at the critical duration is then given by the following expression:

$$F_t = K\left(\frac{T_m}{60}\right) + (\psi + h_o)(\eta - \theta_i)\ln\left(1 + \frac{F_t}{(\psi + h_o)(\eta - \theta_i)}\right) \tag{14}$$

where F_t is the cumulative infiltration at the critical duration (in), K is the saturated hydraulic conductivity of the soil (in/hr), T_m is the critical (maximum) duration (min), ψ is the wetting front soil suction head (in), h_o is the average ponding depth (in), η is the

soil porosity, and θ_i is the initial soil moisture content. The infiltration rate at the critical

storm duration is given below:

$$f_m = K\left(\frac{(\psi + h_o)(\eta - \theta_i)}{F_t} + 1\right) \tag{15}$$

where f_m is the infiltration rate at the critical storm duration (in/hr).

4.3 Evapotranspiration

Evapotranspiration (ET) is one of the key processes in green infrastructure and

bioretention, but scarce research exists on ET from bioretention basins. ET is not

included in most conventional urban hydrological models. The belief that urban ET can

be treated as an abstraction has been referred to as a "gross error" (Denich and Bradford,

2010). One study performed in Pennsylvania using weighing lysimeters found that

between 14% and 35% of the precipitation that entered the bioretention basin was lost

due to evapotranspiration (Hickman Jr, 2011). ET has been included in the formulation of

this model.

Two approaches for the estimation of ET rate are included in this model, each

with different data requirements. The two methods in the model are the Penman-Monteith

method and the Hargreaves-Samani method.

4.3.1 Penman-Monteith

The Penman-Monteith equation has been described as the "Cadillac" of

evapotranspiration models. It offers relatively accurate results, though it is data intensive.

In the early 1970s, the Food and Agriculture Organization of the United Nations (FAO)

developed a method for estimating crop-water requirements, and since it has become a

widely accepted standard, in particular for irrigation studies (Smith et al., 1998). Since

the first FAO Irrigation and Drainage Paper (Doorenbos and Pruitt, 1977), many updates

and revisions have been made. The version used in this model is from the ASCE Task

Committee on Standardization of Reference Evapotranspiration (Walter et al., 2001). The

first step in using the Penman-Monteith equation is to determine an evapotranspiration

rate for a standard reference crop using site specific microclimate parameters. The

reference ET rate is calculated using the equation below:

$$ET_0 = \frac{0.408\Delta(R_n - G) + \gamma\frac{C_n}{T+273}u_2(e_s - e_a)}{\Delta + \gamma(1 + C_d u_2)} \tag{16}$$

where ET_0 is the standardized reference crop evapotranspiration rate (mm/hr), Δ is the

slope vapor pressure-temperature curve (kPa $°C^{-1}$), γ is a psychometric constant

(kPa $°C^{-1}$), R_n is the calculated net radiation at the crop surface (MJ $m^{-2}hr^{-1}$), u_2 is the

mean hourly wind speed at a 2-m height (m/s), e_s is the saturation vapor pressure (kPa),

e_a is the mean actual vapor pressure (kPa), G is the soil heat flux density at the soil

surface (MJ $m^{-2}hr^{-1}$), T is the mean ambient air temperature (°C), C_n is the numerator

constant (K mm $s^3 Mg^{-1}hr^{-1}$), and Cd is the denominator constant (s/m).

The reference ET rate is then multiplied by a crop coefficient factor to calculate

the estimated ET rate for a specific crop:

$$ET_c = K_c(ET_o) \tag{17}$$

where ET_c is the evapotranspiration rate for a specific crop (mm/hr), and K_C is a

dimensionless crop coefficient. For more information on the ASCE Standardization of

Reference Evapotranspiration Equation, see the 2005 ASCE-EWRI Task Committee

Report (Walter et al., 2001).

4.3.2 Hargreaves-Samani

The Hargreaves-Samani method is a less data intensive estimate of potential ET. The original Hargreaves-Samani equation was developed in the early 1980's when simplified ET equations were required in irrigation water requirement computations in California and other semiarid regions (Hargreaves and Samani, 1982). The only data requirements are temperature and latitude. While simple, the method has seen substantial use due to its efficiency and relative reliability. Temperature-radiation potential evapotranspiration approaches such as Hargreaves-Samani actually tend to provide better results for rainfall-runoff models than the Penman approach (Oudin et al., 2005). Relative humidity is not explicitly contained in the equation, but it is implicitly represented by the difference in maximum and minimum temperature. The Hargreaves-Samani equation contained in the model is shown below:

$$\text{ET}_o = 0.0135(\text{KT})(\text{R}_a)(\text{T}_{max} - \text{T}_{min})^{\frac{1}{2}}(\text{T} + 17.8) \tag{18}$$

where KT is an empirical coefficient, R_a is the extraterrestrial radiation (mm/day), T_{max} and T_{min} are the maximum and minimum daily temperature (°C), and T is the average daily temperature (°C). It is recommended to use KT = 0.162 for "interior" regions and KT = 0.19 for coastal regions (Samani, 2000). The reference crop ET rate is then multiplied by a dimensionless crop coefficient K_C (Equation 17).

4.4 Dry Well Hydraulics

There has been little research done on the hydraulics of drywells. Dry wells are located entirely in the vadose zone, resulting in flow from dry wells occurring entirely as

unsaturated flow. Conventional well pumping equations are not valid for unsaturated conditions.

Two-dimensional finite difference methods have been used in a few instances to model the drainage of dry wells (Bandeen, 1987; Massmann, 2004). Bandeen (1987) used the saturated-unsaturated flow model UNSAT to perform three case study simulations of dry well drainage in Tucson, AZ. Massmann (2004) modeled flow from a dry well under transient conditions using the finite difference model VS2DH 3.0. Both of these are two-dimensional models that utilize finite difference techniques to solve Richard's equation, the governing equation for unsaturated conditions.

In a report prepared for the Washington State Department of Transportation (WSDOT), an approach for estimating the steady state infiltration rate from dry wells is presented (Massmann, 2004). The infiltration rate from a dry well is initially high, then decreases as the underlying soil becomes saturated and the hydraulic gradient decreases. Massmann (2004) used the two-dimensional, saturated-unsaturated, finite-difference model VS2DH 3.0 to simulate the transient radial flow system from a dry well. The decrease in infiltration rate versus time can be seen in the simulation run, presented in Figure 15. The dry well modeled is the pre-cast concrete type used by WSDOT shown in Figure 8. The "double-barrel" configuration referenced is a dry well in which two concrete sections are used vertically, resulting in a dry well with a depth of 12 feet.

As can be seen in Figure 15, the infiltration rate over time approaches steady state. To estimate this steady state infiltration rate, Bouwer (2002) and Massmann (2004) advocate the use of an equation developed for reverse auger-hole flow. This approach utilizes a relationship originally developed by the U.S. Bureau of Reclamation (USBR)

for use with the well permeameter method field permeability test (USBR 7300-89) (U.S. Department of the Interior, 1990). This relationship is based on an analytical solution for "Flow from a Test-Hole Located above Groundwater Level" developed by R.E. Glover (Zanger, 1953).

The analytical solution describes the fluxes out of the well by Darcy relationships based on the assumptions of steady-state flow, homogeneous, isotropic, rigid porous medium, and a semi-infinite, field saturated flow domain (Reynolds et al., 1983). The relationship developed by Glover assumes that gravity flow is negligibly small compared to pressure flow.

The analytical relationship recommended by Bouwer (2002) and Massmann (2004) for estimating the dry well flow rate is included in the model and shown below:

$$Q_{dw} = (2\pi K_{dw} H_{dw}{}^2)/C_{dw} \tag{19}$$

$$C_{dw} = \sinh^{-1}\left(\frac{H_{dw}}{a}\right) - \sqrt{(\frac{a}{H_{dw}})^2 + 1 + \frac{a}{H_{dw}}} \tag{20}$$

where Q_{dw} is the flowrate from the drywell (cfs), K_{dw} is the average field saturated hydraulic conductivity over the depth of the well (ft/s), H_{dw} is the depth of the drywell (ft), a is the radius of the dry well (ft), and C_{dw} is a coefficient.

Massmann (2004) used this equation to estimate the infiltration rate for dry wells with various hydraulic conductivity rates and groundwater depths. The infiltration rates calculated ranged from more than 5 cfs to less than 0.1 cfs. The results show that if the depth to the water table is fixed, the infiltration rate into the dry well is linearly proportional to the hydraulic conductivity value (Massmann, 2004).
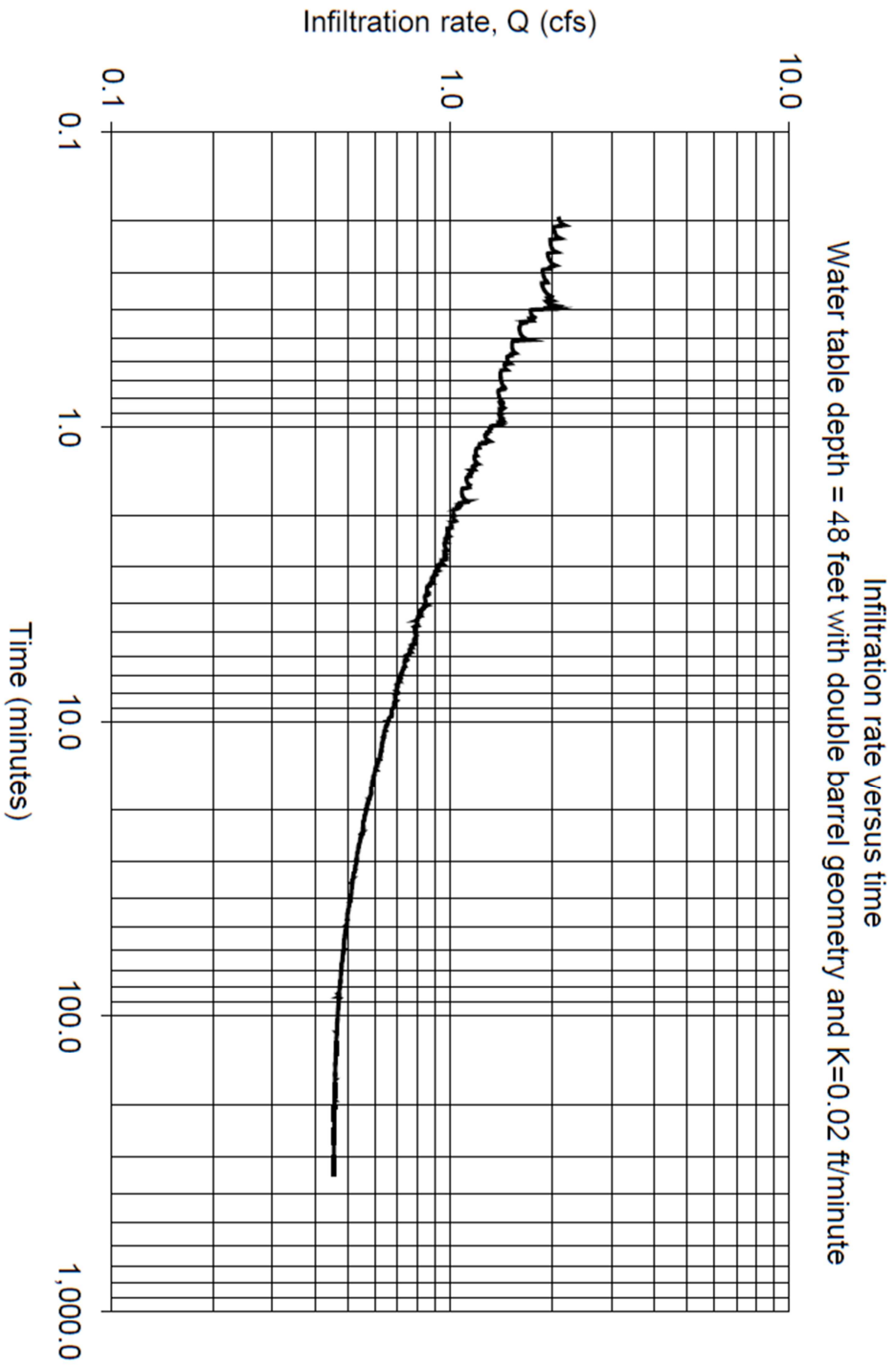
Figure 15. Infiltration Rate Versus Time for a Typical Dry Well Used by WSDOT. Source: (Massmann, 2004)

The infiltration rates estimated using the analytical equation were compared to dry well flow rates observed in the field. The comparison of the estimated values and the observed flow rates are shown in Table 3 and Figure 16. Massmann (2004) hypothesized that the lack of proportionality in the results is due to the hydraulic conductivity values estimated from the grain size curves for the dry wells under-estimating the effective hydraulic conductivity values, resulting in conservative estimates for the infiltration rates.

In designs with multiple dry wells, the hydraulic interaction between the dry wells should be taken into account, as closely spaced dry wells could affect their respective infiltration rates. In the case of multiple conventional wells pumping from the phreatic zone, the cone of depression from one will affect the pumping capacity of a nearby well. Because dry wells are located entirely in the vadose zone, the analytical relationships for multiple conventional wells do not apply.

Massmann (2004) developed recommendations for the spacing of dry wells based upon the results of simulations using the unsaturated flow model VS2DH 3.0. For sites with a water table deeper than 30 feet, the recommended spacing to prevent overlap of groundwater mounds is five times the radius of the excavation for the dry well, or approximately 50 feet for the design used by WSDOT. In general, sites with lower hydraulic conductivity values and sites with shallower groundwater tables required greater spacing (Massmann, 2004). ADEQ recommends a minimum dry well spacing of 100 feet, center to center (Arizona Department of Environmental Quality, 2009).

Table 3. Comparison of Estimated and Observed Dry Well Flow Rates in Washington State. Source: (Massmann, 2004)

Site	Hydraulic conductivity estimates				Dry well flow rates (cfs)		
	Grain Size	Test Pits	Bore hole	Geometric mean	Observed	USBR Equation	Relative Error
NW Tech Park	4	1		8.3E-04	0.568	0.08	86%
Hayford Plaza	4	1		5.9E-03	0.62	1.84	-197%
Shady Slope	3	2		1.3E-03	0.81	0.16	80%
Trickle Creek	1			1.6E-05	0.086	0.01	93%
Summer Crest	2			8.9E-05	0.52	0.04	93%
Midway A	1			1.1E-04	0.03	0.03	-14%
Midway B	1			1.1E-03	0.51	0.96	-87%
Mt. Spokane 1			1	4.6E-04	1.32	0.20	85%
Mt. Spokane 3			1	3.6E-04	1.17	0.15	87%
Westwood N. DW-2	1			2.0E-03	1.5	1.48	2%
Westwood N. DW-3	1			2.9E-03	1.42	1.75	-23%
Westwood N. DW-6	1			1.9E-03	1.11	0.12	89%
Westwood N. DW-7	1			6.1E-04	1.44	1.12	22%
Westwood N. DW-8	1			2.3E-03	0.9	0.92	-3%
Westwood N. DW-9	1			1.3E-04	0.62	0.04	94%
Westwood N. DW-10	1			4.2E-03	0.38	0.76	-100%
Westwood N. DW-12	1			3.4E-04	0.95	0.29	69%
Westwood N. DW-14	1			6.1E-04	0.79	0.54	32%
Westwood N. DW-15	1			6.1E-04	0.74	0.54	27%
Westwood N. DW-20	1			3.8E-05	0.87	0.03	96%
5 Mile Prairie	1			2.3E-03	1.31	1.93	-47%
Dartford	1			6.9E-04	0.28	0.66	-136%
Dartford	1			1.9E-04	0.26	0.07	73%
5 Mile Prairie	1			1.4E-03	0.22	0.84	-283%
5 Mile Prairie	1			4.6E-04	0.27	0.33	-23%
5 Mile Prairie			1	2.3E-05	0.29	0.02	92%
5 Mile Prairie			1	2.3E-04	0.58	0.25	57%
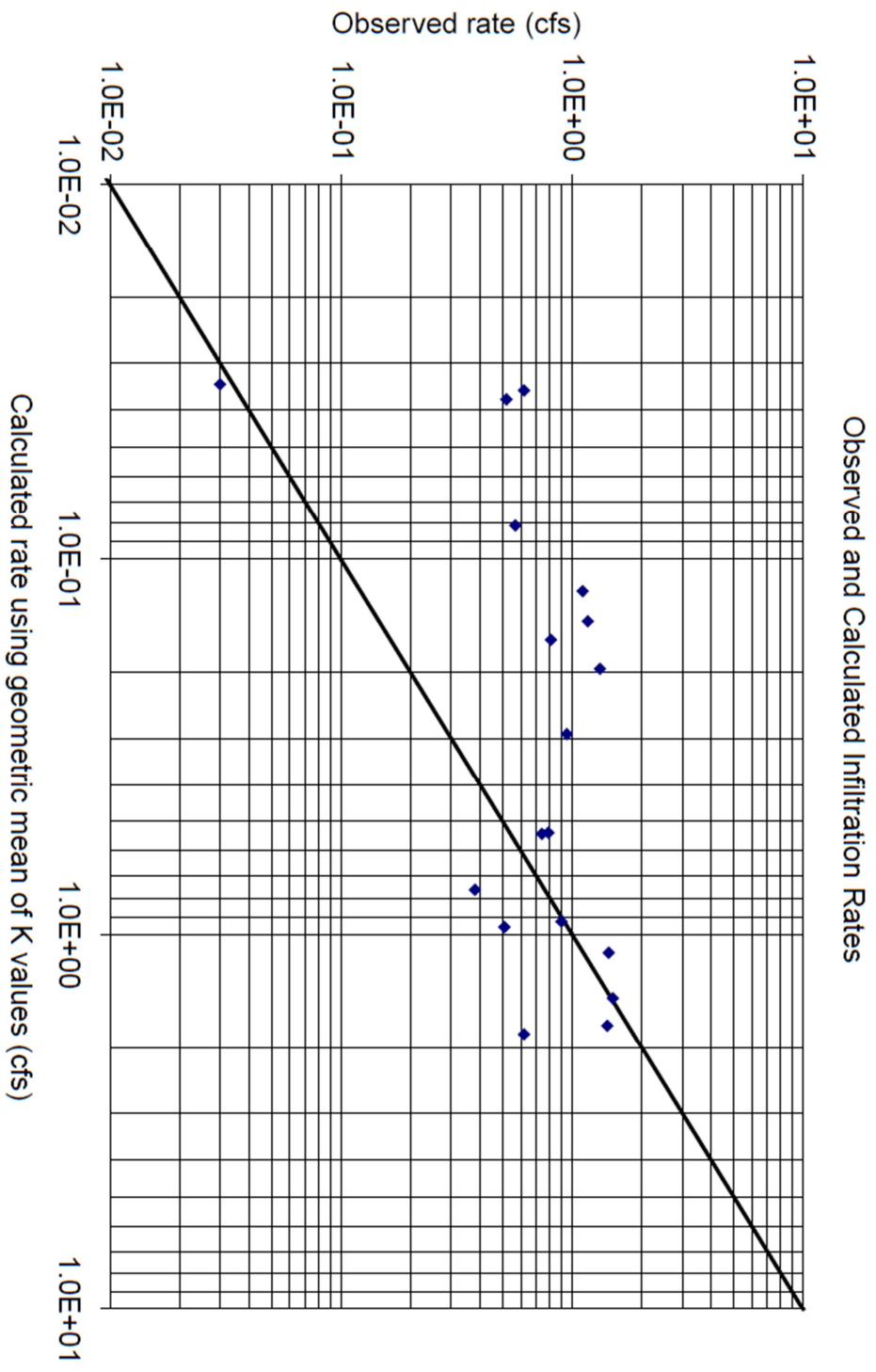
Figure 16. Observed and Calculated Infiltration Rates for Dry Wells in Washington State. Source: (Massmann, 2004)

The hydraulic interaction of dry wells is avoided in the MINLP model by defining an upper limit to the number of dry wells allowed in the basin using the area of the basin computed and a user specified minimum spacing:

$$N_{dw} \leq \frac{LW}{SP_{dw}^2} \tag{21}$$

where N_{dw} is the number of dry wells in the basin (an integer variable), L and W are the length and width of the basin (ft), and SP_{dw} is the user entered minimum dry well spacing (ft). This creates a maximum number of dry wells allowed in the basin based on the area and minimum spacing. The value of N_{dw} is allowed to be less than this maximum number.

The analytical equation used to compute the infiltration flow rate into the well does not take into account clogging of the infiltration media over the lifetime of the well. Cleaning out the settling chamber must occur regularly, because the dry well cannot be cleaned out by "backwashing", and once the infiltration media is clogged the dry well becomes useless. Bouwer (2002) noted that more research is needed on vadose-zone recharge wells to develop an optimum design for well capacity, clogging control, useful life, and minimum long-term cost of recharge per unit volume of water. Ultimately the usefulness of dry wells depends on their useful lives.

4.5 Critical Storm Duration

Phase 1 Bioretention

Using the Green-Ampt infiltration rate and the evapotranspiration rate calculated with the ASCE Penman-Monteith equation or Harman-Samani equation, the implicit solution for the critical storm duration using the modified rational method can be

developed. As in the infiltration model, the volume of storage required is given by the difference between the inflow volume and the outflow volume:

$$V_d = V_{in} - V_{out} \tag{22}$$

where V_d is the detention volume required, and V_{in} and V_{out} are the inflow volume and outflow volume. The inflow volume is defined by Equation 4 as in the infiltration model using the modified rational method. The outflow volume, however, includes ET as well as infiltration. The outflow volume at the time of duration is defined using the Green-Ampt infiltration and the ET rate defined by Penman-Monteith or Hargreaves-Samani:

$$V_{out} = \frac{A_b F\left(\frac{T_d}{60}\right)}{12} + ET_C(T_d/60) \tag{23}$$

where V_{out} is the volume infiltrated or evapotranspired (ft^3) at time T_d (min), A_b is the area of the basin (ft^2), F is the cumulative infiltration (in) as defined by Equation 14, and ET_C is the evapotranspiration rate (in/hr) defined by Equation 17 using the reference ET calculated by Equation 16 or 18. Differentiating Equation 22 for detention volume V_d with respect to the duration T_d and setting the first derivate equal to zero results in an expression for the duration time resulting in the maximum volume required. The storm duration time which forces the maximum storage volume required is called the "critical storm duration." The implicit solution for the critical storm duration is:

$$\frac{(T_m(1-n)+b)}{(T_m+b)^{(n+1)}} = \frac{LW\frac{f_m+ET_c}{(12)(3600)}}{2CA} \tag{24}$$

where n and b are regional intensity coefficients, L and W are the length and width of the basin (ft), ET_c is the evapotranspiration rate (in/hr), C is the dimensionless runoff coefficient for the watershed, and A is the area of the watershed (acres).

Phase 2 & 3 Bioretention with Dry Well(s)

To take into account the flow into a dry well in the determination of the critical storm duration, Equation 22 must be modified to include the dry well flow rate in the outflow volume along with infiltration and ET. The expression for the outflow volume at the time of duration becomes:

$$V_{out} = \frac{A_b F\left(\frac{T_d}{60}\right)}{12} + ET_C(T_d/60) + 60 N_{dw} Q_{dw}(T_d/60) \tag{25}$$

where N_{dw} is the number of dry wells in the basin (one for Phase 2, integer variable for Phase 3), and Q_{dw} is the flow rate from each dry well (cfs) determined using Equations 19 and 20. Differentiating leads to the implicit solution for the critical storm duration with dry wells:

$$\frac{(T_m(1-n)+b)}{(T_m+b)^{(n+1)}} = \frac{LW\frac{f_m+ET_c}{(12)(3600)}+N_{dw}Q_{dw}}{2CA} \tag{26}$$

4.6 Required Storage Volume and Total Drain Time

Phase 1 Bioretention

The required volume of the bioretention basin can be calculated using the critical storm duration and the difference between the inflow volume and the outflow volume (Equation 22). The expression becomes:

$$60Q_P T_m - (LW)\left(\frac{F_t}{12} + \frac{T_m ET_c}{12}\right) = LWd \tag{27}$$

where L, W, and d are, respectively, the length, width and depth of the basin (ft), and Q_P is the peak surface runoff (cfs) given by:

$$Q_P = CiA \tag{28}$$

where C is the dimensionless runoff coefficient, A is the watershed area (acres), and i is the rainfall intensity given by the IDF relationship (Equation 2).

In order to add a constraint limiting inundation time to 36 hours, the total drain time must be calculated (time from when water begins to collect in basin until basin has completely drained). To do this, the cumulative infiltration at the total drain time is calculated in addition to the total drain time. The implicit solution for the cumulative infiltration at the total drain time is given below:

$$F_{tot} = K\left(\frac{T_{tot}}{60}\right) + (\psi + h_o)(\eta - \theta_i)\ln\left(1 + \frac{F_{tot}}{(\psi + h_o)(\eta - \theta_i)}\right) \tag{29}$$

where F_{tot} is the cumulative infiltration at the total drain time (in), and T_{tot} is the total drain time (min). The solution for the total drain time is determined by equating the total runoff volume using the modified rational method to the outflow volume through infiltration and evapotranspiration. The implicit solution for the total drain time is:

$$60Q_P 0.5\big((T_m - T_c) + (T_m + T_c)\big) = LW\left(\frac{F_{tot}}{12}\right) + LW\frac{ET_c T_{tot}}{12} \tag{30}$$

Finally, the total drain time can be constrained to less than 36 hrs:

$$T_{tot} \leq 60Dr \tag{31}$$

where Dr is the total allowable drain time in hours (36 hrs).

Phase 2 & 3 Bioretention with Dry Well(s)

The required storage volume with dry wells must take into account the outflows through the dry wells. With the addition of the dry well flow rate, Equation 26 becomes:

$$60Q_P T_m - (LW)\left(\frac{F_t}{12} + \frac{T_m ET_c}{12}\right) - 60N_{dw}Q_{dw}T_m = LWd \tag{32}$$

The total drain time relationship (Equation 29) must modified as well to take into account the dry well flow rate:

$$60Q_P 0.5\big((T_m - T_c) + (T_m + T_c)\big) = LW\left(\frac{F_{tot}}{12}\right) + LW\frac{ET_cT_{tot}}{12} + 60N_{dw}Q_{dw}T_m \quad (33)$$

4.7 Mathematical Formulation

Phase 1 Bioretention

Objective:

$$\text{Minimize } Z = U_e(V_b) + \big(U_tS_t^{-2} + U_{sh}S_{sh}^{-2} + U_gS_g^{-2}\big)A_v + U_l(A_b) \quad (11)$$

Decision Variables: L, W, and d, the length, width, and depth of the basin (ft).

Subject to:

(a) Relationship defining basin area and volume based on size, shape and depth. Basin is defined as square, and a maximum basin depth is defined, though the user can define any basin shape required. The square constraint is entered simply as: $L = W$ (34)

(b) IDF relationship for the design location:

$$i = \frac{a}{(T_d + b)^n} \quad (2)$$

(c) Time of concentration:

$$t_c = \frac{100L^{0.8}\left[\left(\frac{1000}{CN}\right) - 9\right]^{0.7}}{19000^{\,0.5}} \quad (3)$$

(d) Cumulative infiltration volume as a function of time:

$$F_t = K\left(\frac{T_m}{60}\right) + (\psi + h_o)(\eta - \theta_i)\ln\left(1 + \frac{F_t}{(\psi + h_o)(\eta - \theta_i)}\right) \quad (14)$$

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(e) Infiltration rate as a function of time:

$$f_m = K\left(\frac{(\psi+h_o)(\eta-\theta_i)}{F_t} + 1\right) \tag{15}$$

(f) Evapotranspiration rate with reference crop ET calculated using the Penman-Monteith method or the Hargreaves-Samani method:

$$ET_c = K_c(ET_o) \tag{17}$$

Penman-Monteith method:

$$ET_o = \frac{0.408\Delta(R_n-G)+\gamma\frac{C_n}{T+273}u_2(e_s-e_a)}{\Delta+\gamma(1+C_d u_2)} \tag{16}$$

Hargreaves-Samani method:

$$ET_o = 0.0135(KT)(R_a)(T_{max} - T_{min})^{\frac{1}{2}}(T + 17.8) \tag{18}$$

(g) Implicit solution to determine the critical storm duration to maximize the detention volume based on infiltration and evapotranspiration outflows:

$$\frac{(T_m(1-n)+b)}{(T_m+b)^{(n+1)}} = \frac{LW\frac{f_m+E_c}{(12)(3600)}}{2CA} \tag{24}$$

(h) Volume of storage required to store the storm runoff not infiltrated or evapotranspired at the critical detention time:

$$60Q_P T_m - (LW)\left(\frac{F_t}{12} + \frac{T_m ET_c}{12}\right) = LWd \tag{27}$$

(i) Implicit solution to Green-Ampt infiltration for total infiltration volume:

$$F_{tot} = K\left(\frac{T_{tot}}{60}\right) + (\psi + h_o)(\eta - \theta_i)\ln(1 + \frac{F_{tot}}{(\psi+h_o)(\eta-\theta_i)}) \tag{29}$$

(j) Calculation of total drain time:

$$60Q_P 0.5\big((T_m - T_c) + (T_m + T_c)\big) = LW\left(\frac{F_{tot}}{12}\right) + LW\frac{ET_c T_{tot}}{12} \tag{30}$$

(k) Total basin drain time must be less than 36 hours:

$$T_{tot} \leq 60 Dr \tag{31}$$

Solve model using GAMS NLP solver

Phase 2 Bioretention and Single Dry Well

Objective:

$$\text{Min } Z = U_e(V_b) + (U_t S_t^{-2} + U_{sh} S_{sh}^{-2} + U_g S_g^{-2})A_v + U_l(A_b) + N_{dw}(U_{dw} + U_{drill} H_{dw}) \tag{12}$$

Decision Variables: L, W, d, and H_{dw}, the depth of the dry well (ft).

Subject to the same constraints as above with the following additions / revisions:

(l) Dry well flow rate:

$$Q_{dw} = (2\pi K_{dw} H_{dw}^2)/C_{dw} \tag{19}$$

$$C_{dw} = \sinh^{-1}\left(\frac{H_{dw}}{a}\right) - \sqrt{(\frac{a}{H_{dw}})^2 + 1} + \frac{a}{H_{dw}} \tag{20}$$

(m) Maximum depth of dry well: 10ft above aquifer depth

- Constraint (g) changed to determine critical storm duration with outflows including infiltration, evapotranspiration, and dry wells:

$$\frac{(T_m(1-n)+b)}{(T_m+b)^{(n+1)}} = \frac{LW\frac{f_m+ET_c}{(12)(3600)}+N_{dw}Q_{dw}}{2CA} \tag{26}$$

- Constraint (h) changed to calculate storage volume required with dry well outflow:

$$60Q_P T_m - (LW)\left(\frac{F_t}{12} + \frac{T_m ET_c}{12}\right) - 60N_{dw}Q_{dw}T_m = LWd \tag{32}$$

- Constraint (j) changed to calculation total drain time with dry well outflow:

$$60Q_P 0.5\left((T_m - T_c) + (T_m + T_c)\right) = LW\left(\frac{F_{tot}}{12}\right) + LW\frac{ET_c T_{tot}}{12} + 60N_{dw}Q_{dw}T_m \tag{33}$$

The number of dry wells set to 1. Solve model using GAMS NLP solver.

Phase 3 Bioretention and Variable Number of Dry Wells

Objective:

$$\text{Min } Z = U_e(V_b) + \left(U_t S_t^{-2} + U_{sh} S_{sh}^{-2} + U_g S_g^{-2}\right)A_v + U_l(A_b) + N_{dw}(U_{dw} + U_{drill}H_{dw}) \quad (12)$$

Decision Variables: L,W, d, H_{dw}, and N_{dw}, the number of dry wells as an integer variable.

Subject to the same constraints as Phase 2 with the following addition:

(n) Upper limit to integer variable N_{dw} defined by the dry well minimum spacing

constraint:

$$N_{dw} \leq \frac{LW}{SP_{dw}^2} \quad (21)$$

Solve model using GAMS MINLP solver.

4.8 Methodology for Solving Optimization Models

The NLP problems and MINLP problem are solved using the General Algebraic Modeling System (GAMS). The GAMS home page describes the system as follows: "The General Algebraic Modeling System (GAMS) is a high-level modeling system for mathematical programming and optimization. It consists of a language compiler and a stable of integrated high-performance solvers. GAMS is tailored for complex, large scale modeling applications, and allows you to build large maintainable models that can be adapted quickly to new situations" (https://www.gams.com/).

Initial research and development of GAMS was funded by The World Bank, through the Bank's Research Committee and carried out at the Development Research Center in Washington DC (Bussieck and Meeraus, 2004). Development of the system

took place in a close cooperation of mathematical economists. The most important success factor in the development of the system was the synergy between economics, computer science and operations research (Bussieck and Meeraus, 2004).

The following general principles were used in the design of the system (Rosenthal, 2012):

- All existing algorithmic methods should be available without changing the user's model representation. This includes linear, nonlinear, mixed integer, and mixed integer nonlinear optimizations.

- The optimization problem should be expressible independently of the data it uses.

- The use of a relational data model requires that the allocation of computed resources be automated.

The ability to change algorithmic methods without changing the user's model representation was a primary reason GAMS was chosen to solve the optimization problems developed here. Major advantages of using GAMS summarized by Chattopadhyay (1999) include:

- The coding is easy, fast and compact

- The amount of data can be increased without changing the variables, equations, model, and solve statements.

- Choice of a variety of solving algorithms without altering the rest of the model

The NLP problems developed here were expanded to a MINLP model with minimal changes to the model representation. GAMS allows for relatively easy model modification and expansion, a variety of optimization algorithms that can be changed seamlessly, and clear, concise documentation. The GAMS codes for all three models developed are shown in the Appendices.

5. MODEL APPLICATION AND RESULTS

5.1 Example Application 1

In order to test the applicability of the models developed, they were applied to an example development located on the Arizona State University (ASU) campus. The College Avenue Commons (CAVC) building and property is a recent development that includes a stormwater bioretention facility. All three phases of the optimization model as well as the original infiltration basin model are run for the development area. Phoenix Sky Harbor airport (PHX) is located nearby which has a meteorological station with the data required for the Penman-Monteith ET method. An aerial photograph and the relative location of the site to PHX are shown in Figure 17.

5.1.1 Infiltration Only

The original infiltration model developed by Stafford et al. (2015) is run for the example development. The optimization model sizes the basin based on outflow through infiltration only, without evapotranspiration or dry wells, and without a total drain time constraint.

Development area (watershed area) was estimated using Google Earth. The dimensionless runoff coefficient C for the watershed was estimated as 0.97 based on the area of concrete and roofing from the satellite imagery. The 100 year event IDF parameters a, b, and n were determined for the location using data from NOAA Atlas 14 Point Precipitation Frequency Estimates (hdsc.nws.noaa.gov).
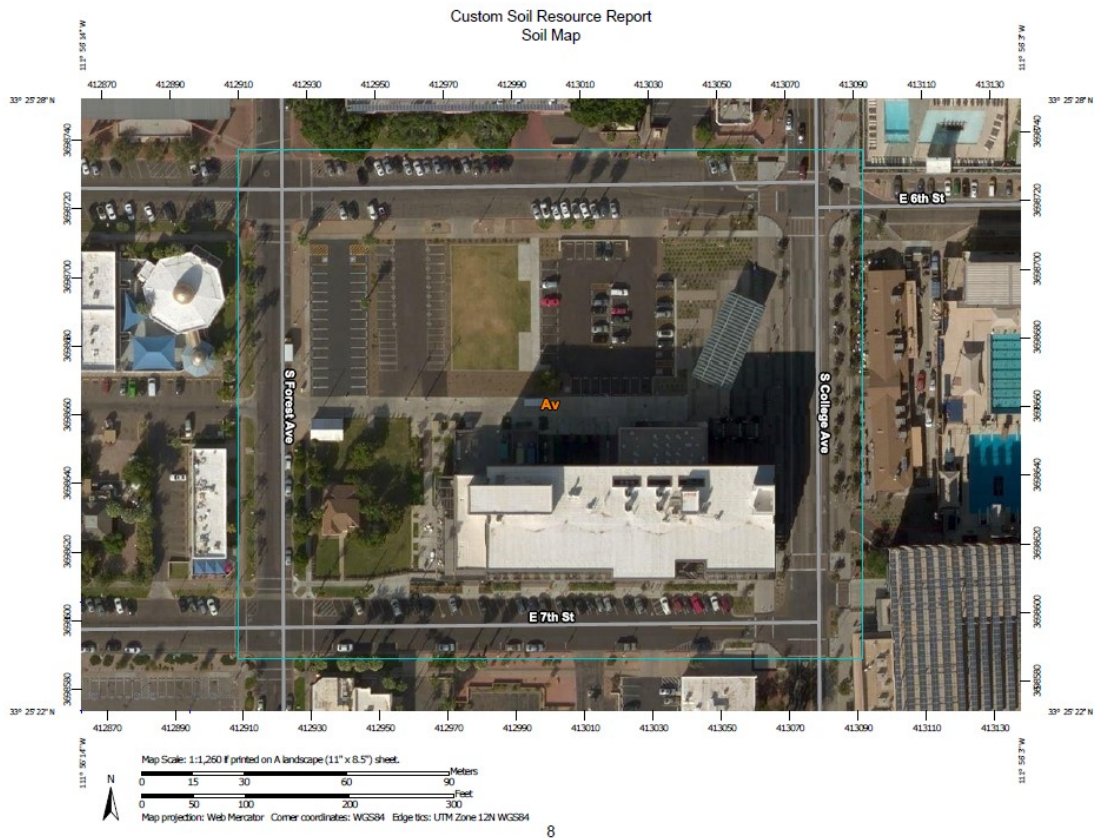
Figure 17. Above: Aerial Photo of the Example Development. Source: USDA Soil Survey Report. Below: Location of CAVC in relation to PHX. Source: Google Earth
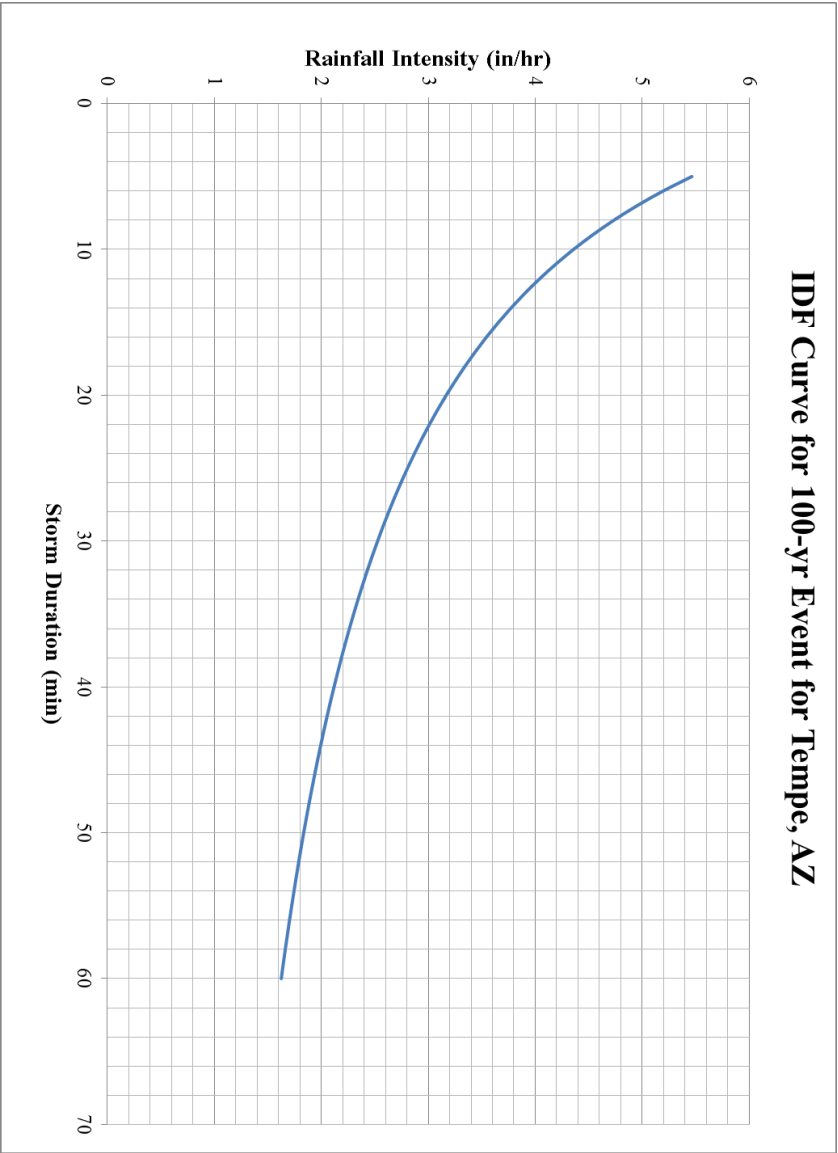
Figure 18. IDF Curve for Example Site. Precipitation data from NOAA Atlas 14 Site 02-8499

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The resulting IDF curve is shown in Figure 18. To determine soil parameters, a custom USDA soil survey report was downloaded for the development area (websoilsurvey.sc.egov.usda.gov). According to the report, the soil unit in the development is 100% Avondale clay loam. The hydrology design manual for Maricopa County gives values of hydraulic conductivity and wetting front suction head for use in Green-Ampt for various soil types in the area (Flood Control District of Maricopa County, 2013). Soil porosity values for different soil types are given in "Ground and Surface Water Hydrology" (Mays, 2011). The initial soil moisture content was assumed to be 0.3. The soil parameters for clay loam and soil types are shown in Table 12. A summary of the data input into the model is shown in Table 4.

The model was run with the site parameters estimated. The basin area is constrained to be a square, so only one side length is reported. The Stafford et al. (2015) model does not include a drain time constraint. The results of the model run are shown in Table 5. Without a drain time constraint, the total drain time computed is 805 hrs (over 33 days). Clay loam is a relatively impermeable soil, resulting in slow infiltration. With the total drain time constraint of 36 hours added, the model results are infeasible. A higher infiltration rate or additional outflows are necessary to meet the 36 hours drain time requirement. One reason that dry wells have been used so extensively in Maricopa County is to meet the drain time requirement.

Table 4. Input Data for Infiltration Optimization Model

	Variable and Unit	Value
U_E	Unit Cost of Excavation ($/cy)	20
U_E	Unit Cost of Land ($/sf)	0.18
U_E	Unit Cost of Seeding ($/acre)	30,000
C	Dimensionless Runoff Coefficient	0.97
A	Watershed Area (acres)	3.6
a	IDF parameter a	45.92
b	IDF parameter b	10
n	IDF parameter n	0.786
	Soil Type	Clay Loam
L	Length of Overland Flow (ft)	400
CN	Weighted Watershed Curve Number	85
S	Average Surface Slope (%)	2.0

Table 5. Infiltration Model Results for CAVC Development

Cost	Basin Depth (ft)	Side Length (ft)	Storage Volume (ft^3)
$56,852	3.0	97.3	28,425

Rational Method Intensity (in/hr)	Critical Infiltration Rate (in/hr)	Peak Inflow Rate (cfs)	Total Drain Time (hrs)
0.7	0.12	2.35	805

Critical Storm Duration (min)	Total Storm Runoff (ft^3)	Infiltration Outflow (ft^3)
206	28,980	28,980 (100%)

5.1.2 Infiltration and Evapotranspiration

The Phase 1 (bioretention) design optimization model was run for the example development using both the Hargreaves-Samani method and Penman-Monteith method for estimating evapotranspiration. The additional costs of vegetation used in the objective function are shown in Table 6.

The Hargreaves-Samani method for determining evapotranspiration requires temperature and solar radiation data. Average daily minimum and maximum temperature data for Tempe, AZ for the month of May was taken from U.S. Climate Data (usclimatedata.com). Extraterrestrial radiation is a function of latitude and day of the year. Calculation of extraterrestrial radiation was done using a calculator developed by Santa Clara University (www.engr.scu.edu/~emaurer/tools/calc_solar_cgi.pl) based on the equation developed by Duffie and Beckman (1980). Radiation data was retrieved for May 15 for the latitude of the site, 33.423819°. A summary of the data input and the reference crop evapotranspiration computed by the model using Equation 18 is shown in Table 7.

The additional meteorological data required for the Penman-Monteith method was obtained from the weather station at Phoenix Sky Harbor International Airport (PHX). Wind speed, dew point, and pressure data were taken from the station at PHX (http://w1.weather.gov/obhistory/KPHX.html), located 3.5 miles to the west of the CAVC (see Figure 17). Parameters for input into the model were computed using this data and methods recommended by ASCE (Walter et al., 2001).

Table 6. Input Vegetation Costs

	Variable and Unit	Value
U_t	Unit Cost of Trees ($/tree)	200
U_{sh}	Unit Cost of Shrub ($/shrub)	50
U_g	Unit Cost of Grasses ($/grass)	10
S_t	Spacing of Trees (ft)	20
S_{sh}	Spacing of Shrubs (ft)	10
S_g	Spacing of Grasses (ft)	5

Table 7. Hargreaves-Samani Method Input Data, and Computed ET

	Variable and Unit	Value
T	Average Daily Temperature (°C)	22.8
T_{max}	Maximum Daily Temperature (°C)	28.3
T_{min}	Minimum Daily Temperature (°C)	17.2
R_a	Extraterrestrial Radiation (mm/day)	16.35
ET_o	Reference Crop Evapotranspiration (in/hr)	**0.008**

The following equations are based on the ASCE Task Committee on Standardization of Reference Evapotranspiration. The psychrometric constant (γ) was determined using mean pressure (P) data at PHX using the following:

$$\gamma = 0.000665P \tag{35}$$

The slope of the saturation vapor pressure-temperature curve Δ ($kPa°C^{-1}$) was computed as:

$$\Delta = \frac{2503\exp\left(\frac{17.27T}{T+237.3}\right)}{(T+237.3)^2} \tag{36}$$

where T is the daily mean air temperature (°C). Saturation vapor pressure e_s (kPa) was computed as:

$$e_s = \frac{0.6108\exp\left(\frac{17.27T_{max}}{T_{max}+237.3}\right)+0.6108\exp\left(\frac{17.27\ min}{T_{min}+237.3}\right)}{2} \tag{37}$$

where T_{max} and T_{min} are the maximum and minimum daily temperatures in °C. Actual vapor pressure e_a (kPa) was computed using the measured dew point temperature:

$$e_a = 0.6108\exp\left[\frac{17.27T_{dew}}{T_{dew}+2\quad.3}\right] \tag{38}$$

where T_{dew} is the measured dew point temperature (°C) taken at Sky Harbor (7:51, 3/18/16). It is recommended the dew point temperature is determined by an early morning (0700 or 0800) measurement (Walter et al., 2001).

Net radiation R_n ($MJm^{-2}d^{-1}$) is determined using short and long wave radiation:

$$R_n = R_{ns} - R_{nl} \tag{39}$$

where R_{ns} is net short-wave radiation and R_{nl} is net outgoing long-wave radiation, both in $MJm^{-2}d^{-1}$.

Net outgoing long-wave radiation is computed as:

$$R_{nl} = \sigma f_{cd}\left(0.34 - 0.14\sqrt{e_a}\right)\left[\frac{T_{max}^4 + T_{min}^4}{2}\right] \tag{40}$$

where σ is the Stefan-Boltsmann constant (4.901×10^{-9} MJ $K^{-4}m^{-2}d^{-1}$), f_{cd} is the cloudiness function (1 for clear skies), e_a is the actual vapor pressure (kPa), and T_{max} and T_{min} are the maximum and minimum absolute daily temperature (K). Net short-wave radiation was computed using the equation for clear-sky solar radiation:

$$R_{so} = (0.75 + 2 \times 10^{-5}z)R_a \tag{41}$$

where R_{so} is the clear-sky solar radiation ($MJm^{-2}d^{-1}$), z is the station elevation above sea level (m), and R_a is the extraterrestrial radiation ($MJm^{-2}d^{-1}$). The magnitude of the daily soil heat flux density (G) is relatively small in comparison with net radiation, thus it is set to zero. A summary of the meteorological data, calculated input parameters, and the reference crop evapotranspiration computed by the model using Equation 16 is given in Table 8.

Many of the accepted values that have been determined for the crop coefficient Kc are for agricultural crops such as alfalfa, wheat, and corn. Table 9 presents values of Kc suggested for use in estimating ET from bioretention facilities (Pitt et al., 2008). The values presented in the table were not determined specifically for arid regions. Plants adapted to arid and semi-arid regions may respond differently to stormwater inflows. A study conducted in Tucson, AZ found that mesquites (normally considered xeric trees) use more water than oaks (normally considered mesic trees) under non-limiting conditions (Levitt et al., 1995).

Table 8. Penman Monteith Method Data, Input Parameters and Computed ET

	Variable and Unit	Value
P	Mean Atmospheric Pressure (kPa)	101
γ	Psychrometric Constant (kPa/°C)	0.0672
T	Daily Mean Air Temperature (°C)	22.8
Δ	Slope of Vapor Pressure-Temp Curve (kPa/°C)	0.1681
u_2	Mean Hourly Wind Speed (m/s)	0
T_{max}	Maximum Daily Temperature (°C)	28.3
T_{min}	Minimum Daily Temperature (°C)	17.2
T_{dew}	Dew Point Temperature (°C)	-3.9
e_s	Saturation Vapor Pressure (kPa)	2.90
e_a	Actual Vapor Pressure (kPa)	0.46
R_n	Net Radiation ($MJm^{-2}d^{-1}$)	0.90
G	Soil Heat Flux Density ($MJm^{-2}d^{-1}$)	0
ET_o	Reference Crop Evapotranspiration (in/hr)	**0.010**

Table 9. Crop Coefficients Recommended for Bioretention. Source: (Pitt et al., 2008)

Plant	Crop Coefficient (Kc)	Root Depth (ft)
Cool Season Grass (turfgrass)	0.80	1
Common Trees	0.70	3
Annuals	0.65	1
Common Shrubs	0.50	2
Warm Season Grass	0.55	1
Prairie Plants (deep rooted)	0.50	6

More research should be done to determine accurate crop coefficient values for xeric species. For Example Application 1, the crop coefficient K_c is set to one, assuming the vegetation in the basin will transpire at the rate of the reference crop. With lower crop coefficient values the model runs were infeasible with the 36 hour drain constraint.

The model was run for the CAVC development site with both evapotranspiration methods. The runs are summarized in Table 10. The reference evapotranspiration rate computed using the Penman-Monteith method and the Hargreaves-Samani method resulted in similar values, and similar model results. The volume of evapotranspiration was over 80% of the total inflow for both methods. The high ET volumes are due to the low infiltration rate, and the high crop coefficient value. When the model was run with lower crop coefficient values, the results were infeasible. The model is only able to achieve the 36 hour drain time requirement in such low permeability soil with high ET rates.

5.1.3 Infiltration, Evapotranspiration, and One Dry Well

The costs for the dry well are added into the objective function. Costs of the dry well include unit costs per dry well and unit costs per foot depth of the dry well. Unit costs per dry well include the cost of the settling chamber, mobilization, man-hole cover, etc. Unit costs per foot depth of dry well include the cost of drilling, gravel-pack, perforated PVC, drill bits, etc. A message was left for Torrent Resources, the principal installer of dry wells in Maricopa County, in an attempt to get more accurate cost information, but no contact was made.

Table 10. Infiltration and ET Model Results

ET Model	Cost	Basin Depth (ft)	Side Length (ft)	Storage Volume (ft^3)
Penman Monteith	$56,806	2.1	101	21,090
Hargreaves-Samani	$61,465	1.7	109	19,960

ET Model	Rational Method Intensity (in/hr)	ET Rate (in/hr)	Critical Infiltration Rate (in/hr)	Peak Inflow Rate (cfs)
Penman Monteith	1.3	0.010	0.26	4.7
Hargreaves-Samani	1.5	0.008	0.26	5.4

ET Model	Critical Storm Duration (min)	Total Storm Runoff (ft^3)	ET Outflow (ft^3)	Infiltration Outflow (ft^3)
Penman Monteith	79	22,312	18,809 (84%)	3,503 (16%)
Hargreaves-Samani	65	21,015	17,067 (81%)	3,948 (19%)

Additional information required for the determination of the dry well flow rate include the radius of the dry well, depth from the ground surface to the aquifer, and the average field saturated hydraulic conductivity over the depth of the dry well. Four foot diameter dry wells are standard in Maricopa County. Average groundwater depth for the CAVC development was obtained from Arizona Department of Water Resources (ADWR). ADWR maintains a well registry with groundwater well information including water level data, searchable by location (https://gisweb.azwater.gov/WellRegistry). The average groundwater depth was computed from four wells selected near the example development, see Figure 19.

Average field saturated hydraulic conductivity of the sub-surface is needed for use in calculating the dry well. Hydraulic conductivity values vary greatly with soil type, from 4.6 in/hr for sand to 0.01 in/hr for clay (Rawls et al., 1983). Observation of sub-surface soils at depth requires a geotechnical investigation, usually involving a drill rig. Geotechnical investigations are standard for new developments, but not possible for this study. Even with a proper geotechnical investigation, an accurate hydraulic conductivity value is not guaranteed. Bouwer (2002) wrote: "The proper value for K is difficult to assess, because the wetted zone is not always saturated and the streamlines have horizontal and vertical components, which complicates matters for anisotropic soils." For Example Application 1, the hydraulic conductivity value at depth was assumed to be the same as the surface hydraulic conductivity. The hydrology design manual for Maricopa County assigns a hydraulic conductivity value of 0.04 in/hr to clay loam (Flood Control District of Maricopa County, 2013).
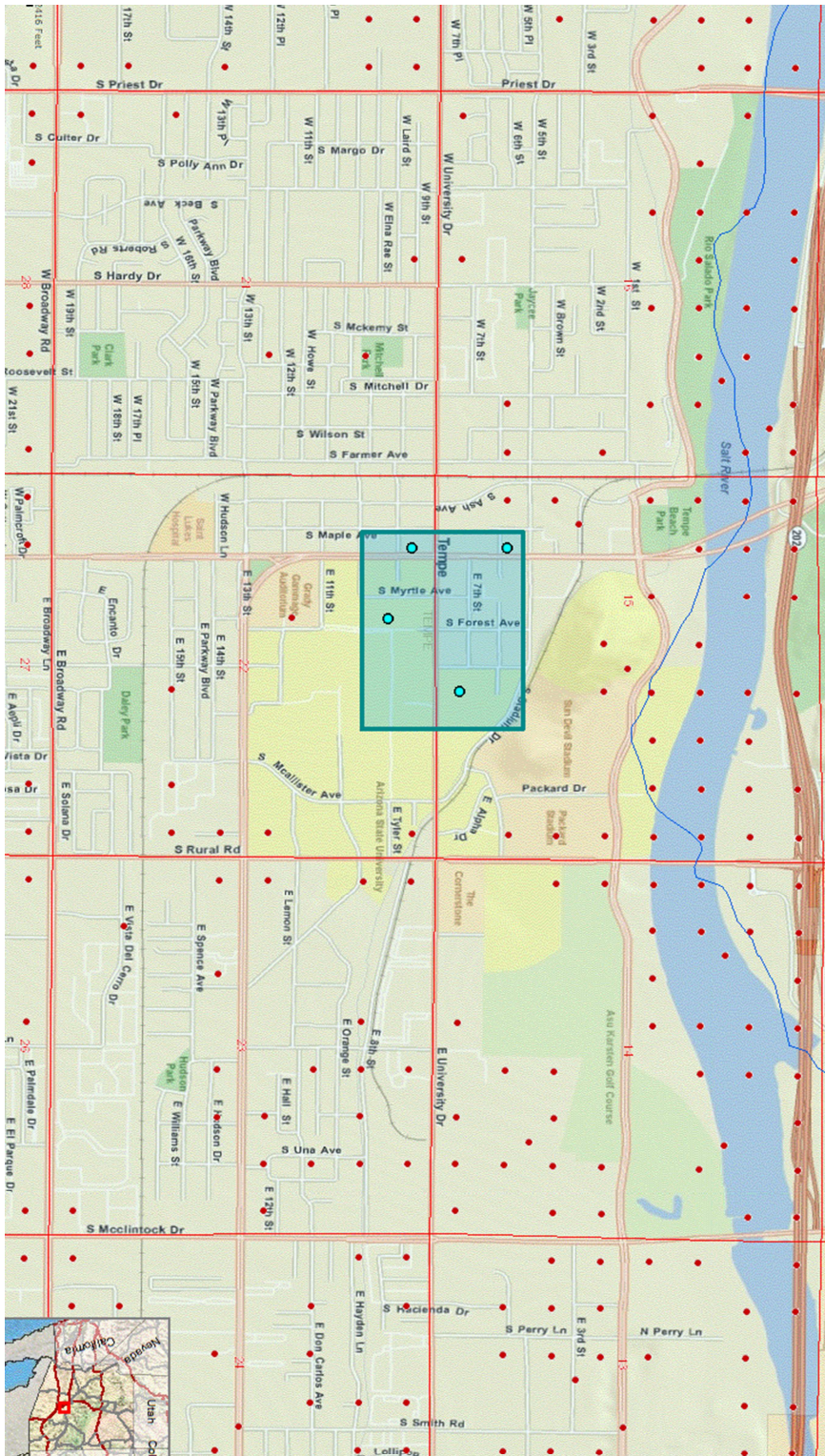
Figure 19. ADWR Well Registry. Selected wells are shown in blue. Source: https://gisweb.azwater.gov/WellRegistry

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A summary of the data input into the model is shown in Table 11. The model was run with both the Penman-Monteith and Hargreaves-Samani methods for estimation of reference ET. The crop coefficient factor Kc was kept at one to allow for comparison to the model without a dry well. The results of the model run are shown in Table 12.

The hydraulic conductivity value of 0.04 in/hr results in a dry well flow rate of 0.014 cfs. This is a low flow rate, and results in less than 1% of the outflow volume flowing through the dry well in both model runs. The FCDMC recommends assuming a 0.1 cfs flow rate into dry wells for design (Flood Control District of Maricopa County, 2007). With a flow rate this low, a dry well is not worth the price. The cost of the basin with one dry well is more expensive than with just bioretention, as the dry well only allows the basin to be slightly smaller (102ft by 102ft versus 109ft by 109ft).

5.1.4 Infiltration, Evapotranspiration, and Variable Number Dry Wells

Based on unsaturated flow modeling of dry wells in Washington State, Massmann (2004) recommends a spacing of five times the radius of the excavation for the dry well to prevent overlap of groundwater mounds for sites with water tables deeper than 30 feet. For the dry wells used by the WSDOT (Figure 8), this results in a spacing of approximately 50 feet. A typical dry well in Arizona has a radius of two feet, resulting in a spacing of only ten feet. However, dry wells in Arizona can be as deep as 180ft, while the ones used by WSDOT are only 8 to 12ft deep. ADEQ recommends a spacing of 100 feet center to center (Arizona Department of Environmental Quality, 2009). A 100 foot minimum spacing is used in the model.

Table 11. Input Data for Dry Well

	Variable and Unit	Value
U_{dw}	Unit Cost of Dry Well ($/dry well)	10,000
U_{drill}	Unit Cost of Drilling ($/ft)	50
SP_{dw}	Dry Well Minimum Spacing (ft)	100
K_{dw}	Average Field Sat. Hydraulic Conductivity (in/hr)	0.04
a	Radius of Dry Well (ft)	2
D_{aq}	Depth to Aquifer (ft)	100

Table 12. Bioretention with Single Dry Well Model Results

ET Model	Cost	Basin Depth (ft)	Side Length (ft)	Storage Volume (ft^3)
Penman Monteith	$66,364	2.3	94	20,186
Hargreaves-Samani	$70,710	1.8	102	19,298

ET Model	Rational Method Intensity (in/hr)	ET Rate (in/hr)	Critical Infiltration Rate (in/hr)	Peak Inflow Rate (cfs)
Penman Monteith	1.5	0.010	0.28	5.3
Hargreaves-Samani	1.7	0.008	0.28	5.8

ET Model	Number of Dry Wells	Depth of Dry Wells (ft)	Dry Well Flow Rate (cfs)	Critical Storm Duration (min)
Penman Monteith	1	90	0.014	67
Hargreaves-Samani	1	90	0.014	58

ET Model	Total Storm Runoff (ft^3)	ET Outflow (ft^3)	Infiltration Outflow (ft^3)	Dry Well Outflow (ft^3)
Penman Monteith	21,151	16,323 (77%)	3,108 (15%)	1,751 (0.08%)
Hargreaves-Samani	20,194	14,920 (74%)	3,523 (17%)	1,751 (0.09%)

The GAMS MINLP model for variable number of dry wells was run for the example application. The value of the integer variable for number of dry wells in the basin was optimized to be zero, resulting in the same results as the NLP model with only infiltration and ET, presented in Table 10.

5.2 Example Application 2

Example Application 2 utilizes the same site specific data as Example Application 1, except with a watershed area of 30 acres rather than 3.6 acres. The larger watershed area is used to demonstrate the use of the MINLP model in situations with multiple dry wells. The MINLP model allowing multiple dry wells was run with different soil types and crop coefficient values, effectively varying the infiltration and evapotranspiration rates. This was done in order to investigate how the model would react to different outflow volumes. The hydraulic conductivity at depth is first assumed to be the same as the soils at the surface, then the hydraulic conductivity is assumed to increase with depth.

5.2.1 Hydraulic Conductivity Constant

In order to effectively model the dry well flow rate in different soil types, the hydraulic conductivity at depth must be estimated. A geotechnical investigation is usually conducted for new developments to determine soil conditions at depth, which was not possible for this study. The model was run assuming the hydraulic conductivity of the soils over the depth of the dry well are the same as the hydraulic conductivity of the surface soils. The hydrology design manual for Maricopa County gives values of hydraulic conductivity and wetting front suction head for use in Green-Ampt for various

soil types in the area (Flood Control District of Maricopa County, 2013). Soil porosity values for the soil types are from "Ground and Surface Water Hydrology" (Mays, 2011). The initial soil moisture content was assumed to be 0.3 for all soil types. The soil parameters for clay loam and soil types are shown in Table 13.

The reference evapotranspiration rate used in all model runs is calculated using the Penman-Monteith method, as the results of the two ET methods do not vary much for this application. The results of the MINLP model runs are shown in Table 14. The model forces the basin to be square, so only one side length is shown. The model runs with the clay loam soil type and crop coefficient value of 0.5 or 0.2 were infeasible due to the low outflow rates. The peak inflow rate and the total storm runoff volume change for each model run. This is due to changing critical storm durations for the varying outflow rates. Different storm durations will result in different rainfall intensities, as represented in the IDF curve in Figure 18. The critical storm duration changes with changes in the outflow rates. A change in the outflow rate will change the storm duration that will result in the maximum storage volume required.

Dry wells were only included in three of the model runs, even though the watershed area is 30 acres. This is due to the low flow rates from the dry wells. The only model run that was feasible with clay loam was with a crop coefficient of 1.0, and even then the basin did not include a dry well because the flow rate would be 0.01 cfs. In soils where the hydraulic conductivity at depth is not greater than the surface hydraulic conductivity, dry wells are most often not worth the cost.

Table 13. Soil Parameters for Green-Ampt Infiltration and Dry Well Flow Rate. Source: *(Flood Control District of Maricopa County, 2013) **(Mays, 2011)

Soil Type	Sandy Loam	Silt Loam	Clay Loam
Hydraulic Conductivity* (in/hr)	0.40	0.25	0.04
Wetting Front Suction Head* (in)	4.3	6.6	8.2
Porosity**	0.45	0.50	0.46
Initial Soil Moisture Content	0.3	0.3	0.3

Table 14. MINLP Model Results for 30-Acre Development with Constant Hydraulic Conductivity Values

Soil / Kc	Cost	Basin Depth (ft)	Side Length (ft)	Storage Volume (ft^3)
Sandy Loam / 1.0	$229,764	3.0	200	120,261
Silt Loam / 1.0	$243,354	3.0	206	127,844
Clay Loam / 1.0	$403,752	2.1	290	175,752
Sandy Loam / 0.5	$236,028	3.0	196	115,672
Silt Loam / 0.5	$265,381	2.1	232	115,032
Sandy Loam / 0.2	$242,630	3.0	193	111,277
Silt Loam / 0.2	$284,120	2.7	202	112,561

| | Rational Method | | Critical Infiltration Rate | |
Soil / Kc	Intensity (in/hr)	ET Rate (in/hr)	(in/hr)	Peak Inflow Rate (cfs)
Sandy Loam / 1.0	2.7	0.010	1.6	79
Silt Loam / 1.0	2.5	0.010	1.3	73
Clay Loam / 1.0	1.3	0.010	0.3	39
Sandy Loam / 0.5	2.9	0.005	1.7	84
Silt Loam / 0.5	2.9	0.005	1.3	84
Sandy Loam / 0.2	3.0	0.002	1.7	88
Silt Loam / 0.2	3.0	0.002	1.4	87

| | | | Dry Well Flow Rate | Critical Storm Duration |
Soil / Kc	Number of Dry Wells	Depth of Dry Wells (ft)	(cfs)	(min)
Sandy Loam / 1.0	0	N/A	N/A	26
Silt Loam / 1.0	0	N/A	N/A	31
Clay Loam / 1.0	0	N/A	N/A	79
Sandy Loam / 0.5	1	N/A	N/A	24
Silt Loam / 0.5	0	N/A	N/A	24
Sandy Loam / 0.2	2	90	0.15	22
Silt Loam / 0.2	4	90	0.09	22

| | Total Storm Runoff | | | |
Soil / Kc	(ft^3)	ET Outflow (ft^3)	Infiltration Outflow (ft^3)	Dry Well Outflow (ft^3)
Sandy Loam / 1.0	125,397	61,297 (49%)	64,100 (51%)	0
Silt Loam / 1.0	113,340	75,997 (67%)	57,343 (51%)	0
Clay Loam / 1.0	185,933	156,746 (84%)	29,188 (16%)	0
Sandy Loam / 0.5	120,128	33,774 (28%)	68,620 (57%)	17,734 (15%)
Silt Loam / 0.5	119,909	49,897 (42%)	70,012 (58%)	0
Sandy Loam / 0.2	115,324	13,105 (11%)	66,452 (58%)	35,767 (31%)
Silt Loam / 0.2	116,654	15,244 (13%)	55,894 (48%)	45,516 (39%)

5.2.3 Hydraulic Conductivity Greater at Depth

To investigate how the MINLP model responds to higher permeability soils located beneath the surface, the model runs for Example Application 2 were repeated with hydraulic conductivity values over the depth of the dry well greater than the hydraulic conductivity of the surface soils. This was done because dry wells are normally drilled into permeable formations in the vadose zone (Bouwer, 2002). ADEQ recommends a minimum penetration of 10 continuous feet into "permeable porous soils" (Arizona Department of Environmental Quality, 2009). Torrent Resources notes that in their experience in the alluvial deposits in the Phoenix area "deeper soils typically equate to higher infiltration rates" (Torrent Resources, 2015).

To be clear, the assumption of greater hydraulic conductivity at depth is a big assumption and extremely location dependent. This assumption should only be made when a complete geotechnical investigation to the depth of the bottom of the dry wells confirms that indeed there are much more permeable soils that the dry well(s) would drain to.

Assumptions were made for the hydraulic conductivity at depth for each of the surface soil types. It was assumed that with a surface soil type of clay loam, the hydraulic conductivity at depth would increase to that of silt loam. It was assumed that with a surface soil type of silt loam, the hydraulic conductivity at depth would increase to that of sandy loam. It was assumed that with a surface soil of sandy loam, the hydraulic conductivity would increase to 0.60 in/hr, half of the hydraulic conductivity of loamy sand (Flood Control District of Maricopa County, 2013). These assumed values and the soil parameters for Green-Ampt for the different soil types are shown in Table 15.

The MINLP model was run for the 30-acre development with various crop coefficients and soil types. The results of the model runs are shown in Table 16. Unlike the model runs with a constant hydraulic conductivity, feasible solutions were found for the basins with clay loam and low ET rates. However, even with the higher dry well flow rates, no dry wells were included in the basins with a crop coefficient of 1.0. The dry well flow rates ranged from 0.09 cfs to 0.20 cfs. The design dry well flow rate suggested by FCDMC is 0.1 cfs (Flood Control District of Maricopa County, 2007). Torrent Resources claims that the average flow rate for dry wells they install in Maricopa County is 0.5 cfs (Torrent Resources, 2015). There is no publically available data on the flow rates of existing dry wells installed in Maricopa County. Observed flow rates for 27 dry wells in Washington State were published by Massmann (2004). The flow rates range from 0.03 cfs to 1.44 cfs, with a mean of 0.72 cfs. The observed flow rate data and comparison to the estimated flow rates are presented in Table 3 and Figure 16.

The largest number of dry wells in the model runs with a soil type of sand loam was two (with a Kc of 0.2), less than the maximum allowed, reinforcing the conclusion that if permeable soils are present at the surface and land is available, dry wells may not be worth the cost, even if more permeable soils are located deeper. In the case of clay loam with crop coefficient values of 0.5 and 0.2, 6 and 7 dry wells (maximum) are added to the basin, respectively. In this case low permeability soils are located at the surface and higher permeability soils are located below, making dry wells an effective option.

Table 15. Assumed Hydraulic Conductivity Values for Dry Well Flow Rate and Soil Parameters for Green-Ampt Infiltration. Source: *(Flood Control District of Maricopa County, 2013) **(Mays, 2011)

Surface Soil Type	Sandy Loam	Silt Loam	Clay Loam
Hydraulic Conductivity* (in/hr)	0.40	0.25	0.04
Wetting Front Suction Head* (in)	4.3	6.6	8.2
Porosity**	0.45	0.50	0.46
Initial Soil Moisture Content	0.3	0.3	0.3
Assumed Hydraulic Conductivity over Depth of Dry Well (in/hr)	0.60	0.40	0.25

Table 16. MINLP Model Results for 30-acre Development with Greater Hydraulic Conductivity Values at Depth.

Soil / Kc	Cost	Basin Depth (ft)	Side Length (ft)	Storage Volume (ft^3)
Sandy Loam / 1.0	$229,764	3.0	200	120,261
Silt Loam / 1.0	$243,354	3.0	206	127,844
Clay Loam / 1.0	$403,752	2.1	290	175,752
Sandy Loam / 0.5	$232,684	3.0	195	113,811
Silt Loam / 0.5	$253,159	3.0	198	117,137
Clay Loam / 0.5	$392,489	2.3	245	141,159
Sandy Loam / 0.2	$236,500	3.0	190	107,870
Silt Loam / 0.2	$259,349	3.0	194	112,512
Clay Loam / 0.2	$448,776	1.7	278	130,130

Soil / Kc	Rational Method Intensity (in/hr)	ET Rate (in/hr)	Critical Infiltration Rate (in/hr)	Peak Inflow Rate (cfs)
Sandy Loam / 1.0	2.7	0.010	1.6	79
Silt Loam / 1.0	2.5	0.010	1.3	73
Clay Loam / 1.0	1.3	0.010	0.3	39
Sandy Loam / 0.5	2.9	0.005	1.7	85
Silt Loam / 0.5	2.8	0.005	1.4	83
Clay Loam / 0.5	2.2	0.005	0.4	64
Sandy Loam / 0.2	3.1	0.002	1.8	91
Silt Loam / 0.2	3.0	0.002	1.5	87
Clay Loam / 0.2	2.5	0.002	0.4	72

Soil / Kc	Number of Dry Wells	Depth of Dry Wells (ft)	Dry Well Flow Rate (cfs)	Critical Storm Duration (min)
Sandy Loam / 1.0	0	N/A	N/A	26
Silt Loam / 1.0	0	N/A	N/A	31
Clay Loam / 1.0	0	N/A	N/A	79
Sandy Loam / 0.5	1	90	0.20	23
Silt Loam / 0.5	2	90	0.15	25
Clay Loam / 0.5	6	90	0.09	38
Sandy Loam / 0.2	2	90	0.20	20
Silt Loam / 0.2	3	90	0.15	22
Clay Loam / 0.2	7	90	0.09	31

Soil / Kc	Total Storm Runoff (ft^3)	ET Outflow (ft^3)	Infiltration Outflow (ft^3)	Dry Well Outflow (ft^3)
Sandy Loam / 1.0	125,397	61,297 (49%)	64,100 (51%)	0
Silt Loam / 1.0	133,340	75,997 (57%)	57,343 (43%)	0
Clay Loam / 1.0	185,933	15,6746 (84%)	29,188 (16%)	0
Sandy Loam / 0.5	118,181	31,055 (26%)	64,001 (54%)	23,126 (20%)
Silt Loam / 0.5	121,546	34,205 (28%)	51,870 (43%)	35,472 (29%)
Clay Loam / 0.5	145,605	55,929 (38%)	21,402 (15%)	68,274 (47%)
Sandy Loam / 0.2	111,789	11,007 (10%)	57,541 (51%)	43,241 (39%)
Silt Loam / 0.2	116,499	13,184 (11%)	49,937 (43%)	53,378 (46%)
Clay Loam / 0.2	134,040	28,782 (21%)	25,606 (19%)	79,653 (59%)

6. SUMMARY AND CONCLUSIONS

6.1 Summary of Results

All phases of the model were successfully applied to Example Application 1. The Avondale clay loam located at the project site resulted in low infiltration rates in all phases of the model. The infiltration basin designed with no drain time constraint resulted in a computed total drain time of over a month, and when a drain time constraint of 36 hours added, the model run was infeasible. In locations with low permeability soils such as clay loam, the outflows must be greater to drain the basin in the required time. This can include an underdrain, imported high permeability soil media, a dry well, or evapotranspiration.

With the addition of the evapotranspiration outflow to the basin, an optimal solution was feasible with the 36 hours drain time constraint. However, the solution was feasible only with a crop coefficient value of one, the maximum. The Penman-Monteith method and the Hargreaves-Samani method resulted in similar reference ET values (0.10 in/hr and 0.08 in/hr, respectively). This demonstrates the applicability of the Hargreaves-Samani, and that the Penman-Monteith method isn't inherently more accurate because of its greater data requirements. The choice between the two methods should be made based upon the reliable data available for the development site.

For the bioretention basin with no dry wells, ET accounted for 84% of the outflow volume with the Penman-Monteith ET rate and 81% of the outflow volume with the Hargreaves-Samani ET rate, both very high values. This is due to the low permeability soils and the high crop coefficient value used. Lower crop coefficient values would

reduce the ET rate, but resulted in infeasible solutions. This reinforces the need for a dry well or underdrain to facilitate faster drain times.

The introduction of a dry well to the basin increased the outflow rate from the basin only slightly, but the basin was more expensive that with no dry wells based on the cost function input. The dry well flow rate was so low that it accounts for less than 1% of the total outflow volume from the basin. This is because the hydraulic conductivity at depth was assumed to be the same as the clay loam located at the surface, 0.04 in/hr. ET accounted for 77% of the outflow with the Penman-Moneith method and 74% with the Hargreaves-Samani method. The ET rates were calculated based on a crop coefficient factor of one so that the results could be directly compared to the results of the model with no dry well. A lower crop coefficient such as the ones presented in Table 8 would result in lower ET rates, though plants adapted to arid and semi-arid regions may respond differently to stormwater inflows.

The MINLP model for the development resulted in a value of zero for the integer variable defining the number of dry wells in the basin, resulting in the same results as the bioretention NLP model with no dry wells. The primary bioretention basin actually constructed at the CAVC development, shown in Figure 20, does not include a dry well. However, this basin can't be compared directly to the model results because it drains only a fraction of the development area. The size of the basin is estimated to be 80ft by 35ft by 2.5ft, resulting in a volume of approximately seven thousand cubic feet.

Figure 20. Bioretention Basin Installed at CAVC Development. Photos: Mason Lacy
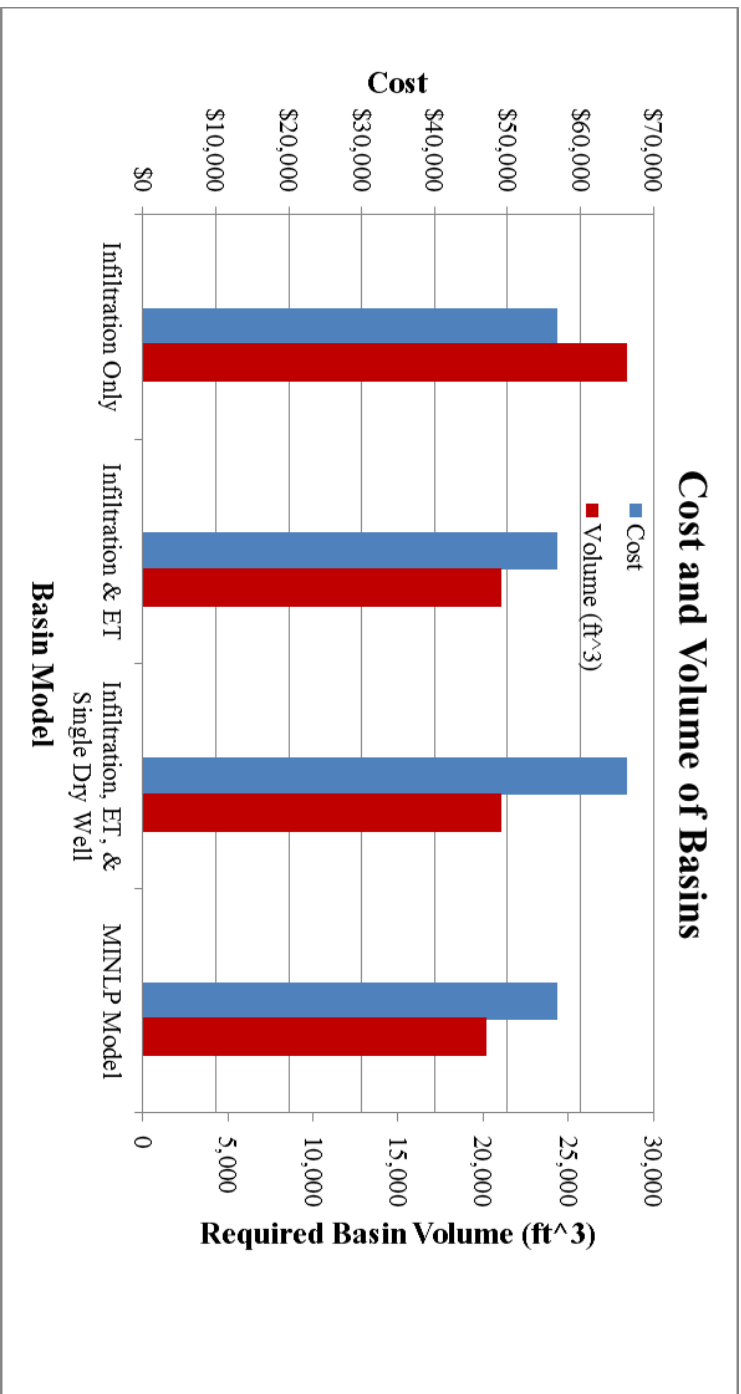
Figure 21. Cost and Volume Comparison of the Different Model Runs for Example Application 1

Figure 21 shows a comparison of the cost and volume of the basins designed by the models. The results shown have an ET rate calculated from the Penman-Monteith equation, as the results from the two methods were almost identical.

For Example Application 2, the MINLP model was run with a watershed area of 30 acres rather than 3.6 acres to investigate scenarios with multiple dry wells. Different soil types and different crop coefficient values changed the method of stormwater outflow drastically. Figure 22 shows the portion of total stormwater inflow that infiltrated, evaporated or transpired, or flowed into a dry well for the model runs with the same hydraulic conductivities at depth as at the surface. Figure 23 shows the cost and number of dry wells for these model runs.

For the model runs with a crop coefficient of 1.0, the portion of water infiltrated decreases as the soil type changes from higher permeability to lower permeability, resulting in more water available for evapotranspiration. The model run with sandy loam and a Kc value of 1.0 has nearly equal ET and infiltration outflow volumes, while the model run with silt loam has a lower infiltration volume and a greater ET volume. This change is even greater with a soil type of clay loam. No dry wells are added to the basin for any of the soil types with a crop coefficient of 1.0.

For the model runs with a crop coefficient of 0.5, a dry well is added when the soil type is sandy loam, no dry well is added with silt loam, and the clay loam solution is infeasible. The change from sandy loam to silt loam results in a decreased surface infiltration rate and dry well flow rate, however the dry well flow rate is impacted more as it becomes no longer economical to include a dry well in the basin.

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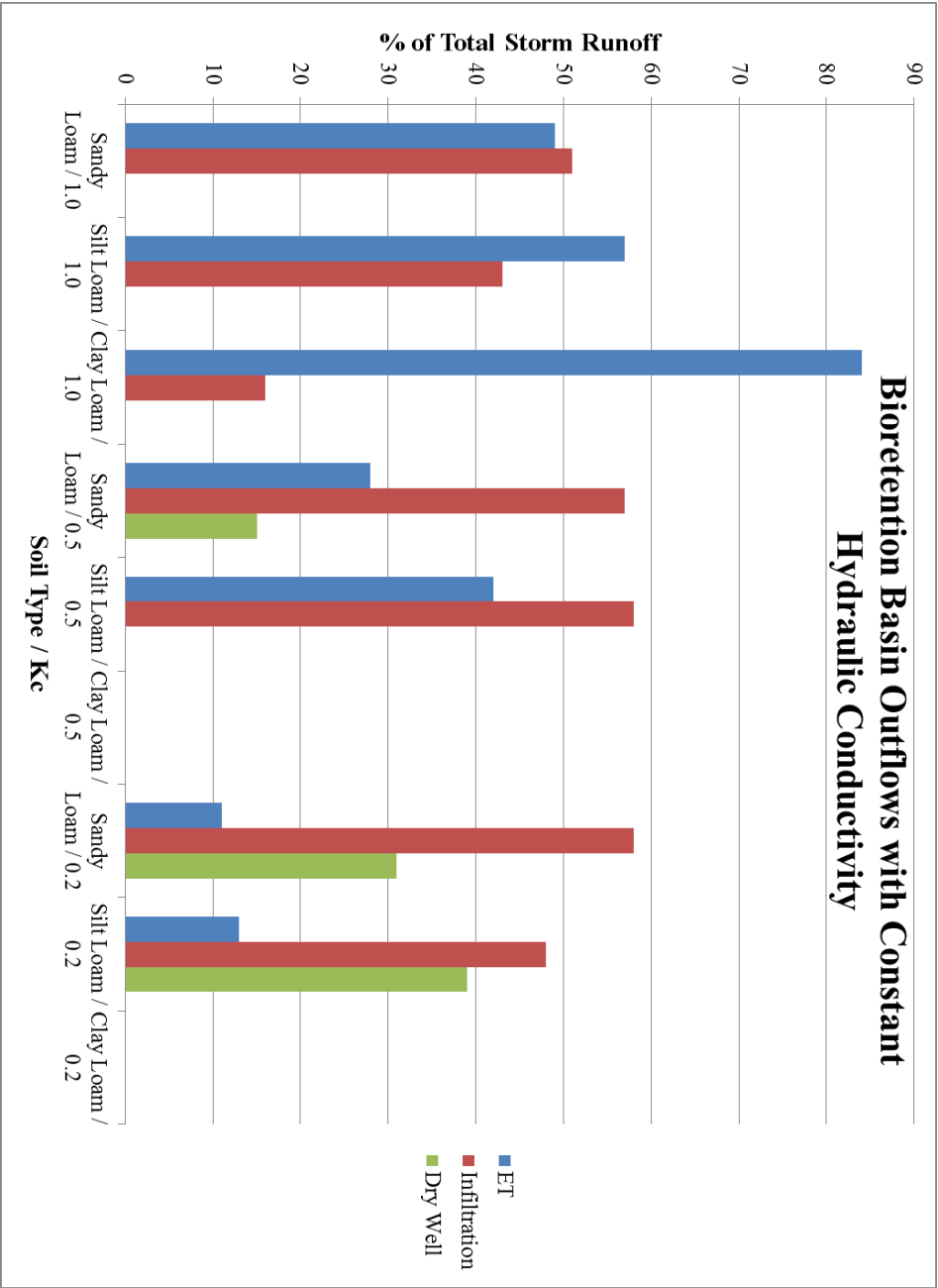
Figure 22. ET, Infiltration, and Dry Well Outflows in % of Total Inflow for Example 2 with Constant Hydraulic Conductivity.
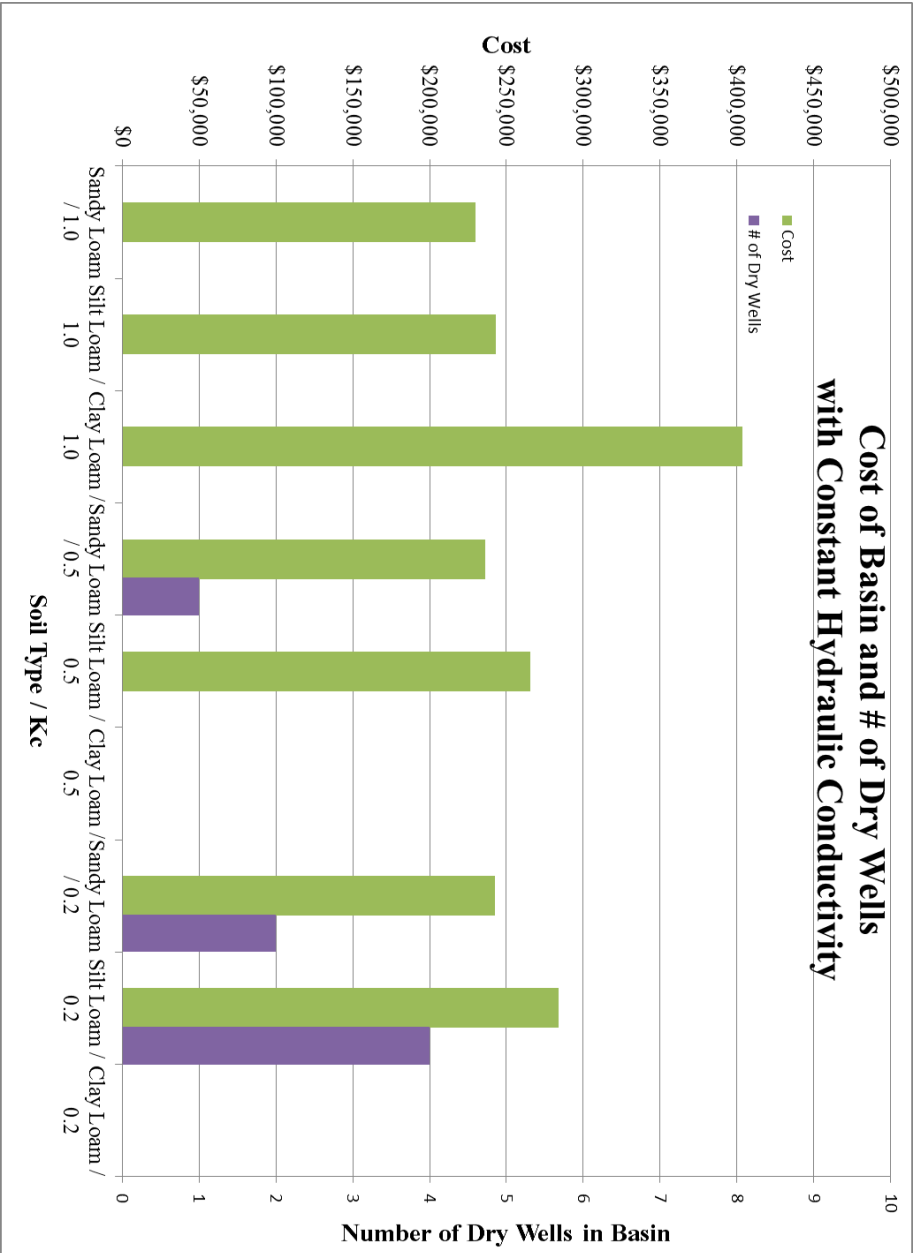
Figure 23. Cost and Number of Dry Wells in Basin for Model Runs in Figure 22
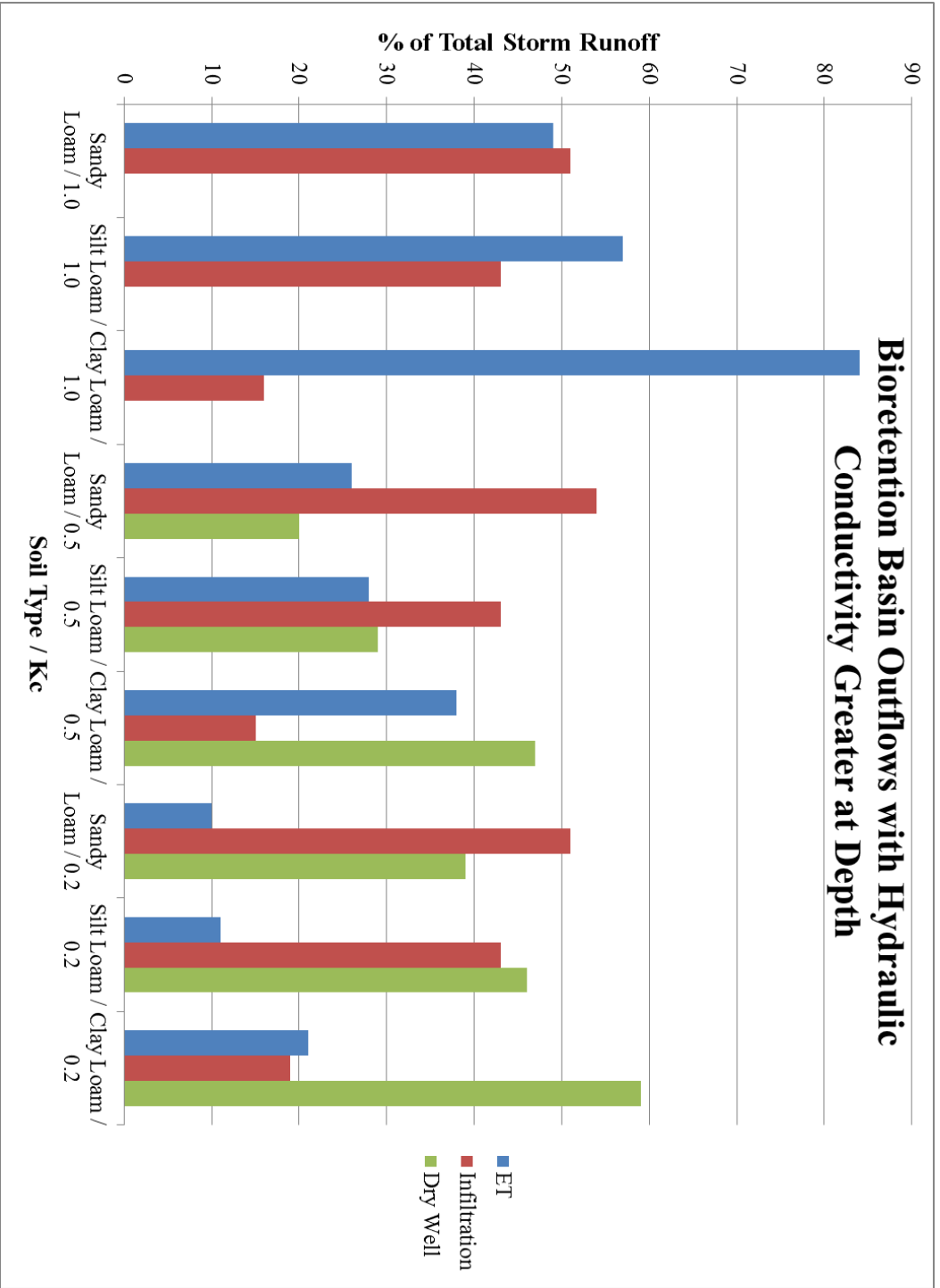
Figure 24. ET, Infiltration, and Dry Well Outflows in % of Total Inflow for Example Application 2 with Higher Hydraulic Conductivity at Depth
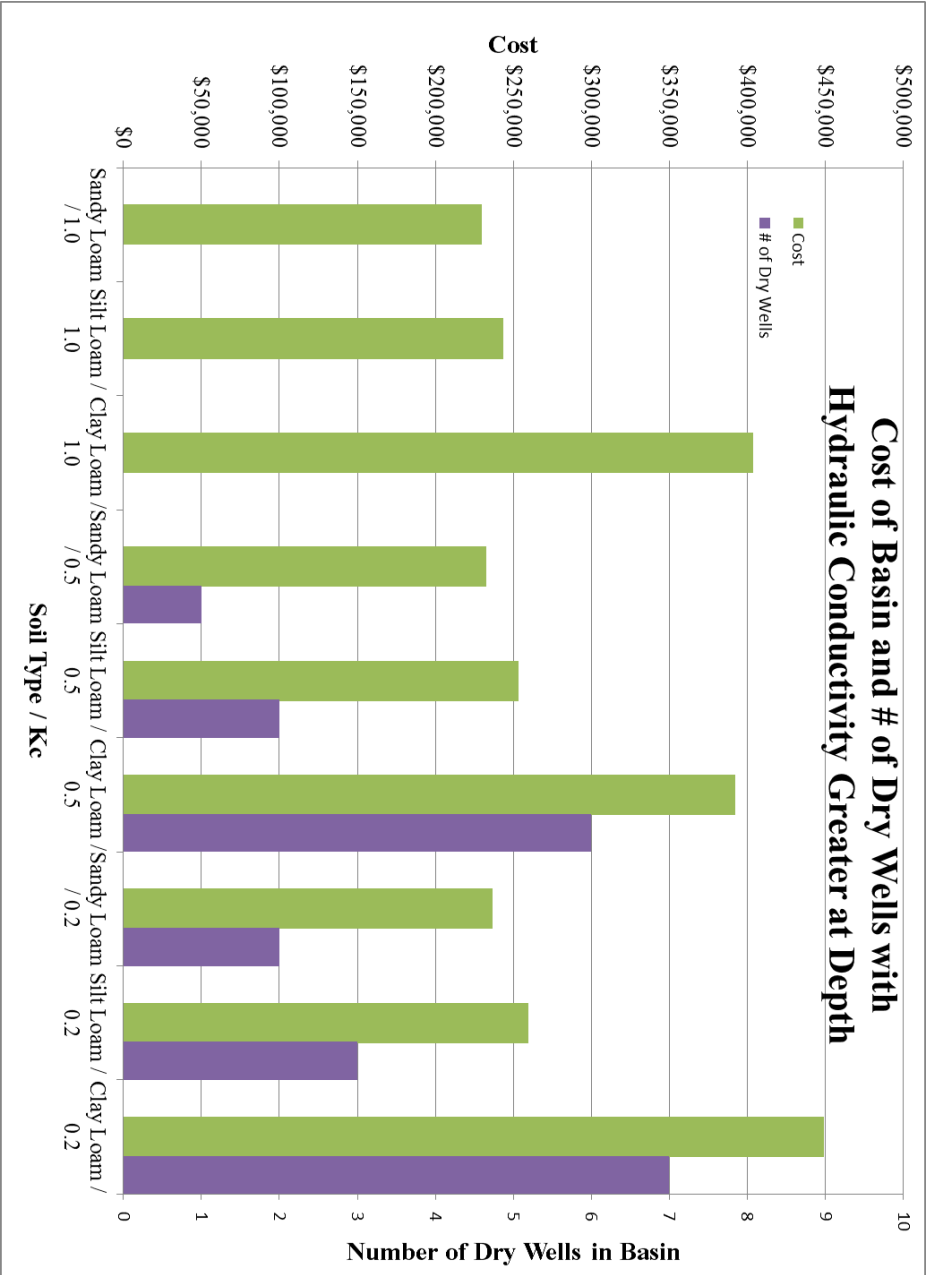
Figure 25. Cost and Number of Dry Wells in Basin for Model Runs in Figure 24

Dry wells are included in both the basins with a crop coefficient of 0.2, though surface infiltration is the largest portion of outflow volume. Surface infiltration is the largest outflow for all of the model runs except when the crop coefficient is 1.0 causing high ET rates. As would be expected, the portion of stormwater removed by ET decreases as the crop coefficient value decreases.

The results of the model runs with a higher hydraulic conductivity at depth are presented in Figure 24 and Figure 25. Solutions were feasible for all of the surface soil types. Even with the higher dry well flow rates, for the runs with a crop coefficient of 1.0 no dry wells were used. Dry wells were used in the rest of the model runs.

For the model runs with a crop coefficient of 0.5 or 0.2, as the surface soil type becomes less permeable excess water is removed primarily through the dry wells. The portion of ET outflow does increase with the lower infiltration rates, but the portion of outflow volume through the dry wells experiences a greater increase. The costs of the basins with clay loam are all the most expensive because of the cost of the dry wells or additional land needed to make up for the low permeability soil. More dry wells were used for the model runs where it was assumed that soils at depth were more permeable than the soils at the surface because the higher flow rates made up for the additional cost.

6.2 Conclusions

The outflows of a bioretention basin with dry wells can vary dramatically dependent on site parameters, as can be seen in the MINLP model runs. This demonstrates the applicability of using an optimization model such as the one developed in this study for the design of these basins, especially in larger watersheds. Care must be

taken to develop an accurate cost function for use in the optimization model, and determine accurate site parameters. The longest run time for any of the model runs was under 1.5 seconds, creating an efficient design tool.

Dry wells can be an effective approach to drain a basin in the required time, and the possibility of recharging depleted aquifers is exciting in many arid regions. However, an accurate description of the hydraulic conductivity at depth is essential for the use of dry wells. Based on the results of the MINLP model, dry wells are generally not worth the cost unless the soils become more permeable with depth. Also, the mathematical formulation of the model does not take into account clogging that is likely to occur over the life of the dry well. More research should be done to understand the benefits and consequences of using dry wells as a stormwater BMP. Even with the limitations of the model used here, it is an improvement over many of the simplistic methods now used to design dry wells in retention and bioretention basins. In practice, the number of dry wells in a basin can be reduced based on a constant head percolation test performed on a newly constructed dry well (Torrent Resources, 2015). This completely disregards the effects that clogging may have or any other reduction in efficiency over the lifetime of the dry well.

The model presented here could be used for many applications. The optimization model developed can be freely edited by the user to fit the parameters of a specific site. It is also relatively easy to modify or expand the model in the GAMS environment to customize its use for local design practices or standards. Optimization models for the design of stormwater BMPs have the promise to increase the efficiency and quality of the final facility.

6.3 Suggestions for Future Research

More research needs to be done on the hydraulics of dry wells, as well as the overall influence of dry wells on a watershed. Additional analytical methods for the estimation of the flow rate from dry wells should be developed, as well as studies testing the effectiveness of existing analytical methods such as the one used in this thesis. The hydraulic interaction between multiple dry wells needs to be better understood for facilities with multiple dry wells. There is little information on the performance of dry wells over a long period of time, and prior to wide adoption of the technology the effects of sediment clogging, maintenance, etc. needs to be better understood.

A limitation of the current model is the accuracy of ET rates based on crop coefficients. More accurate crop coefficient factors for use in bioretention in arid regions would be beneficial. Alternative methods for calculating ET in arid regions should also be explored.

The model developed in this thesis could be modified or expanded to include additional stormwater management options. An underdrain or overflow structure could be added into the model. The bioretention design optimization could be connected with permeable pavements or vegetative filter strips. Many times in practice several stormwater BMPs are used together to treat runoff, such as a grass swale leading to a bioretention basin.

From a storm water management perspective, water quality treatment is a primary design goal for bioretention basins and other stormwater BMPs. At this point, analytical methods for determining pollutant reduction in bioretention basins are not robust enough for inclusion in the model. There are many complicated treatment processes occurring

including sedimentation, filtration, sorption, plant uptake and storage, and microbial decomposition. As more methods are developed, it would be interesting to add pollutant loading to the model, and the resulting treatment effectiveness of the bioretention basin. Pollutant loadings could include nitrogen and/or phosphorous, sediment, hydrocarbons, and heavy metals.

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APPENDIX A

GAMS NLP MODEL FOR BIORETENTION BASIN OPTIMIZATION WITH NO

DRY WELL

```
 1 $ TITLE Optimization Model for Design of Bioretention Basin with No Dry Well
 2 *
 3 *       Optimization Model created by Mason Lacy for Master's Thesis
 4 *       Spring 2016, Arizona State University Hydrosystems Engineering
 5 *       Thesis Advisor: L.W. Mays
 6 *
 7 *       Model expanded from the optimization model developed for
 8 *       infiltration basins by Stafford et al. (2015)
 9 *
10 *       Problem: Optimize the size of a bioretention basin given
11 *                certain model constraints and parameters of the soil
12 *                conditions, infiltration rate, meteorological data,
13 *                vegetation parameters, and any other necessary
14 *                parameters.
15 *
16 *       ***All soil parameters, watershed properties, etc., are
17 *                variable and freely editable by the user.
18 *
19 *----------------------------------------------------------------
20 *
21 SCALARS
22         U_e     UNIT COST OF EXCAVATION ($ PER CU.FT.)   /0.74/
23 *               (BASED ON ~$20/CU.YD. - IDAHO AREA)
24         U_t     UNIT COST OF TREES ($ PER TREE)          /200/
25 *               (TREES PLANTED AT 20FT SPACING)
26         U_sh    UNIT COST OF SHRUBS ($ PER SHRUB)        /50/
27 *               (SHRUBS PLANTED AT 10FT SPACING)
28         U_g     UNIT COST OF GRASSES ($ PER GRASS UNIT) /10/
29 *               (GRASSES PLANTED AT 5FT SPACING)
30         U_l     UNIT COST OF LAND ($ PER SQ. FT.)        /0.6887/
31 *               (BASED ON $30,000/AC - USDA 2013 CROPLAND, AVG. VALUE)
32         S_t     SPACING OF TREES (FT)                    /20/
33 *               (TREE SPACING IN VEGETATED BASIN)
34         S_sh    SPACING OF SHRUBS (FT)                   /10/
35 *               (SPACING OF SHRUB SPECIES IN VEGETATED BASIN)
36         S_G     SPACING OF GRASSES (FT)                  /5/
37 *               (SPACING OF GRASS SPECIES IN VEGETATED BASIN)
38 *
39 *       MODIFIED RATIONAL METHOD SCALARS
40         C       DIMENSIONLESS RUNOFF COEFFICIENT         /0.97/
41 *               ESTIMATED FOR CAVC BASED ON GOOGLE EARTH IMAGERY
42         A       WATERSHED (DEVELOPMENT) AREA (ACRES)     /3.6/
43 *               ESTIMATED USING GOOGLE EARTH AREA FOR CAVC PROPERTY
44         a_i     REGIONAL INTENSITY "a" COEFFICIENT       /45.92/
45         b_i     REGIONAL INTENSITY "b" COEFFICIENT       /10/
46         n_i     REGIONAL INTENSITY "n" COEFFICIENT       /0.786/
47 *               REGIONAL COEFFICIENTS COMPUTED FROM NOAA
48 *               ATLAS 14 VALUES FOR PRECIPITATION RETURN PERIODS
49 *
50 *       GREEN-AMPT INFILTRATION SCALARS
51 *       ***Soil Parameters for Green-Ampt model can be found on
52 *               pp.317 of 'Ground and Surface Water Hydrology' (Mays)
53 *
54 *       K       HYDRAULIC CONDUCTIVITY (IN PER HR)       /0.43/
55 *       P       PSI = WETTING FRONT SUCTION HEAD (IN)    /4.33/
56 *       nu      POROSITY                                 /0.453/
57 *       T_i     THETA_i = INITIAL SOIL MOISTURE CONTENT /0.3/
58 *       assumes sandy loam
59
60 *       K       HYDRAULIC CONDUCTIVITY (IN PER HR)       /0.26/
```

```
61 *        P      PSI = WETTING FRONT SUCTION HEAD (IN)   /6.57/
62 *        nu     POROSITY                                /0.501/
63 *        T_i    THETA_i = INITIAL SOIL MOISTURE CONTENT /0.3/
64 *     assumes silt loam
65
66        K      HYDRAULIC CONDUCTIVITY (IN PER HR)      /0.04/
67        P      PSI = WETTING FRONT SUCTION HEAD (IN)   /8.22/
68        nu     POROSITY                                /0.464/
69        T_i    THETA_i = INITIAL SOIL MOISTURE CONTENT /0.3/
70 *     assumes clay loam
71 *
72 *        EVAPOTRANSPIRATION PARAMETERS
73 *        Choose either Penman Monteith or Hargreaves-Samani
74 *
75 *        ASCE Penman Monteith
76 *      Rn     NET RADIATION (MJ PER M^2*HR)           /0.90/
77 *      G      SOIL HEAT FLUX DENSITY (MJ PER M^2*HR)  /0/
78 *      Ta     MEAN HOURLY AIR TEMPERATURE (DEGREES C) /22.8/
79 *      u2     MEAN HOURLY WIND SPEED @ 2M (M PER SEC) /0/
80 *      es     SATURATION VAPOR PRESSURE (KPA)         /2.90/
81 *      ea     MEAN ACTUAL VAPOR PRESSURE (KPA)        /0.46/
82 *      Delta  SLOPE SAT VAPOR PRESS-TEMP (KPA PER C)  /.1681/
83 *      Gamma  PSYCHROMETRIC CONSTANT (KPA PER C)      /.0672/
84 *      C_n    NUMERATOR CONSTANT (K*MM*S^3 PER MG*HR) /66/
85 *      C_d    DENOMINATOR CONSTANT (S PER M)          /0.25/
86 *      Kc     CROP COEFFICIENT                        /1/
87 *     parameters estimated from meteorological data from
88 *     Phoenix Sky Harbor Airport
89 *
90 *        Hargreaves-Samani
91        KT     EMPIRICAL COEFFICIENT                   /0.162/
92 *     0.162 for interior regions, 0.19 for coastal regions
93        Ra     EXTRATERRESTRIAL RADIATION (MM PER DAY) /16.35/
94 *     water depth equivalent, function of day of yr and latitude
95        Tmax   MAX DAILY AIR TEMP (DEGREES C)          /28.3/
96        Tmin   MIN DAILY AIR TEMP (DEGREES C)          /17.2/
97        Tavg   AVG DAILY AIR TEMP (DEGREES C)          /22.8/
98        Kc     CROP COEFFICIENT                        /1/
99 *
100 *        TIME OF CONCENTRATION CALCULATION SCALARS
101        L_O    LENGTH OF OVERLAND FLOW (FT)            /400/
102        CN     WEIGHTED CURVE NUMBER FOR WATERSHED     /85/
103        S      AVERAGE SURFACE SLOPE (PERCENT)         /3.0/
104 *
105 *        DESIGN CRITERIA
106        FB     FREEBOARD ALLOWANCE (FT)                /1.0/
107        SL     SLOPE WIDTH AROUND BASIN PERIMETER (FT) /9/
108        DR     MAX TIME OF INUNDATION (DRAIN TIME) (HR)/36/;
109 *     allows for 1.0 feet of freeboard at the top of the
110 *     infiltration basin for flood risk mitigation
111 *     36 hr drain time required by Maricopa County
112 *
113 *----------------------------------------------------------------
114 *
115 VARIABLES
116        Z      COST IN DOLLARS (OBJECTIVE FUNCTION VARIABLE);
117 *             objective is to MINIMIZE the cost, Z, for the basin
118 *             while meeting the flood control objective
119
120 POSITIVE VARIABLES
```

```
121          L       LENGTH OF INFILTRATION BASIN (FT)
122          W       WIDTH OF INFILTRATION BASIN (FT)
123          d       DEPTH OF INFILTRATION BASIN (FT)
124          i       RATIONAL METHOD INTENSITY (IN PER HR)
125          F_t     CUMULATIVE GREEN-AMPT INFILTRATION (IN)
126          Q_p     PEAK DISCHARGE (CFS)
127          T_m     CRITICAL (MAXIMUM) DURATION (MIN)
128          f_m     INFILTRATION RATE (IN PER HR)
129          ET_m    EVAPOTRANSPIRATION RATE (IN PER HR)
130          t_c     TIME OF CONCENTRATION (MIN)
131          T_tot   TOTAL TIME OF INUNDATION (MIN)
132          F_tot   TOTAL INFILTRATION UNTIL EMPTY (IN)
133          P_H     PSI + AVG PONDING DEPTH IN BASIN (IN)
134          V_MOD   VOLUME OF RETENTION BASIN (CUBIC FT)    ;
135
136
137 EQUATIONS
138          V_req      VOLUME OF INFILTRATION BASIN REQUIRED
139          i_RAT      RATIONAL METHOD RAINFALL INTENSITY CALCULATION
140          GA_INF     GREEN AMPT CUMULATIVE INFILTRATION
141          Tm_CALC    CRITICAL DURATION CALCULATION
142          fm_CALC    INFILTRATION RATE AT CRITICAL DURATION
143 *        ET_CALC_PM ASCE PENMAN-MONTEITH EVAPOTRANSPIRATION RATE
144          ET_CALC_HS HARGREAVES-SAMANI EVAPOTRANSPIRATION RATE
145          SQUARE     ENSURES INFILTRATION BASIN IS SQUARE
146          CONC       TIME OF CONCENTRATION CALC
147          Q_peak     PEAK DISCHARGE CALCULATION
148          OBJ        OBJECTIVE FUNCTION
149          INF_TOT    GREEN AMPT CUMULATIVE INFILTRATION TOTAL UNTIL EMPTY
150          Ttot_CALC  TOTAL TIME OF INUNDATION
151          IN_LIM     INUNDATION TIME LIMITATION
152          P_H_CALC   CALC OF PSI + AVG PONDING DEPTH IN BASIN
153          VOL        CALC OF VOLUME OF RETENTION BASIN (FOR REFERENCE ONLY);
154
155
156 GA_INF.. F_t =E= (K)*(T_m/60)+(P+(d*12)/2)*(nu-T_i)*
157          LOG(1+(F_t/(((P+(d*12)/2)*(nu-T_i)))));
158 *        implicit solution to cumulative Green-Ampt infiltration equation
159 *        at critical duration [in]
160
161 fm_CALC..f_m =E= K*(1+(P+(d*12)/2)*(nu-T_i)/F_t);
162 *        calculation of infiltration rate at critical storm duration [in/hr]
163
164 *ET_CALC_PM..ET_m =E= (Kc/25.4)*(.408*Delta*(Rn-G)+Gamma*(C_n/(Ta+273))*
165 *                u2*(es-ea))/(Delta+Gamma*(1+C_d*u2));
166 *        calculation of evapotranspiration rate given parmeters (in/hr)
167
168 ET_CALC_HS..ET_m =E= Kc*0.0135*KT*Ra*(Tmax-Tmin)**0.5*(Tavg+17.8)/(24*25.4);
169 *        calculation of ET using Hargreaves-Samani Method
170
171 Tm_CALC..(T_m*(1-n_i)+b_i)/(T_m+b_i)**(n_i+1)-((L*W)*(f_m+ET_m)/(12*3600))
172               /(2*C*A)=E=0;
173 *        implicit solution to critical storm duration (time to max detention)
174
175 i_RAT..  i =E= a_i/((T_m+b_i)**n_i);
176 *        solution to modified rational method rainfall intensity
177
178 V_req..  60*Q_p*T_m-(L*W)*(F_t/12+ET_m*T_m/12) =E= L*W*d;
179 *        calculation for volumetric requirement of infiltration basin
180 *        based on inflow, time of duration, time of concentration,
```

```
181 *        and infiltration and evapotranspiration based on area of the basin
182
183 P_H_CALC..  P_H =E= P+(d*12)/2 ;
184 *        calculation of suction head plus average ponding depth (reference only)
185
186 SQUARE..  L =E= W;
187 *        equation to constrain length and width forcing resulting basin
188 *        shape to be square
189
190 CONC..    t_c =E= (100*(L_O**0.8)*((1000/CN)-9)**0.7)/(1900*(S**0.5));
191 *        calculation for time of concentration (SCS Lag equation)
192 *        to be used in the volumetric requirement calculation above
193
194 Q_peak..  Q_p =E= C*i*A;
195 *        calculation for peak surface runoff
196
197 INF_tot..  F_tot =E= (K)*(T_tot/60)+(P+(d*12)/2)*(nu-T_i)*
198           LOG(1+(F_tot/(((P+(d*12)/2)*(nu-T_i)))));
199 *        implicit solution to Green-Ampt infiltration equation for total [in]
200
201 Ttot_CALC..  60*Q_p*.5*((T_m-t_c)+(T_m+t_c)) =E=
202           L*W*(F_tot/12)+L*W*(ET_m*T_tot/12) ;
203 *        calculation of total time of inundation (min)
204
205 IN_LIM..  T_tot =L= 60*DR ;
206 *        limit on total time of inundation (drain time) (DR=24hrs)
207
208 OBJ..     Z =E= U_e*((L*W+2*SL**2+SL*L+SL*W)*(d+FB))+
209           (U_t*(S_t**(-2))+U_sh*(S_sh**(-2))+U_g*(S_g**(-2)))*((L*W)+(2*L+2*W)*
210           ((SL**2+(d+FB)**2)**.5)) + U_l*(L*W+4*SL**2+2*(SL*L+SL*W));
211 *        objective function to minimize costs based on excavation,
212 *        plants/planting, and land costs
213
214 VOL..  V_MOD =E= L*W*d    ;
215 *        volume of basin from model (for reference only)
216
217 **limiting factors for basin dimensions
218 F_t.lo = 0.01 ;
219 L.lo = 20;
220 W.lo = 20;
221 L.up = 1000;
222 W.up = 1000;
223 d.lo = 1.5;
224 d.up = 3.0;
225
226 *------------------------------------------------------------------
227
228 MODEL BIORETENTION_BASIN /ALL/;
229
230 *------------------------------------------------------------------
231
232 SOLVE BIORETENTION_BASIN USING NLP MINIMIZING Z;
233 *SOLVE BIORETENTION_BASIN USING NLP MAXIMIZING Z;
234
235 *------------------------------------------------------------------
236
237 DISPLAY t_c.l, i.l, d.l, L.l, W.l, Z.l, ET_m.l, Q_p.l, T_m.l, f_m.l,
238           F_t.l, T_tot.l, P_H.l, V_MOD.l ;
239
```

APPENDIX B

GAMS NLP MODEL FOR BIORETENTION BASIN OPTIMIZATION WITH

SINGLE DRY WELL

```
  1 $ TITLE Optimization Model for Design of Bioretention Basin with Single Dry Well
  2 *
  3 *          Optimization Model created by Mason Lacy for Master's Thesis
  4 *          Spring 2016, Arizona State University Hydrosystems Engineering
  5 *          Thesis Advisor: L.W. Mays
  6 *
  7 *          Model expanded from the optimization model developed for
  8 *          infiltration basins by Stafford et al. (2015)
  9 *
 10 *          Problem: Optimize the size of a bioretention basin & depth of
 11 *                   sinlge dry well given certain model constraints and
 12 *                   parameters of the soil conditions, infiltration rate,
 13 *                   meteorological data, vegetation parameters, drywell data
 14 *                   and other necessary parameters.
 15 *
 16 *          ***All soil parameters, watershed properties, etc., are
 17 *                   variable and freely editable by the user.
 18 *
 19 *-----------------------------------------------------------------
 20 *
 21 SCALARS
 22 *          PARAMETERS FOR COST MINIMIZATION
 23          U_e     UNIT COST OF EXCAVATION ($ PER CU.FT.)   /0.74/
 24 *                (BASED ON ~$20/CU.YD. - IDAHO AREA)
 25          U_t     UNIT COST OF TREES ($ PER TREE)          /200/
 26 *                (TREES PLANTED AT 20FT SPACING)
 27          U_sh    UNIT COST OF SHRUBS ($ PER SHRUB)        /50/
 28 *                (SHRUBS PLANTED AT 10FT SPACING)
 29          U_g     UNIT COST OF GRASSES ($ PER GRASS UNIT) /10/
 30 *                (GRASSES PLANTED AT 5FT SPACING)
 31          U_l     UNIT COST OF LAND ($ PER SQ. FT.)        /0.6887/
 32 *                (BASED ON $30,000/AC - USDA 2013 CROPLAND, AVG. VALUE)
 33          S_t     SPACING OF TREES (FT)                    /20/
 34 *                (TREE SPACING IN VEGETATED BASIN)
 35          S_sh    SPACING OF SHRUBS (FT)                   /10/
 36 *                (SPACING OF SHRUB SPECIES IN VEGETATED BASIN)
 37          S_G     SPACING OF GRASSES (FT)                  /5/
 38 *                (SPACING OF GRASS SPECIES IN VEGETATED BASIN)
 39          U_dw    UNIT COST OF DRYWELL ($ PER DRY WELL)    /10000/
 40 *
 41          U_drill COST OF DRILLING DRYWELL ($ PER FT)      /50/
 42 *
 43 *          MONDIFIED RATIONAL METHOD SCALARS
 44          C       DIMENSIONLESS RUNOFF COEFFICIENT          /0.97/
 45 *                ESTIMATED FOR CAVC BASED ON GOOGLE EARTH IMAGERY
 46          A       WATERSHED (DEVELOPMENT) AREA (ACRES)     /3.6/
 47 *                ESTIMATED USING GOOGLE EARTH AREA FOR CAVC PROPERTY
 48          a_i     REGIONAL INTENSITY "a" COEFFICIENT       /45.92/
 49          b_i     REGIONAL INTENSITY "b" COEFFICIENT       /10/
 50          n_i     REGIONAL INTENSITY "n" COEFFICIENT       /0.786/
 51 *                REGIONAL COEFFICIENTS COMPUTED FROM NOAA
 52 *                ATLAS 14 VALUES FOR PRECIPITATION RETURN PERIODS
 53 *
 54 *          GREEN-AMPT INFILTRATION SCALARS
 55 *          ***Soil Parameters for Green-Ampt model can be found on
 56 *                   pp.317 of 'Ground and Surface Water Hydrology' (Mays)
 57 *
 58 *          K       HYDRAULIC CONDUCTIVITY (IN PER HR)       /0.43/
 59 *          P       PSI = WETTING FRONT SUCTION HEAD (IN)    /4.33/
 60 *          nu      POROSITY                                 /0.453/
```

```
61  *      T_i     THETA_i = INITIAL SOIL MOISTURE CONTENT /0.3/
62  *   assumes sandy loam
63
64  *      K       HYDRAULIC CONDUCTIVITY (IN PER HR)      /0.26/
65  *      P       PSI = WETTING FRONT SUCTION HEAD (IN)   /6.57/
66  *      nu      POROSITY                               /0.501/
67  *      T_i     THETA_i = INITIAL SOIL MOISTURE CONTENT /0.3/
68  *   assumes silt loam
69
70         K       HYDRAULIC CONDUCTIVITY (IN PER HR)      /0.04/
71         P       PSI = WETTING FRONT SUCTION HEAD (IN)   /0.22/
72         nu      POROSITY                               /0.464/
73         T_i     THETA_i = INITIAL SOIL MOISTURE CONTENT /0.3/
74  *   assumes clay loam
75  *
76  *   EVAPOTRANSPIRATION PARAMETERS
77  *   Choose either Penman Monteith or Hargreaves-Samani
78  *
79  *   ASCE Penman Monteith
80  *      Rn      NET RADIATION (MJ PER M^2*HR)          /0.90/
81  *      G       SOIL HEAT FLUX DENSITY (MJ PER M^2*HR) /0/
82  *      Ta      MEAN HOURLY AIR TEMPERATURE (DEGREES C) /22.8/
83  *      u2      MEAN HOURLY WIND SPEED @ 2M (M PER SEC) /0/
84  *      es      SATURATION VAPOR PRESSURE (KPA)        /2.90/
85  *      ea      MEAN ACTUAL VAPOR PRESSURE (KPA)       /0.46/
86  *      Delta   SLOPE SAT VAPOR PRESS-TEMP (KPA PER C) /.1681/
87  *      Gamma   PSYCHROMETRIC CONSTANT (KPA PER C)     /.0672/
88  *      C_n     NUMERATOR CONSTANT (K-MM*S^3 PER MG*HR) /66/
89  *      C_d     DENOMINATOR CONSTANT (S PER M)         /0.25/
90  *      Kc      CROP COEFFICIENT                      /1/
91  *   parameters estimated from meteorological data from
92  *   Phoenix Sky Harbor Airport
93  *
94  *   Hargreaves-Samani
95         KT      EMPIRICAL COEFFICIENT                 /0.162/
96  *   0.162 for interior regions, 0.19 for coastal regions
97         Ra      EXTRATERRESTRIAL RADIATION (MM PER DAY) /16.35/
98  *   water depth equivalent, function of day of yr and latitude
99         Tmax    MAX DAILY AIR TEMP (DEGREES C)        /28.3/
100        Tmin    MIN DAILY AIR TEMP (DEGREES C)        /17.2/
101        Tavg    AVG DAILY AIR TEMP (DEGREES C)        /22.8/
102        Kc      CROP COEFFICIENT                      /.5/
103 *
104 *   DRY WELL FLOW RATE
105 *   parameters for sinlge dry well in bioretention basin
106        K_dw    HYDRAULIC CONDUCTIVITY (IN PER HR)    /.06/
107 *              average field saturated over depth of drywell
108        a_dw    RADIUS OF DRY WELL (FT)              /2/
109        D_aq    DEPTH TO AQUIFER FROM SURFACE (FT)   /100/
110        N_dw    NUMBER OF DRYWELLS                   /1/
111 *
112 *   TIME OF CONCENTRATION CALCULATION SCALARS
113        L_O     LENGTH OF OVERLAND FLOW (FT)         /400/
114        CN      WEIGHTED CURVE NUMBER FOR WATERSHED  /85/
115        S       AVERAGE SURFACE SLOPE (PERCENT)      /2.0/
116 *
117 *   DESIGN CRITERIA
118        FB      FREEBOARD ALLOWANCE (FT)             /1.0/
119        SL      SLOPE WIDTH AROUND BASIN PERIMETER (FT) /9/
120        DR      MAX TIME OF INUNDATION (DRAIN TIME) (HR)/36/  ;
```

```
121 *        allows for 1.0 feet of freeboard at the top of the
122 *        infiltration basin for flood risk mitigation
123 *        36 hr drain time required by Maricopa County
124 *
125 *----------------------------------------------------------------
126 *
127 VARIABLES
128          Z        COST IN DOLLARS (OBJECTIVE FUNCTION VARIABLE)   ;
129 *                 objective is to MINIMIZE the cost, Z, for the basin
130 *                 while meeting the flood control objective
131
132 POSITIVE VARIABLES
133          L        LENGTH OF INFILTRATION BASIN (FT)
134          W        WIDTH OF INFILTRATION BASIN (FT)
135          d        DEPTH OF INFILTRATION BASIN (FT)
136          i        RATIONAL METHOD INTENSITY (IN PER HR)
137          F_t      CUMULATIVE GREEN-AMPT INFILTRATION (IN)
138          Q_p      PEAK DISCHARGE (CFS)
139          Qtotvol  TOTAL VOLUME OF STORMWATER RUNOFF PRODUCED (CUBIC FT)
140          Qetvol   TOTAL VOLUME OF STORMWATER EVAPOTRANSPIRED (CUBIC FT)
141          Qdwvol   TOTAL VOLUME OF STORMWATER FLOW INTO DRYWELL (CUBIC FT)
142          Qinfvol  TOTAL VOLUME OF STORMWATER INFILTRATED (CUBIC FT)
143          T_m      CRITICAL (MAXIMUM) DURATION (MIN)
144          f_m      INFILTRATION RATE (IN PER HR)
145          ET_m     EVAPOTRANSPIRATION RATE (IN PER HR)
146          t_c      TIME OF CONCENTRATION (MIN)
147          T_tot    TOTAL TIME OF INUNDATION (MIN)
148          F_tot    TOTAL INFILTRATION UNTIL EMPTY (IN)
149          P_H      PSI + AVG PONDING DEPTH IN BASIN (IN)
150          H_dw     DEPTH OF DRY WELL (FT)
151          Q_dw     FLOW RATE FROM DRY WELL (CFS)
152          V_MOD    VOLUME OF RETENTION BASIN (CUBIC FT)   ;
153
154 EQUATIONS
155          V_req      VOLUME OF INFILTRATION BASIN REQUIRED
156          Q_dw_CALC  FLOW RATE FROM DRY WELL CALCULATION
157          i_RAT      RATIONAL METHOD RAINFALL INTENSITY CALCULATION
158          GA_INF     GREEN AMPT CUMULATIVE INFILTRATION
159          Tm_CALC    CRITICAL DURATION CALCULATION
160          fm_CALC    INFILTRATION RATE AT CRITICAL DURATION
161 *        ET_CALC_PM ASCE PENMAN-MONTEITH EVAPOTRANSPIRATION RATE
162          ET_CALC_HS HARGREAVES-SAMANI EVAPOTRANSPIRATION RATE
163          SQUARE     ENSURES INFILTRATION BASIN IS SQUARE
164          CONC       TIME OF CONCENTRATION CALC
165          Q_peak     PEAK DISCHARGE CALCULATION
166          OBJ        OBJECTIVE FUNCTION
167          INF_TOT    GREEN AMPT CUMULATIVE INFILTRATION TOTAL UNTIL EMPTY
168          Ttot_CALC  TOTAL TIME OF INUNDATION
169 *        following four equations for reference only
170          Qtot_CALC  CALC OF TOTAL VOLUME OF STORMWATER PRODUCED
171          Qet_CALC   CALC OF TOTAL VOLUME OF STORMWATER EVAPOTRANSPIRED
172          Qdw_CALC   CALC OF TOTAL VOLUME OF STORMWATER FLOW INTO DRYWELL
173          Qinf_CALC  CALC OF TOTAL VOLUME OF STORMWATER INFILTRATED
174          IN_LIM     INUNDATION TIME LIMITATION
175          P_H_CALC   CALC OF PSI + AVG PONDING DEPTH IN BASIN
176          VOL        CALC OF VOLUME OF RETENTION BASIN (FOR REFERENCE ONLY);
177
178
179 Q_dw_CALC.. Q_dw =E= (2*pi*(K_dw/43200)*(H_dw**2))/(LOG(H_dw/a_dw+
180          sqrt((H_dw/a_dw)**2+1))-sqrt((a_dw/H_dw)**2+1+a_dw/H_dw)) ;
```

```
181 *          calculation of flow rate from dry well using Glover solution
182
183 GA_INF.. F_t =E= (K)*(T_m/60)+(P+(d*12)/2)*(nu-T_i)*
184          LOG(1+(F_t/(((P+(d*12)/2)*(nu-T_i)))));
185 *        implicit solution to cumulative Green-Ampt infiltration equation
186 *        at critical duration [in]
187
188 fm_CALC..f_m =E= K*(1+(P+(d*12)/2)*(nu-T_i)/F_t);
189 *        calculation of infiltration rate at critical storm duration [in/hr]
190
191 *ET_CALC_PM..ET_m =E= (Kc/25.4)*(.408*Delta*(Rn-G)+Gamma*(C_n/(Ta+273))*
192 *               u2*(es-ea)) /(Delta+Gamma*(1+C_d*u2));
193 *        calculation of evapotranspiration rate given parmeters (in/hr)
194
195 ET_CALC_HS..ET_m =E= Kc*0.0135*KT*Ra*(Tmax-Tmin)**0.5*(Tavg+17.8)/(24*25.4);
196 *        calculation of ET using Hargreaves-Samani Method
197
198 Tm_CALC..(T_m*(1-n_i)+b_i)/(T_m+b_i)**(n_i+1)-((L*W)*(f_m+ET_m)/(12*3600)
199               +Q_dw*N_dw)/(2*C*A)=E=0;
200 *        implicit solution to critical storm duration (time to max detention)
201
202 i_RAT.. i =E= a_i/((T_m+b_i)**n_i);
203 *        solution to modified rational method rainfall intensity
204
205 V_req.. 60*Q_p*T_m-(L*W)*(F_t/12+ET_m*T_m/12)-60*T_m*Q_dw*N_dw =E= L*W*d;
206 *        calculation for volumetric requirement of infiltration basin
207 *        based on inflow, time of duration, time of concentration,
208 *        and infiltration and evapotranspiration based on area of the basin
209
210 P_H_CALC.. P_H =E= P+(d*12)/2 ;
211 *        calculation of suction head plus average ponding depth (reference only)
212
213 SQUARE.. L =E= W;
214 *        equation to constrain length and width forcing resulting basin
215 *        shape to be square
216
217 CONC..   t_c =E= (100*(L_O**0.8)*((1000/CN)-9)**0.7)/(1900*(S**0.5));
218 *        calculation for time of concentration (SCS Lag equation)
219 *        to be used in the volumetric requirement calculation above
220
221 Q_peak.. Q_p =E= C*i*A;
222 *        calculation for peak surface runoff
223
224 INF_tot.. F_tot =E= (K)*(T_tot/60)+(P+(d*12)/2)*(nu-T_i)*
225          LOG(1+(F_tot/(((P+(d*12)/2)*(nu-T_i)))));
226 *        implicit solution to Green-Ampt infiltration equation for total [in]
227
228 Ttot_CALC.. 60*Q_p*.5*((T_m-t_c)+(T_m+t_c)) =E=
229          L*W*(F_tot/12)+L*W*(ET_m*T_tot/12)+60*T_tot*Q_dw*N_dw ;
230 *        calculation of total time of inundation (min)
231
232 Qtot_CALC.. Qtotvol =E= 60*Q_p*.5*((T_m-t_c)+(T_m+t_c)) ;
233 *        calculation of the total volume of stormwater produced
234 *        in the design storm event (cubic ft) - for reference only
235
236 Qet_CALC.. Qetvol =E= L*W*(ET_m*T_tot/12);
237 *        calculation of total volume of stormwater evapotranspired (cubic ft)
238 *        for reference only
239
240 Qdw_CALC.. Qdwvol =E= 60*T_tot*Q_dw*N_dw;
```

```
241 *       calculation of total stormwater volume into drywell (cubic ft)
242 *       for reference only
243
244 Qinf_CALC.. Qinfvol =E= L*W*(F_tot/12);
245 *       calculation of total stormwater volume infiltrated (cubic ft)
246 *       for reference only
247
248 IN_LIM.. T_tot =L= 60*DR ;
249 *       limit on total time of inundation (drain time) (DR=24hrs)
250
251 OBJ..    Z =E= U_e*((L*W+2*SL**2+SL*L+SL*W)*(d+FB))+
252          (U_t*(S_t**(-2))+U_sh*(S_sh**(-2))+U_g*(S_g**(-2)))*((L*W)+(2*L+2*W)*
253          ((SL**2+(d+FB)**2)**.5)) + U_l*(L*W+4*SL**2+2*(SL*L+SL*W))+
254          N_dw*(U_dw+U_drill*H_dw);
255 *       objective function to minimize costs based on excavation,
256 *       plants/planting, drywell and drilling, and land costs
257
258 VOL.. V_MOD =E= L*W*d      ;
259 *       volume of basin from model (for reference only)
260
261 **limiting factors for basin dimensions
262 F_t.lo = 0.01 ;
263 L.lo = 20;
264 W.lo = 20;
265 L.up = 500;
266 W.up = 500;
267 d.lo = 1.5;
268 d.up = 3.0;
269 *max depth allowed is 3ft per Maricopa County
270 H_dw.lo = 20;
271 H_dw.up = D_aq - 10;
272
273 *-------------------------------------------------------------------
274
275 MODEL BIORETENTION_BASIN /ALL/;
276
277 *-------------------------------------------------------------------
278
279 SOLVE BIORETENTION_BASIN USING NLP MINIMIZING Z;
280
281 *-------------------------------------------------------------------
282
283 DISPLAY t_c.l, i.l, d.l, L.l, W.l, Z.l, ET_m.l, Q_p.l, T_m.l, f_m.l,
284         F_t.l, T_tot.l, P_H.l, V_MOD.l, Q_dw.l, H_dw.l,
285         Qtotvol.l, Qetvol.l, Qdwvol.l, Qinfvol.l ;
286
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APPENDIX C

GAMS MINLP MODEL FOR BIORETENTION BASIN OPTIMIZATION WITH

VARIABLE NUMBER OF DRY WELLS

```
 1 $ TITLE Optimization Model for Design of Bioretention Basins with Dry Wells
 2 *
 3 *        Optimization Model created by Mason Lacy for Master's Thesis
 4 *        Spring 2016, Arizona State University Hydrosystems Engineering
 5 *        Thesis Advisor: L.W. Mays
 6 *
 7 *        Model expanded from the optimization model developed for
 8 *        infiltration basins by Stafford et al. (2015)
 9 *
10 *        Problem: Optimize the size of a bioretention basin & depth & number
11 *                 of drywells given certain model constraints and parameters
12 *                 of the soil conditions, infiltration rate, meteorological
13 *                 data, vegetation parameters, drywell data and other necessary
14 *                 parameters.
15 *
16 *        ***All soil parameters, watershed properties, etc., are
17 *                 variable and freely editable by the user.
18 *
19 *-----------------------------------------------------------------
20 *
21 SCALARS
22 *        PARAMETERS FOR COST MINIMIZATION
23          U_e      UNIT COST OF EXCAVATION ($ PER CU.FT.)   /0.74/
24 *                 (BASED ON ~$20/CU.YD. - IDAHO AREA)
25          U_t      UNIT COST OF TREES ($ PER TREE)          /200/
26 *                 (TREES PLANTED AT 20FT SPACING)
27          U_sh     UNIT COST OF SHRUBS ($ PER SHRUB)        /50/
28 *                 (SHRUBS PLANTED AT 10FT SPACING)
29          U_g      UNIT COST OF GRASSES ($ PER GRASS UNIT)  /10/
30 *                 (GRASSES PLANTED AT 5FT SPACING)
31          U_l      UNIT COST OF LAND ($ PER SQ. FT.)        /0.6887/
32 *                 (BASED ON $30,000/AC - USDA 2013 CROPLAND, AVG. VALUE)
33          S_t      SPACING OF TREES (FT)                    /20/
34 *                 (TREE SPACING IN VEGETATED BASIN)
35          S_sh     SPACING OF SHRUBS (FT)                   /10/
36 *                 (SPACING OF SHRUB SPECIES IN VEGETATED BASIN)
37          S_g      SPACING OF GRASSES (FT)                  /5/
38 *                 (SPACING OF GRASS SPECIES IN VEGETATED BASIN)
39          U_dw     UNIT COST OF DRYWELL ($ PER DRY WELL)    /10000/
40 *
41          U_drill  COST OF DRILLING DRYWELL ($ PER FT)      /50/
42 *
43 *        MODDIFIED RATIONAL METHOD SCALARS
44          C        DIMENSIONLESS RUNOFF COEFFICIENT         /0.97/
45 *                 ESTIMATED FOR CAVG BASED ON GOOGLE EARTH IMAGERY
46          A        WATERSHED (DEVELOPMENT) AREA (ACRES)     /30/
47 *                 WATERSHED AREA FOR EXAMPLE 2
48          a_i      REGIONAL INTENSITY "a" COEFFICIENT       /45.92/
49          b_i      REGIONAL INTENSITY "b" COEFFICIENT       /10/
50          n_i      REGIONAL INTENSITY "n" COEFFICIENT       /0.786/
51 *                 REGIONAL COEFFICIENTS COMPUTED FROM NOAA
52 *                 ATLAS 14 VALUES FOR PRECIPITATION RETURN PERIODS
53 *
54 *        GREEN-AMPT INFILTRATION SCALARS
55 *        ***Soil Parameters for Green-Ampt model can be found on
56 *                 pp.317 of 'Ground and Surface Water Hydrology' (Mays)
57 *
58          K        HYDRAULIC CONDUCTIVITY (IN PER HR)       /0.43/
59          P        PSI = WETTING FRONT SUCTION HEAD (IN)    /4.33/
60          nt       POROSITY                                /0.453/
```

117

```
61          T_i     THETA_i = INITIAL SOIL MOISTURE CONTENT /0.3/
62 *    assumes sandy loam
63
64 *      K       HYDRAULIC CONDUCTIVITY (IN PER HR)     /0.26/
65 *      P       PSI = WETTING FRONT SUCTION HEAD (IN)  /6.57/
66 *      nu      POROSITY                               /0.501/
67 *      T_i     THETA_i = INITIAL SOIL MOISTURE CONTENT /0.3/
68 *    assumes silt loam
69
70 *      K       HYDRAULIC CONDUCTIVITY (IN PER HR)     /0.04/
71 *      P       PSI = WETTING FRONT SUCTION HEAD (IN)  /8.22/
72 *      nu      POROSITY                               /0.464/
73 *      T_i     THETA_i = INITIAL SOIL MOISTURE CONTENT /0.3/
74 *    assumes clay loam
75 *
76 *    EVAPOTRANSPIRATION PARAMETERS
77 *    Choose either Penman Monteith or Hargreaves-Samani
78 *
79 *    ASCE Penman Monteith
80      Rn      NET RADIATION (MJ PER M^2*HR)          /0.30/
81      G       SOIL HEAT FLUX DENSITY (MJ PER M^2*HR) /0/
82      Ta      MEAN HOURLY AIR TEMPERATURE (DEGREES C) /22.8/
83      u2      MEAN HOURLY WIND SPEED @ 2M (M PER SEC) /0/
84      es      SATURATION VAPOR PRESSURE (KPA)        /2.90/
85      ea      MEAN ACTUAL VAPOR PRESSURE (KPA)       /0.46/
86      Delta   SLOPE SAT VAPOR PRESS-TEMP (KPA PER C) /.1681/
87      Gamma   PSYCHROMETRIC CONSTANT (KPA PER C)     /.0672/
88      C_n     NUMERATOR CONSTANT (K*MM*S^3 PER MG*HR) /66/
89      C_d     DENOMINATOR CONSTANT (S PER M)         /0.25/
90      Kc      CROP COEFFICIENT                       /.2/
91 *    parameters estimated from meteorological data from
92 *    Phoenix Sky Harbor Airport
93 *
94 *    Hargreaves-Samani
95 *    KT      EMPIRICAL COEFFICIENT                  /0.162/
96 *    0.162 for interior regions, 0.19 for coastal regions
97 *    Ra      EXTRATERRESTRIAL RADIATION (MM PER DAY) /16.35/
98 *    water depth equivalent, function of day of yr and latitude
99 *    Tmax    MAX DAILY AIR TEMP (DEGREES C)         /30/
100 *    Tmin    MIN DAILY AIR TEMP (DEGREES C)         /10/
101 *    Tavg    AVG DAILY AIR TEMP (DEGREES C)         /20/
102 *    Kc      CROP COEFFICIENT                       /0.5/
103 *
104 *    DRY WELL FLOW RATE
105 *    parameters for drywells in bioretention basin
106     K_dw    HYDRAULIC CONDUCTIVITY (IN PER HR)     /.6/
107 *            average field saturated over depth of drywell
108     a_dw    RADIUS OF DRY WELL (FT)                /2/
109     D_aq    DEPTH TO AQUIFER FROM SURFACE (FT)     /100/
110     SP_dw   MINIMUM DRYWELL SPACING (FT)           /50/
111 *    spacing sets upper bound for integer variable N_dw
112 *
113 *    TIME OF CONCENTRATION CALCULATION SCALARS
114     L_0     LENGTH OF OVERLAND FLOW (FT)           /400/
115     CN      WEIGHTED CURVE NUMBER FOR WATERSHED    /85/
116     S       AVERAGE SURFACE SLOPE (PERCENT)        /2.0/
117 *
118 *    DESIGN CRITERIA
119     FB      FREEBOARD ALLOWANCE (FT)               /1.0/
120     SL      SLOPE WIDTH AROUND BASIN PERIMETER (FT) /9/
```

```
121          DR      MAX TIME OF INUNDATION (DRAIN TIME) (HR)/36/ ;
122 *        allows for 1.0 feet of freeboard at the top of the
123 *        infiltration basin for flood risk mitigation
124 *        36 hr drain time required by Maricopa County
125 *
126 *
127 *----------------------------------------------------------------
128 *
129 VARIABLES
130          Z       COST IN DOLLARS (OBJECTIVE FUNCTION VARIABLE)    ;
131 *                objective is to MINIMIZE the cost, Z, for the basin
132 *                while meeting the flood control objective
133
134 POSITIVE VARIABLES
135          L       LENGTH OF INFILTRATION BASIN (FT)
136          W       WIDTH OF INFILTRATION BASIN (FT)
137          d       DEPTH OF INFILTRATION BASIN (FT)
138          i       RATIONAL METHOD INTENSITY (IN PER HR)
139          F_t     CUMULATIVE GREEN-AMPT INFILTRATION (IN)
140          Q_p     PEAK DISCHARGE (CFS)
141          Qtotvol TOTAL VOLUME OF STORMWATER RUNOFF PRODUCED (CUBIC FT)
142          Qetvol  TOTAL VOLUME OF STORMWATER EVAPOTRANSPIRED (CUBIC FT)
143          Qdwvol  TOTAL VOLUME OF STORMWATER FLOW INTO DRYWELL (CUBIC FT)
144          Qinfvol TOTAL VOLUME OF STORMWATER INFILTRATED (CUBIC FT)
145          T_m     CRITICAL (MAXIMUM) DURATION (MIN)
146          f_m     INFILTRATION RATE (IN PER HR)
147          ET_m    EVAPOTRANSPIRATION RATE (IN PER HR)
148          t_c     TIME OF CONCENTRATION (MIN)
149          T_tot   TOTAL TIME OF INUNDATION (MIN)
150          F_tot   TOTAL INFILTRATION UNTIL EMPTY (IN)
151          P_H     PSI + AVG PONDING DEPTH IN BASIN (IN)
152          H_dw    DEPTH OF DRY WELL (FT)
153          Q_dw    FLOW RATE FROM DRY WELL (CFS)
154          V_MOD   VOLUME OF RETENTION BASIN FROM MODEL(CUBIC FT)        ;
155
156 INTEGER VARIABLE N_dw NUMBER OF DRY WELLS IN BASIN ;
157
158 EQUATIONS
159          V_req       VOLUME OF INFILTRATION BASIN REQUIRED
160          Q_dw_CALC   FLOW RATE FROM DRY WELL CALCULATION
161          N_dw_MAX    ENSURES MIN DRYWELL SPACING IS NOT VIOLATED
162          i_RAT       RATIONAL METHOD RAINFALL INTENSITY CALCULATION
163          GA_INF      GREEN AMPT CUMULATIVE INFILTRATION
164          Tm_CALC     CRITICAL DURATION CALCULATION
165          fm_CALC     INFILTRATION RATE AT CRITICAL DURATION
166          ET_CALC_PM ASCE PENMAN-MONTEITH EVAPOTRANSPIRATION RATE
167 *         ET_CALC_HS HARGREAVES-SAMANI EVAPOTRANSPIRATION RATE
168          SQUARE      ENSURES INFILTRATION BASIN IS SQUARE
169          CONC        TIME OF CONCENTRATION CALC
170          Q_peak      PEAK DISCHARGE CALCULATION
171          OBJ         OBJECTIVE FUNCTION
172          INF_TOT     GREEN AMPT CUMULATIVE INFILTRATION TOTAL UNTIL EMPTY
173          Ttot_CALC   TOTAL TIME OF INUNDATION
174 *        following four equations for reference only
175          Qtot_CALC   CALC OF TOTAL VOLUME OF STORMWATER PRODUCED
176          Qet_CALC    CALC OF TOTAL VOLUME OF STORMWATER EVAPOTRANSPIRED
177          Qdw_CALC    CALC OF TOTAL VOLUME OF STORMWATER FLOW INTO DRYWELL
178          Qinf_CALC   CALC OF TOTAL VOLUME OF STORMWATER INFILTRATED
179          IN_LIM      INUNDATION TIME LIMITATION
180          P_H_CALC    CALC OF PSI + AVG PONDING DEPTH IN BASIN
```

```
181           VOL        CALC OF VOLUME OF RETENTION BASIN (FOR REFERENCE ONLY);
182
183
184 Q_dw_CALC.. Q_dw =E= (2*pi*(K_dw/43200)*(H_dw**2))/(LOG(H_dw/a_dw+
185              sqrt((H_dw/a_dw)**2+1))-sqrt((a_dw/H_dw)**2+1+a_dw/H_dw)) ;
186 *           calculation of flow rate from dry well using Glover solution
187
188 N_dw_MAX.. N_dw =L= (L*W)/(4*SP_dw**2);
189 *           maximum number of drywells so that minimum spacing is not violated
190
191 GA_INF.. F_t =E= (K)*(T_m/60)+(P+(d*12)/2)*(nu-T_i)*
192          LOG(1+(F_t/(((P+(d*12)/2)*(nu-T_i)))));
193 *           implicit solution to cumulative Green-Ampt infiltration equation
194 *           at critical duration [in]
195
196 fm_CALC..f_m =E= K*(1+(P+(d*12)/2)*(nu-T_i)/F_t);
197 *           calculation of infiltration rate at critical storm duration [in/hr]
198
199 ET_CALC_PM..ET_m =E= (Kc/25.4)*(.408*Delta*(Rn-G)+Gamma*(C_n/(Ta+273))*
200              u2*(es-ea)) /(Delta+Gamma*(1+C_d*u2));
201 *           calculation of evapotranspiration rate given parmeters (in/hr)
202
203 *ET_CALC_HS..ET_m =E= Kc*0.0135*KT*Ra*(Tmax-Tmin)**0.5*(Tavg+17.8)/(24*25.4);
204 *           calculation of ET using Hargreaves-Samani Method
205
206 Tm_CALC..(T_m*(1-n_i)+b_i)/(T_m+b_i)**(n_i+1)-((L*W)*(f_m+ET_m)/(12*3600)
207              +Q_dw*N_dw)/(2*C*A)=E=0;
208 *           implicit solution to critical storm duration (time to max detention)
209
210 i_RAT..  i =E= a_i/((T_m+b_i)**n_i);
211 *           solution to modified rational method rainfall intensity
212
213 V_req..  60*Q_p*T_m-(L*W)*(F_t/12+ET_m*T_m/12)-60*T_m*Q_dw*N_dw =E= L*W*d;
214 *           calculation for volumetric requirement of infiltration basin
215 *           based on inflow, time of duration, time of concentration,
216 *           and infiltration and evapotranspiration based on area of the basin
217
218 P_H_CALC.. P_H =E= P+(d*12)/2 ;
219 *           calculation of suction head plus average ponding depth (reference only)
220
221 SQUARE.. L =E= W;
222 *        equation to constrain length and width forcing resulting basin
223 *        shape to be square
224
225 CONC..   t_c =E= (100*(L_O**0.8)*((1000/CN)-9)**0.7)/(1900*(S**0.5));
226 *           calculation for time of concentration (SCS Lag equation)
227 *           to be used in the volumetric requirement calculation above
228
229 Q_peak.. Q_p =E= C*i*A;
230 *           calculation for peak surface runoff
231
232 INF_tot.. F_tot =E= (K)*(T_tot/60)+(P+(d*12)/2)*(nu-T_i)*
233          LOG(1+(F_tot/(((P+(d*12)/2)*(nu-T_i)))));
234 *           implicit solution to Green-Ampt infiltration equation for total [in]
235
236 Ttot_CALC.. 60*Q_p*.5*((T_m-t_c)+(T_m+t_c)) =E=
237          L*W*(F_tot/12)+L*W*(ET_m*T_tot/12)+60*T_tot*Q_dw*N_dw ;
238 *           calculation of total time of inundation (min)
239
240 Qtot_CALC.. Qtotvol =E= 60*Q_p*.5*((T_m-t_c)+(T_m+t_c)) ;
```

```
241 *          calculation of the total volume of stormwater produced
242 *          in the design storm event (cubic ft) - for reference only
243
244 Qet_CALC.. Qetvol =E= L*W*(ET_m*T_tot/12);
245 *          calculation of total volume of stormwater evapotranspired (cubic ft)
246 *          for reference only
247
248 Qdw_CALC.. Qdwvol =E= 60*T_tot*Q_dw*N_dw;
249 *          calculation of total stormwater volume into drywell (cubic ft)
250 *          for reference only
251
252 Qinf_CALC.. Qinfvol =E= L*W*(F_tot/12);
253 *          calculation of total stormwater volume infiltrated (cubic ft)
254 *          for reference only
255
256 IN_LIM.. T_tot =L= 60*DR ;
257 *          limit on total time of inundation (drain time) (DR=24hrs)
258
259 OBJ..     Z =E= U_e*((L*W+2*SL**2+SL*L+SL*W)*(d+FB))+
260           (U_t*(S_t**(-2))+U_sh*(S_sh**(-2))+U_g*(S_g**(-2)))*((L*W)+(2*L+2*W)*
261           ((SL**2+(d+FB)**2)**.5)) + U_l*(L*W+4*SL**2+2*(SL*L+SL*W))+
262           N_dw*(U_dw+U_drill*H_dw);
263 *          objective function to minimize costs based on excavation,
264 *          plants/planting, drywell and drilling, and land costs
265
266 VOL.. V_MOD =E= L*W*d     ;
267 *          volume of basin from model (for reference only)
268
269 **limiting factors for basin dimensions
270 F_t.lo = 0.01 ;
271 L.lo = 20;
272 W.lo = 20;
273 L.up = 500;
274 W.up = 500;
275 d.lo = 1.5;
276 d.up = 3.0;
277 *max depth allowed is 3ft per Maricopa County
278 H_dw.lo = 20;
279 H_dw.up = D_aq - 10;
280
281 *-------------------------------------------------------------------
282
283 MODEL BIORETENTION_BASIN /ALL/;
284
285 *-------------------------------------------------------------------
286
287 SOLVE BIORETENTION_BASIN USING MINLP MINIMIZING Z;
288
289 *-------------------------------------------------------------------
290
291 DISPLAY t_c.l, i.l, d.l, L.l, W.l, Z.l, ET_m.l, Q_p.l, T_m.l, f_m.l,
292         F_t.l, T_tot.l, P_H.l, V_MOD.l, Q_dw.l, H_dw.l,
293         Qtotvol.l, Qetvol.l, Qdwvol.l, Qinfvol.l, N_dw.l ;
294
```

APPENDIX D

PHOTOGRAPHS OF BIORETENTION AND DRY WELLS