ABSTRACT

Title of Document: THE DEVELOPMENT, CALIBRATION, AND USE OF A SPATIO-TEMPORAL MODEL FOR THE DESIGN AND EVALUATION OF CONSTRUCTED WETLANDS BASED ON SUSTAINABLITY METRICS

Jennifer Marie Olszewski, Doctorate of Philosophy, 2014

Directed By: Dr. Richard H. McCuen, Department of Civil and Environmental Engineering

The focus of this research was the development of a spatio-temporal model of a constructed wetland that can be used to evaluate policy elements and design practices from the perspective of wetland sustainability. The model was calibrated with data obtained from a wetland that treats runoff from an agricultural field. Sustainability metrics were developed to reflect an array of wetland functions including wildlife habitat, flood control, downstream hydrologic regime, wetland water balance, groundwater recharge and baseflow maintenance, aesthetics, and water quality functions. The model can be optimized by the user across this array of wetland functions, each of which was defined in terms of metrics relevant to sustainability. Stakeholders will be able to weight the metrics for each of these wetland functions in order to maximize sustainability for their specific goals. Optimally, this model will aid design engineers and policy makers in designing constructed wetlands as a function of necessary functions, location, and influent water quantity and quality characteristics.

THE DEVELOPMENT, CALIBRATION, AND USE OF A SPATIO-TEMPORAL MODEL FOR THE DESIGN AND EVALUATION OF CONSTRUCTED WETLANDS BASED ON SUSTAINABLITY METRICS

By

Jennifer Marie Olszewski

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Advisory Committee:
Professor Richard H. McCuen, Chair
Professor Kaye Brubaker
Professor Allen P. Davis
Professor Barton Forman
Professor Adel Shirmohammadi

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Chapter 1: Introduction and Background

1.1 THE NEED FOR CONSTRUCTED WETLANDS

Many studies have shown that urban development negatively impacts both the hydrology and water quality of surrounding streams and other natural water bodies (Barco et al. 2008). High energy runoff causes flashy hydrographs and is the source of many of the symptoms associated with the "Urban Stream Syndrome," which includes channelization of streams and rivers, increased nutrient loadings, and decreased biotic diversity (Walsh et al. 2005). While the reduction of stream meanders limits areas of denitrification, increased nutrient loadings can further increase phosphorous and nitrogen levels. Agricultural runoff has also been cited as a large contributor of non-point source pollution to natural waterways (Kadlec and Knight 1996). Excess nutrients from fertilizers and animal waste runoff promote eutrophic aquatic ecosystems downstream. Eutrophication, in turn, can reduce biodiversity and overall natural water health (Beman et al. 2005).

Natural treatment systems, including constructed wetlands, offer a more sustainable solution to wastewater and stormwater runoff treatment than current conventional facilities such as detention ponds and wastewater treatment plants (Kadlec and Knight 1996; Campbell and Ogden 1999). A wetland is an area of land with saturated soil, either permanently or seasonally, sustaining plant species that grow well under such conditions. These areas of saturated soil generally represent the transition from land to open water in a landscape. Both constructed and natural wetlands serve crucial functions including flood control, erosion control, groundwater recharge, nutrient cycling, pollutant retention, and food chain support (Finlayson and Moser 1991). Wetlands can be used as buffers between urban areas and aquatic ecosystems, redistributing runoff in time and space and treating runoff before it reaches natural streams. Constructed wetlands are wetlands that have been specifically designed to treat a number of different types of water, such as urban and agricultural runoff, municipal, industrial, and acid mine drainage (USEPA 2000). They can be classified as either surface-flow or sub-surface flow wetlands (Mitsch and Gosselink 2007).

The effectiveness of a wetland is greatly influenced by its design. Current design methods often use overly simplified indices to determine the physical characteristics of a proposed wetland. An inadequate design will cause the wetland to fail to serve its intended functions, with its long-term sustainability unmet. This study will focus mainly on modeling surface-flow constructed wetlands that are designed to treat municipal wastewater, urban and agricultural runoff, as well as to provide habitat.

PROBLEM:	Constructed wetlands could be a sustainable treatment solution to runoff and wastewater effluent. Current generalized design criteria, however, are overly simplified such that non-optimal designs can result.

1.2 CONSTRUCTED WETLAND MODELING

While the EPA has very specific water quality based design guidelines for wastewater treatment wetlands, the scientific basis for urban runoff water quality wetland design, as defined by the Maryland Department of the Environment (MDE), is not as well defined (USEPA 2000; Hayes et al. 2000; MDE 2009). Conversely, water quantity control is not greatly accounted for in EPA wastewater wetland design, while the Army Corps of Engineers (USACE) and the Maryland Department of the Environment (MDE) concentrate heavily on controlling water quantity and overall hydrology (USEPA 2000; Hayes et al. 2000; MDE 2009). A number of MDE specifications regarding the water depth requirements and wetland spatial organization are not visibly supported by any research or theory. Additionally, the origin and effects of a number of literature design guidelines remain untestested and not well-understood. Therefore, a more comprehensive approach to wetland design is needed. Specifically, the designer or policy maker needs more in-depth knowledge of the effect of design decisions.

A number of wetland models have been developed in order to better understand and predict wetland behavior. Kumar and Zhao (2011) identified two primary model types in the literature, black-box models and process-based models. They defined black-box models as simplistic models that used general rate constants to relate the influent and effluent pollutant concentrations of a wetland. Some examples include first-order models, regression models, time-dependent retardant models, tanks-in-series (TIS) models, Monod models, neural networks, and statistical methods (Kumar and Zhao 2011). The most extensively used black-box model is the first-order model or the k*C model (Kadlec and Knight 1996; Kumar and Zhao 2011). However, Kadlec (2000) found first-order models to be inadequate for modeling wetland behavior, emphasizing the need for a better understanding of internal wetland hydraulics. This study also found that first-order, plug-flow models only accurately described the data with which they were calibrated and did not perform well with different data sets in the same range (Kadlec 2000).

Process-based models focus on modeling the physical processes within a wetland in order to better understand water movement and pollutant behavior (Kumar and Zhao 2011). A number of process-based models define transformation and degradation processes using the Monod equation, which by itself is a black-box model. While process-based models can provide more insight to wetland processes, more detailed data collection would also be required to validate and calibrate such models (Kumar and Zhao 2011). One study also suggested that more intensive wetland monitoring could provide necessary data to create better, process-based models, allowing the modeling of multifunction wetlands (Ng and Eheart 2008). Therefore, a need exists for more detailed wetland models with adequate data to calibrate them. The literature, however, does not give any guidance on exactly what data are needed and how much data are necessary to accurately calibrate a process-based model. It would be of value to know the marginal benefit of additional measured data. This leads to the question: How accurate can wetland designs be if we currently lack data to properly calibrate the models? A lack of data is certainly a source of design uncertainty. Modeling can provide some idea of the value of data, especially the marginal benefit of additional data.

PROBLEM: Existing wetland models are overly simplified to effectively model spatio-temporal processes and the connection between the database required to calibrate intensive process-based models and their accuracy is not well-defined.

1.3 SUSTAINABILITY

Models that connect ecosystem sustainability and wetland performance metrics were not found in the literature. Groffman et al. (2006) stressed the need for establishing a connection between ecosystem functions (i.e., water quality improvement, flood control, etc.) and the contributing ecosystem components and processes. Long-term wetland performance is very much tied to the general sustainability of the facility. If a constructed wetland ecosystem degrades below the intended state for which it was designed, it will not perform the intended ecosystem functions (i.e., water quality improvement, etc.). Therefore, if one design component fails to function properly, that wetland ecosystem function will be impaired and also impair the overall wetland's ability to perform other functions. For a wetland to maintain its effectiveness the components of the wetland must be located and sized using long-term sustainability criteria.

PROBLEM: Current design guidelines do not address long-term wetland sustainability criteria.

1.4 MODEL CALIBRATION

Model calibration is crucial to the development of a reliable and useful model. If multi-function constructed wetlands are going to be effective over the long-term, they must be designed based on sustainability criteria. This will require accurate assessments of the connections between sustainability metrics and measured long-term effectiveness criteria. These connections are possible only through calibrating the model with measured data. The process of fitting the model to measured data enhances its reliability. However, the development of models should not wait until extensive data are available, as concept-based, physical process models can be used to identify the type of data needed and the value of larger sample sizes at appropriate time and space scales.

PROBLEM: The failure to quantitatively connect the relationship between wetland inputs and outputs will lead to inaccurate designs.

1.5 UNCERTAINTY AND SENSITIVITY ANALYSES

Both uncertainty and sensitivity analyses are ubiquitous tools in the environmental field. Studies, however, have not approached uncertainty and sensitivity from the perspective of a system's defined sustainability. While sensitivity analysis is a common univariate tool for determining the relevance of single input to output values, it does not provide a measure of the expected variation of an effect. It is congruent to knowing a mean value but not understanding the accuracy of the mean. Uncertainty analysis is an alternative to sensitivity analysis that produces a measure of the range of a criterion. It could be used to evaluate both existing design criteria (e.g., the percentage of the wetland devoted to the macropool) and long-term sustainability metrics. Knowing the uncertainty associated with model input parameters allows the model user to judge the faith than can be placed in the component design.

PROBLEM: Significant design inaccuracy can result from uncertainties in wetland model inputs and components. Assessing uncertainty in the context of sustainability metrics has not been reported in the literature.

1.6 SUMMARY OF PROBLEMS

Current wetland design models are inadequate because they do not address problems such as the following:

- Wetland design criteria do not necessarily reflect long-term sustainability.
- Wetland design methods are often overly simplified and not based on systematic assessments.
- The current design methods are insufficiently sophisticated to enable the design engineer to understand how changing individual components of the wetland of a proposed design can influence overall wetland effectiveness.
- Better designs of constructed wetlands can be achieved by understanding the uncertainty and sensitivity of outputs and components of the wetland.

1.7 RESEARCH GOALS AND OBJECTVES

The aim of the current study was to develop a model to simulate wetland design conditions to meet the objectives of an array of stakeholders and to determine the most sustainable wetland design that would meet their needs. The model allows different types of constructed wetlands to be represented and allows for the analysis of unique wetland designs with differing function combinations. While a farmer may be most concerned with nitrogen and phosphorus control, wildlife organizations may place higher value on the wildlife habitat adjacent to the farm. A well-designed wetland can address the concerns of these conflicting stakeholders. The proposed model will be able to identify an optimal wetland design that balances the objectives of various stakeholders.

The overall goal and objectives of this research are listed as follows:

Goal: To model and evaluate the sensitivity of the main functions of constructed wetlands using sustainability metrics as the primary criteria.

Objectives:

1. To formulate a long-term spatio-temporal model of a multipurpose constructed wetland, which includes relevant processes and components of different types of wetland systems.

2. To define sustainability as it applies to wetlands and to develop metrics based on sustainability principles that connect wetland design with intended wetland functions.

3. To calibrate the model using both hydrologic and water quality data from an actual wetland.

4. To quantify the sensitivity and uncertainty associated with each design function and contributing variable.

5. To evaluate the reliability of design criteria currently used in the design of wetlands and assess whether or not they lead to sustainable designs.

Chapter 2: Literature Review

This section is a detailed compilation of the current and relevant wetland, sustainability, and uncertainty literature. It begins by defining each type of wetland to be incorporated in the model as well as their respective hydrologic and water quality performance. Information collected on modeling wetland hydrology and chemistry is included next. Sustainability as it pertains to wetland performance is then discussed. Finally, the uncertainty analysis literature is reviewed.

2.1 DEFINING WETLANDS AND THEIR FUNCTIONS

A wetland is an area of land with saturated soil either permanently or seasonally, sustaining plant species that grow well under such conditions (Kadlec and Knight 1996; Trimble 2008). These areas of saturated soil generally represent the transition from land to water in a landscape. Both constructed and natural wetlands serve crucial functions including flood control, erosion control, nutrient cycling, pollutant retention, and food chain support (Finlayson and Moser 1991). As a result, wetlands can be used as buffers between urban areas and aquatic ecosystems, redistributing and purifying runoff before it reaches natural streams. Constructed wetlands are wetlands that have been specifically designed to treat a number of different types of water, such as urban and agricultural runoff, municipal, industrial, and acid mine drainage (USEPA 2000). Constructed wetlands can be classified as either surface-flow or sub-surface flow wetlands (Mitsch and Gosselink 2007). This study will focus mainly on surface-flow wetlands. Habitat wetlands are also a type of constructed wetland created to provide wildlife habitat (Kadlec and Knight 1996).

The constructed wetlands of concern for this study are municipal wastewater wetlands, urban and agricultural runoff wetlands, and habitat wetlands. The current study will incorporate design components from these different types of constructed wetlands into one general model.

The Army Corps of Engineers (USACE) emphasized the importance of designing a wetland with the site characteristics, and its desired functions in mind (Hayes et al. 2000). Depending on its location, a site may not be able to perform a given function. For example, a site with a very low water table and low permeable soil will not provide significant groundwater recharge or baseflow maintenance. The USACE used specific design criteria to achieve the desired functions for a wetland. Design criteria were defined as quantitative measures of wetland component services, and divided into four categories, biological, hydrologic, geotechnical, and engineering design (Hayes et al. 2000).

Biological criteria include vegetation type and corresponding water depth, shoreline slopes, media type, and nutrient demands (Hayes et al. 2000). Because biology can change dramatically from site to site, there are few broad guidelines for biological criteria. The USACE used nine main hydrologic criteria for design, including (1) hydrologic setting, (2) flooding duration and timing, (3) flooding depth, (4) flow velocities, (5) flow resistance, (6) hydraulic retention time (HRT), (7) storage capacity, (8) surface area, and (9) wind fetch (Hayes et al. 2000). Seven specific geotechnical criteria were also taken into account, (1) geological setting, (2) geomorphic setting, (3) wetland form and size, (4) soil composition and texture, (5) hydrogeologic processes, (6) geomorphic processes, and (7) geomorphic trends (Hayes et al. 2000).

The USACE also divided wetland functions into three categories, (1) hydrology, (2) water quality, and (3) life support. Some functions may be compatible while others are conflicting (Hayes et al. 2000). Table 2-1 further defines wetland functions and their interactions. Hydrologic functions include groundwater recharge, groundwater discharge (movement of water from groundwater to surface water), flood flow alteration (temporary storage, volume and flow reduction), and shoreline stabilization. Sediment/Toxicant retention and nutrient removal/transformation are the main wetland water quality functions. Because wetlands are ecosystems they also provide life support to a number of organisms. The main life support functions are production export (vegetation and organic material leaving the wetland), aquatic diversity and abundance, and wildlife (birds, mammals, amphibians, reptiles) diversity and abundance (Hayes et al. 2000).

2.2 CURRENT WETLAND DESIGN GUIDELINES

The following section outlines the design procedures and criteria for stormwater management constructed wetlands as defined by the Maryland Department of the Environment (MDE), for municipal wastewater treatment constructed wetlands as defined by the United States Environmental Protection Agency (USEPA), agricultural wastewater treatment wetlands as specified by the National Resources Conservation Service (NRCS), and wildlife habitat constructed wetland as designed by the NRCS

Table 2-1 General wetland functions and their interactions as defined by the U.S. Army Corps of Engineers (USACE), where * indicates two functions are compatible, 0 indicates no known interaction, and X suggests a probable conflict (Hayes et al. 2000).

Function	Interaction with								
	Hydrologic				Water Quality		Life Support		
	Groundwater Recharge	Groundwater Discharge	Flood flow Alteration	Shoreline Stabilization	Sediment/ Toxicant Retention	Nutrient Removal/ Transformation	Production Export	Aquatic Diversity/ Abundance	Wildlife Diversity/ Abundance
Groundwater Recharge		0	*	0	X	*	0	0	0
Groundwater Discharge	0		X	X	X	0	*	*	*
Flood flow Alteration	*	X		*	*	*	*	0	0
Shoreline Stabilization	0	X	*		*	*	0	X	X
Sediment/Toxicant Retention	0	0	*	*		*	0	X	X
Nutrient Removal/ Transformation	*	0	*	*	*		X	0	0
Production Export	X	*	0	0	0	0		*	0
Aquatic Diversity/ Abundance	X	*	*	0	0	0	0		*
Wildlife Diversity/ Abundance	X	*	*	*	0	0	0	0	

2.2.1 MDE Stormwater Treatment Wetlands

Urban stormwater is generally high in metals content due to vehicle emissions and tire and brake wear; as well as high in organic content due to leaves, wood litter, and road materials. Urban stormwater can also contain high nutrient loads due to fertilizer use as well as seasonal senescence (Kadlec and Knight 1996). Urban runoff also requires volume and peak flow reduction due to minimal infiltration in impervious surfaces. These wetlands are often limited by available space in urban areas (Kadlec and Knight 1996).

As specified by the Maryland Department of the Environment (MDE), stormwater constructed wetlands are designed to hold enough runoff to prevent flooding downstream, to promote settling of TSS, and to reduce peak flows. Nutrient control is not well defined by the MDE. Typical components of a stormwater wetland are an inlet, a forebay, high (< 6-in. water depth) and low (6-18-in. water depth) marsh areas, a wet pond, an emergency spillway, a micropool, an outlet, and a wetland buffer that extends 25-ft from the maximum water level, and an additional 15-ft from any buildings or structures. The wetland surface area should be 1 to 2% of the drainage area and water depths should be allocated accordingly, a minimum of 35% of surface area must be less than or equal to 6 in. and at least 65% of it must be less than or equal to 18 in. to promote sustainable wetland vegetation (MDE 2009). Therefore at least 30% of the surface area should be designated to depths between 6 in. and 18 in. In general, the wetland walls should have a slope of 3:1 or flatter for erosion control purposes. Additionally, the overall wetland length-to-width ratio must be at least 1.5:1 to avoid short-circuiting (MDE 2009).

MDE also specifies a number of design parameters that are applicable to all stormwater best management practices. These design parameters are given in designated volumes, water quality volume (WQ_v), recharge volume (Re_V), channel protection storage volume (Cp_V), overbank flood protection volume (Qp), and extreme flood volume (Qf). The water quality volume WQ_v (ac-ft) is the storage required to capture and treat runoff from 90% of the average annual rainfall (42-in. for Maryland) and is defined by the following equation (MDE 2009):

$$WQ_v = \frac{P \cdot R_v \cdot DA}{12} \tag{2-1}$$

where P is the precipitation depth, which is set to 1 in. in the Eastern Rainfall Zone, R_v is the volumetric runoff coefficient, and DA is the drainage area (acres). R_v was defined as (MDE 2009):

$$R_v = 0.05 + 0.009 \cdot I \tag{2-2}$$

where I represents the percent of imperviousness (%) of the drainage area. Two methods were used to estimate the required amount of water to be infiltrated from a given design site. Water can be infiltrated via structural or non-structural methods. Structural methods use the wetland and/or other designed BMP facilities such as infiltration trenches to infiltrate water from the drainage area. Non-structural methods infiltrate water within the drainage area by routing water from impervious surfaces to grass channels, filter strips, stream buffers, etc. before it reaches the constructed wetland. If water is infiltrated within the wetland, a percent volume method is used to calculated the volume of water that must infiltrate down to groundwater within the wetland (MDE 2009):

$$\mathrm{Re}_v = \frac{S \cdot R_v \cdot DA}{12} \tag{2-3}$$

where S is the soil specific recharge factor (values listed in Table 2-2), and Re_v is the required recharge volume (ac-ft). If Re_v is to be treated non-structurally, the percent area method is used (MDE 2009):

$$\mathrm{Re}_v = S \cdot A_i \tag{2-4}$$

where A_i is the impervious area cover (acres), and in this case Re_v is equal to the area (acres) of non-structural treatment that must be provided for infiltration of water from the impervious surfaces within the drainage area.

Table 2-2 NRCS soil groups and their corresponding soil specific recharge factors (MDE 2009). Recharge factors are dimensionless fractions.

Soil Group	**Soil Specific Recharge Factor (S)**
A	0.38
B	0.26
C	0.13
D	0.07

The Cp_v, Q_p, and Q_f volumes are chosen based on desired storm size control. The Cp_v is typically equivalent to the volume produced by the drainage area during a 1-yr, 24-hr storm. The corresponding detention time for the Cp_v volume is determined based on the water use designation of downstream waters (given in Table 2-3). Wetlands that drain into waters with trout require a 12-hr detention time in order to maintain acceptable water temperatures (MDE 2009). Conversely, wetlands serving general water uses such as waters designated for recreational use or nontidal warmwater aquatic life could drain the Cp_v volume over a 24-hr period.

Table 2-3 Different categories of water use and their corresponding maximum allowable extended detention times.

Water Use	Maximum Allowable of Extended Detention (hrs)
Use I (general)	24
Use II (tidal)	N/A (if direct discharge)
Use III (reproducing trout)	12
Use IV (recreational trout)	12

Both the Q_p and Q_f volume controls may vary depending on the requirements of the local authorities. However, the Q_p often represents the volume required to reduce the peak discharge rate for a 10-yr, 24-hr storm from the drainage area to pre-development values. The Q_f generally controls up to a 100-yr, 24hr flood if construction is allowed in the 100-yr floodplain and downstream conveyance is not adequate at such flows (MDE 2009). The MDE uses TR-55 to determine all relevant post- and pre-development flows associated with the calculation of Cp_v, Q_p, and Q_f and their corresponding weir and orifice configurations.

MDE also requires specific design criteria for certain wetland components. The inlet to the wetland must be 6 in. below the source collection site, and it should not be submerged at normal pool levels. All stormwater wetlands are also required to have a forebay and a micropool, which are settling pools at the inlet and before the outlet of the wetland. The forebay should be at least 3 ft deep and should be sized at 0.1-in./acre of drainage area. The micropool should be about 3 to 6 ft deep, account for about 5% of the wetland surface area, and be designed to drain within 24 hours of a storm event. If extended detention is included in the design, it should account for 50% of the wetland surface area and should have a water depth no higher than 3-ft above the normal pool (MDE 2009).

2.2.2 USEPA Municipal Wastewater Treatment Wetlands

Municipal wastewater treatment wetlands are designed to reduce pollutant levels according to wastewater effluent regulations (USEPA 2000). These wetlands are used to treat primary-treated municipal wastewater effluent, which generally has higher pollutant loads than stormwater. As a result, the goals of a wastewater treatment wetland are to remove harmful bacteria and viruses, reduce nutrient loads, reduce BOD and TSS, and to remove any other pollutants (USEPA 2000). Downstream hydrology is not a main design factor for these wetlands.

According to USEPA, a treatment wetland consists of three consecutive zones (see Figure 2-1). The first and third zones are identical, both with a depth of 2 to 3ft and heavily vegetated with emergent vegetation. Ideally, these zones are anaerobic, i.e., have very low oxygen levels. Zone 2 has a water depth of 4 to 5 ft with submerged and floating vegetation, which promote reaeration and aerobic conditions. Soil in all zones is generally anaerobic. Zone 1 promotes ammonification, flocculation, sedimentation, and minimal BOD reduction, while zone 2 allows for further BOD removal. Nitrification, and sedimentation and denitrification are the primary functions of zone 3 (USEPA 2000). If flow is laminar, settling will be optimized. Dense vegetation and controlled influent flow can help reduce velocities, allowing for laminar flow (USEPA 2000). These zones are not physically separated by berms or other structures. Therefore, in reality, the treatment process is not strictly sequential. USEPA (2000) suggested that separating these zones into individual cells with outlet controls could also promote greater wetland performance and performance control.

In general, average (Q_{avg}) and maximum (Q_{max}) primary effluent discharge rates are used to design treatment wetlands. For calculation purposes Q_{max} is assumed to be $2Q_{avg}$ (USEPA 2000). Both flowrates are dependent on the WWTP service area water usage (USEPA 2000). While flows will fluctuate based on peak hours, seasonal variation, and extreme events (e.g., fires), the flow entering wastewater treatment wetlands is generally relatively constant.

USEPA (2000) also cited a number of treatment wetland design specifications to promote sufficient pollutant reduction. Both zone 1 and zone 3 should have an average retention time of 2-days for optimal settling and to ensure significant denitrification (USEPA 2000). Zone-2 should have a retention time of 2 to 3 days to ensure sufficient time for BOD removal and nitrification, but not enough time for algal blooms to occur, which could add TSS to the water and eventually cause significant dissolved oxygen (DO) reduction in the aerobic zone-2 (USEPA 2000). An overall length-to-width ratio is not required. Ratios of installed wastewater wetlands range from 1:1 up to 90:1 with a suggested optimum range of 3:1 to 5:1 (USEPA 2000). Larger length-to-width ratios were not found to significantly improve performance or more closely mimic plug flow behavior, which promotes the most efficient water quality performance, versus smaller ratios (USEPA 2000).

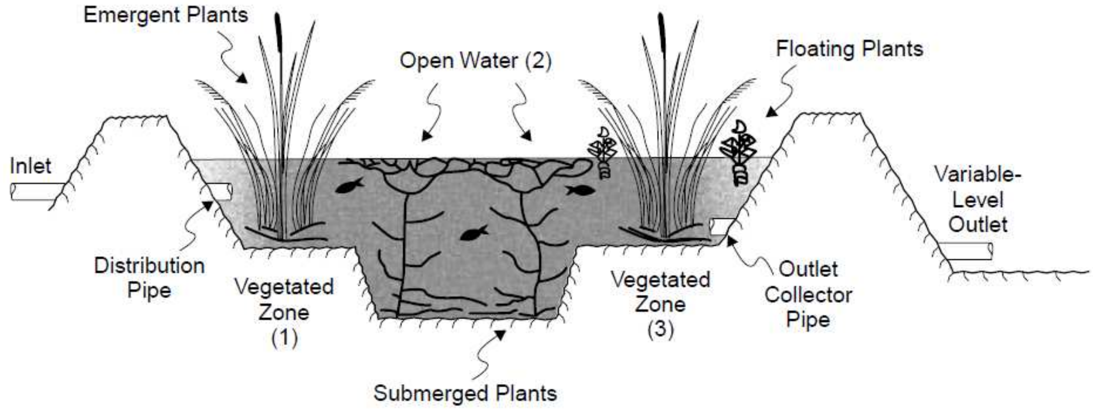

Figure 2-1 Cross-section of an example municipal wastewater treatment wetland with sequential zones 1, 2, and 3. Zones 1 and 3 are shallow with emergent vegetation while zone 2 is deeper with submerged vegetation (USEPA 2000).

In addition to the three-zone wetland system, an inlet settling zone was also incorporated into designs for which influent water was expected to have high TSS concentrations. This inlet area was specified to have a hydraulic retention time of 1 day at Q_{avg} and a depth of 3 ft. It should span the width of the inlet cell (zone 1) of the wetland. An inlet settling zone was discouraged for influent waters with lower levels of settleable solids and high algae (such as oxidation lagoons) and or dissolved solids concentrations that required flocculation and other processes for removal; however, quantitative limits were not given (USEPA 2000).

In order to reduce influent flowrates to treatment wetlands, USEPA (2000) also suggested that wetlands be divided into at least two parallel trains, each containing a minimum of three cells (zones 1, 2, and 3). This configuration allows for better system hydraulics as well as re-routing of water when maintenance is required in one zone. Additional cells (or zones) can be added in series so long as every open water zone (defined as zone 2 in the basic configuration) is followed by a shallow zone (zones 1 and 3 in basic configuration) (USEPA 2000). Data were not given to compare wetland performance of wetlands with different cell configurations.

2.2.2.1 Municipal Wastewater Treatment Wetland Design Procedure

USEPA (2000) used an example to outline the USEPA design procedure for a municipal wastewater treatment wetland. This example treatment wetland was designed as secondary treatment in a municipal wastewater treatment plant. Therefore, this wetland design was required to meet USEPA water quality standards for secondary effluent, of which selected constituents are shown in Table 2-4 (USEPA 2000). Based on these effluent requirements, the corresponding maximum areal loading rates for BOD, TSS, and TKN (see Table 2-4) could be estimated for a given wetland design (USEPA 2000). Additionally, typical water quality concentrations of primary-treated water are shown in Table 2-5.

Table 2-4 Secondary effluent required concentrations for BOD, TSS, and TKN and corresponding treatment wetland areal loading rates for treatment wetlands composed of both fully vegetated and open-space areas as defined by the USEPA (2000).

Constituent	Secondary Effluent Requirements (mg/L)	Wetland Areal Loading Rate (kg/ha-d)
BOD	30	60
TSS	30	50
TKN (NH_3-N+Org-N)	10	Not adequate data

Table 2-5 Estimated effluent water quality characteristics of primary treatment and facultative lagoons (USEPA 2000).

Constituent	Typical Primary Effluent (mg/L)	Typical Facultative Lagoon Effluent (mg/L)
BOD	40-200	11-35
TSS	55-230	20-80
TN	20-85	8-22
NH_3-N	15-40	0.6-16
NO_3-N	0	0.1-0.8

An areal loading size method, which relates influent areal loading rates with effluent concentrations of target constituents (e.g., BOD, TSS, TKN, etc.), is used by the EPA to size treatment wetlands (USEPA 2000). While the areal loading method is common due to its simplicity, it often does not capture the complexity of processes and transformations that occur within a wetland (Kadlec and Knight 1996; USEPA 2000). The areal loading rate (r_{AL}) was defined accordingly (USEPA 2000):

$$r_{AL} = 0.001 \cdot \frac{Q \cdot C}{A_w} \tag{2-5}$$

where Q represents the influent flowrate (m^3/d) to the wetland, C is the influent concentration of the constituent of concern, A_w is the wetland area (ha), and r_{AL} is the areal loading rate (kg/ha-d) for constituent C. The term 0.001 serves as a conversion factor.

The hydraulic retention time (t_{HR}) was also an important factor in EPA treatment wetland design. The hydraulic retention times of zones 1 and 3 should 2 days, with longer t_{HR} values allowing time for harmful algal blooms (see Table 2-6). Zone 3 should have a t_{HR} of at least 2 days. The following equation was used to determine the overall wetland t_{HR} (USEPA 2000):

$$t_{HR} = 10{,}000 \cdot \frac{A_w \cdot h_w \cdot \varepsilon}{Q} \tag{2-6}$$

where t_{HR} is the wetland hydraulic retention time (days), h_w represents the mean wetland depth (m), ε is a measure of vegetation density and the resulting porosity of the flow path through the wetland (dimensionless), and 10,000 is a conversion factor from ha to m^2. From Table 2-6, for example, zone 2 has a ε of 1 because the submerged vegetation does not cause flow constriction. However, zones 1 and 3 have a ε of 0.75 due to the tenuous flowpath created by dense emergent vegetation. Equation 2-6 can also be applied to individual zones. The depths, hydraulic retention times, porosity, and vegetation type specified by USEPA (2000) for each zone are summarized in Table 2-6.

Table 2-6 Different zones within a wastewater treatment wetland and their corresponding design parameters (USEPA 2000).

Zone	**Hydraulic Retention Time at Q_{max} (days)**	**Water depth (ft)**	**Porosity ε**	**Vegetation Type**
1	2	2-3	0.75	Emergent
2	≥ 2	4-5	1	Submerged
3	2	2-3	0.75	Emergent

2.2.3 NRCS Agricultural Wastewater Treatment Wetland

Agricultural wastewater is generally comprised of animal waste from agricultural areas. Throughout the literature, wetlands were found to be used for the treatment of wastewater from dairy, swine, and chicken farms (USEPA 1995; Cronk 1996; NRCS 2002). As a result of these wastewater sources, wetland influents have very high organic contents. Farmers often want to preserve a fraction of the nutrients within the wastewater for later irrigation of crops (NRCS 2002). The amount of nutrients needed for irrigation depends on the total organic content of the animal wastewater as well as the area of cropland requiring irrigation.

Constructed wetlands can be used to reduce wastewater pollutant and organic contents to acceptable levels for cropland irrigation. Therefore, constructed wetlands can be implemented as a natural method for recycling agricultural wastewater for use as irrigation water. Due to the high nutrient content of agricultural wastewater, it is crucial to use wetland plants that are tolerant of high nutrient levels (Cronk 1996). For example, Cronk (1996) cited that both giant bulrush and softstem bulrush were observed to survive in water with NH_3 concentrations greater than 200 mg/L.

In addition to constructed wetlands, pretreatment is generally required to reduce pollutant and organic levels below toxic levels for wetland vegetation. Pretreatment facilities can include a combination of a number primary treatment facilities including settling ponds, waste treatment lagoons, flotation tanks, and mechanical separators (USEPA 1995; Cronk 1996; NRCS 2002).

2.2.3.1 Agricultural Wastewater Treatment Wetland Design Procedure

In order to design an agricultural wastewater treatment wetland, the wastewater as well as the nutrient requirements for the receiving cropland must be characterized. The NRCS Environmental Engineering Handbook (2002) suggested sizing wetlands based on either a 5-day biochemical oxygen demand (BOD) or the total nitrogen (TN) required effluent loads. Because TN area requirements are most often greater, the NRCS wetland sizing method was based on TN effluent loads rather than BOD loads.

Most agricultural wastewater treatment wetlands must be zero-discharge facilities, meaning that wetland effluent cannot enter receiving natural streams. As a result, a number of designs require wastewater storage facility at the outlet of the wetland in order to store wetland effluent during non-growing seasons. These storage periods were also incorporated into the NRCS wetland design method as discussed in more depth later in this section (NRCS 2002).

The NRCS outlined two design procedures for constructed wetlands, (1) the presumptive method and (2) the field test method. The presumptive method is intended for the design of a wetland for an agricultural site that has either not been constructed or from which data have not been collected. Conversely, the field test method requires characteristic waste quality and quantity data of pre-treatment effluent water. Both methods require the knowledge of nutrient requirements of receiving cropland. Because it uses more data specific to each design site, the field test method was cited as being more accurate of the two methods (NRCS 2002). Both methods are discussed here.

2.2.3.2 Presumptive Design Method

The NRCS presumptive method for constructed wetland design procedure is outlined below:

1. Estimate the average daily TN_d (lb-TN/d) and annual TN_a (lb-TN/d) pre-treatment effluent total TN loads. These loads can be estimated based on the proposed type and magnitude of agricultural activity (e.g., a dairy farm with 500 cows), and the type of pretreatment used.
2. Determine cropland area (acres) required to utilize pre-treatment effluent total TN loads (NRCS 2002):

$$A_R = \frac{TN_a}{\left(\dfrac{\text{Crop TN Requirment}}{\text{Fraction of TN after losses}}\right)} \tag{2-7}$$

where A_R represents the required cropland area (ac) to utilize the annual pre-treatment TN load TN_a (lb-TN/yr). The crop TN requirement (lb-N/yr) is the total annual TN demand of the crops planted on the receiving cropland, and the fraction of TN after losses represents all TN available to the crop after losses due to irrigation and storage. If the resulting required area is greater than the existing cropland area, a constructed wetland is required to further reduce TN loads before the wastewater can be used for cropland irrigation. Otherwise, a wetland is not required (NRCS 2002).

3. Estimate the daily total N_i (lb-TN/d) required for the available cropland (NRCS 2002):

$$N_i = \frac{\text{Available cropland} \times \text{Crop TN requirement}}{365\text{d/yr} \times (\text{fraction of TN after losses})} \tag{2-8}$$

where the available cropland represents the area of the receiving cropland (ac).

4. Estimate the average daily constructed wetland influent volume Q_d (gal/d). This estimate can be made based on the facility size, water requirements, as well as geographical parameters such as evaporation from the pre-treatment area, rainfall depths, etc (NRCS 2002).

5. Calculate the average daily total TN effluent concentration C_e (mg/L) required from the wetland in order to provide sufficient TN to the receiving cropland (NRCS 2002):

$$C_e = \frac{N_i}{Q_d} \times \left(\frac{1gal}{3.79L}\right)\left(\frac{mg}{2.2E-6lb}\right) \tag{2-9}$$

6. Calculate areal loading rate *LR* (lb/ac/d) into the constructed wetlands (NRCS 2002):

$$LR = 0.609C_e - 7.0 \quad (2\text{-}10)$$

7. Calculate the resulting wetland surface area (ac) (NRCS 2002):

$$\text{Surface Area} = \frac{TN_d}{LR} \quad (2\text{-}11)$$

Once the surface area is calculated, the wetland length and width can be determined based on available land and topography. Ideally a L:W ratio of 3:1 to 4:1 is desired (NRCS 2002).

2.2.3.3 Field Test Design Method

If pre-treatment water quality samples were available, the field test method of design can be used in place of the presumptive method. The field test method estimates TN wetland reduction assuming that the wetland acts as a plug flow reactor. A number of the steps in this method are the same as those defined for the presumptive method.

1. Estimate the average daily Q_d (ft^3/d) and annual Q_a (ft^3/yr) pre-treatment effluent volumes. Both Q_d and Q_a can be estimated as suggested in Step 4 of the presumptive method. Flow data from pretreatment effluent may also provide actual Q_d and Q_a values (NRCS 2002).

2. Estimate the average daily TN_d (lb-TN/d) and annual TN_a (lb-TN/d) pre-treatment effluent total TN loads based on Q_d and the recorded pretreatment effluent (wetland influent) daily nitrogen concentration TN_i (mg/L), which would be collected from the site (contrary to the presumptive method where

TN_d and TN_a must be estimated based on agricultural activity, magnitude, and pretreatment facilities) (NRCS 2002):

$$TN_d = \left(\frac{3.79L}{gal}\right)\left(\frac{2.2E-6lb}{mg}\right)Q_d \cdot TN_i \tag{2-12}$$

$$TN_a = \left(\frac{365d}{yr}\right) \times TN_d \tag{2-13}$$

3. Determine cropland area (acres) required to utilize pre-treatment effluent total TN loads (same as Step 2 of the presumptive method).

4. Estimate the daily total N_i (lb-TN/d) required for the available cropland (same as Step 3 for the presumptive method).

5. Calculate the average daily total TN effluent concentration C_e (mg/L) required from the wetland in order to provide sufficient TN to the receiving cropland (same as Step 5 of the presumptive method):

6. Calculate the nitrogen reaction rate k_T (m/yr) based on the annual average operating temperature T (°C) of the wetland (NRCS 2002):

$$k_T = k_{20}\theta^{T-20} \tag{2-14}$$

   where k_{20} is the nitrogen rate constant (m/yr) at 20°C, θ is the temperature correction factor (dimensionless). NRCS (2002) suggests respective values of θ and k_{20} of 1.06 and 14 m/yr for TN calculations.

7. Calculate the resulting wetland surface area (ac) (NRCS 2002):

$$\text{Surface Area} = -(0.305)\frac{1ac}{43{,}560\,ft^2}\left(\frac{Q_a}{k_T}\right)\ln\left[\frac{(C_e - C^*)}{(TN_i - C^*)}\right]\frac{365}{t_{CW}} \tag{2-15}$$

where C^* is the background TN concentration (mg/L) in the wetland (NRCS suggested a value of 10mg/L), and t_{CW} represents the total number of days the wetland is operational within one year (days) (NRCS 2002).

8. Calculate the estimated wetland hydraulic detention time (days) (NRCS 2002):

$$t_d = (\text{Surface Area}) \times D \times \frac{n}{Q_d} \tag{2-16}$$

where D is the water depth within the wetland and n is the wetland porosity, which is estimated based on the type of vegetation to be incorporated in the wetland (e.g., wetlands planted with *Typha* spp are estimated to have an n of 0.90). The NRCS (2002) requires a minimum detention time of 6 days.

9. Calculate required winter storage volume (ft^3) if applicable (NRCS 2002):

$$\text{Winter storage volume} = (365 - t_{CW}) \times Q_d \tag{2-17}$$

Winter storage volume is only calculated if the wetland is not operational over the entire year (NRCS 2002).

2.2.3.4 General Wetland Configuration

NRCS suggested either a very slight wetland bottom slope or a flat bottom to avoid large discrepancies in water depths at the inlet and outlet of the wetland (e.g., a wetland with a bottom slope of 0.005ft/ft and a length of 100 ft may have a water depth of 6 in. at the inlet and a water depth of 12 in. at the outlet). If a sloped bottom is chosen, the wetland length should be adjusted appropriated as to ensure that all resulting water depths are appropriate for wetland vegetation growth. If a flat bottom is chosen, a deep zone located at the outlet is also suggested to ensure the outlet pipe has sufficient head for appropriate outflow rates.

Similar to the USEPA (2000) municipal wastewater treatment design, NRCS (2002) suggested dividing the wetland into parallel cell trains for maintenance purposes. In order to reduce channelization, each cell train could also be divided into shorter cells in series separated by berms parallel to the flow that would allow for even distribution over the width of the cell train. A L:W ratio of 3:1 to 4:1 should be used for the entire wetland while individual cell trains may have L:W ratios up to 10:1 or 20:1 (NRCS 2002).

Embankments should be made to maintain a wetland operating depth of 8-12 in. (the wetland may be allowed to dry out during non-operational months). Additionally, embankments should be designed to control at least a 25-yr 24-hr storm event. NRCS (2002), did not, however, specify a design storm for which wetland embankments should be designed. Ice formation depths during colder months should also be considered into embankment height design. A liner should also be incorporated in order to maintain water depths as well as to prevent groundwater contamination (NRCS 2002).

Inlets to the wetland should provide even distribution across the width of the wetland in order to minimize short-circuiting and channelization. If parallel trains are included in the design, inflow should be evenly distributed between trains. The wetland outlet should be designed based on the flow requirements of the receiving cropland. Finally, NRCS required a water balance analysis of the wetland system to ensure a wetland would be feasible in the proposed area. All berms should be designed accordingly (NRCS 2002).

2.2.4 NRCS Habitat Wetlands

The primary concern for habitat wetlands is to either restore or to create lost habitat for wetland species such as waterfowl, native wetland vegetation, aquatic mammals, reptiles, and amphibians. In accordance with the Clean Water Act of 1974, mitigation and restoration of polluted or destroyed wetlands is currently required. A number of habitat wetlands are also designed for public uses such as hiking and bird watching (Kadlec and Knight 1996). Design emphasis is placed on mimicking natural wetland ecosystems and biodiversity rather than hydrology and water quality improvement.

2.2.4.1 Wildlife Habitat Wetland Design Guidelines

Unlike the other types of wetlands, habitat wetlands do not have a well-structured design procedure due to their complexity and dependence on site characteristics and ecosystem needs. The NRCS (2009), however, outlined a number of water depth, area, and habitat island requirements for different wetland animals.

For waterfowl, the wetland surface area should be comprised of less than or equal to 20% of water with a depth of 3-4 ft, 30% with a depth of 1.5-3 ft, and the remainder with a depth of less than 1.5 ft. Wetlands designed for diving duck habitat should limit emergent vegetation cover to 50% of the total wetland surface area. Additionally, it was cited that wading birds and shorebirds demand seasonally dry mudflats with seasonal water depth of 1-4 in. Waterfowl require side slopes of 8:1 to 16:1 as well as irregular shorelines (NRCS 2009).

Amphibians require less than or equal to 20% of the wetland surface area to be comprised of water depths of 3-5 ft and greater than 50% to have depths less than 1.5 ft (NRCS 2009). These water depth requirements should promote multiple habitat types including mudflats, emergent vegetation areas, and submerged vegetation areas; which are all crucial in the entire life cycle of many amphibians and reptiles. A total of 5 basking structures (e.g., logs, boulders, etc.) per acre of wetland was also suggested by NRCS (2009). Amphibians and reptiles are also sensitive to pesticides. Therefore, wetlands designed for amphibian and reptile habitat should be located at sites with low pesticide concentrations (NRCS 2009).

Wetland furbearers (e.g., beavers, muskrats, etc.) were defined by the NRCS to require wetlands in which at least 20% of the surface are dedicated to water depths of 3-5 ft and the remainder of the surface area dedicated to depths of less than 3 ft (NRCS 2009). These water depth requirements assume no mechanical water control measures are used. If water levels can be controlled, NRCS suggested that water depths of 6-12 in be maintained during the growing season to promote emergent vegetation growth and depths of 3-5 ft be maintained during winter and fall months. Side slopes for the wetland were also specified to be between 3:1 and 16:1 (NRCS 2009).

Wildlife islands are also crucial for promoting waterfowl nesting. Wetlands must have an area of at least 10 acres in order to support habitat islands (NRCS 2009). These islands should be kidney-shaped, 15 ft wide at the base, with a height of about 1-3 ft above normal water levels, a top width of at least 6 ft, side slopes of 10:1, and top slopes of 4:1 (NRCS 2009). Islands must also be placed at least 300 ft away from each other and from the wetland shoreline for predator protection and territorial purposes. The NRCS (2009) suggested an average of 1 acre of habitat islands for each square mile of wetland area. Therefore, 4 habitat islands should be placed within each 10-acre plot of wetland (0.4 habitat islands/acre of wetland).

In addition to the above specifications, a number of other habitat structure designs and considerations were specified by the NRCS (2008). These considerations should be applied on a site-by-site basis depending on the habitat goals and limitations of the wetland. The effects of these structures were assumed to be outside the scope of the current study, which focused on the overall wetland layout, flowpath, and water chemistry rather than complex habitat dynamics. Habitat wetlands, again, must meet Code 378 Pond Standards. All berms should be designed accordingly.

2.3 WETLAND PERFORMANCE

Wetland hydrology is characteristically variable; water levels can fluctuate hourly, daily, seasonally, or even unpredictably (Mitsch and Gosselink 2007). The hydrology of a wetland greatly impacts the soil and nutrients, and, in turn, the biota of a wetland by controlling retention time and the flowpath of inflowing pollutants. Furthermore, sedimentation and plant growth from nutrients can impact wetland morphology and therefore hydrology. As a result, the relationship between wetland hydrology and water quality is complex and cyclical (Kadlec and Knight 1996; Mitsch and Gosselink 2007). Changes in hydrology will affect wetland water quality and vice versa.

Both water quality and hydrology play a large role in the biodiversity and ecological health of wetlands. One study used controlled wetland microcosms to determine the effects of different hydrologic regimes and nutrient loadings on vegetation diversity and density (Nygaard and Ejrnaes 2009). Microcosms fed with low levels of nutrients with a low simulated water table (30-cm below the surface) were found to maintain the greatest vegetation diversity. Conversely, microcosms fed with high levels of nutrients were dominated by different species depending on the simulated water-table height (10 or 30-cm below the surface). Harsher conditions simulated by low nutrient levels did not allow any one species to dominate and overtake the microcosm (Nygaard and Ejrnaes 2009). This study concluded that both hydrology and water quality impact the biodiversity and health of wetlands (Nygaard and Ejrnaes 2009).

2.3.1 Hydrologic Performance

Natural wetlands are formed by a high water table, which maintains soil saturation and is connected to baseflow as well as groundwater flow. Because constructed wetlands are often built in areas where natural wetlands did not exist, not every site has an appropriately high water table to keep soil saturated. At sites with lower water tables, a liner with low infiltration (clay, plastic, etc.) must be placed below the wetland soil in order to keep the wetland saturated (Kadlec and Knight 1996). The Maryland Department of the Environment suggests four main liner types, a 6-12-in clay liner with a maximum permeability of 1×10^{-5}cm/s, a 30-mm polyethylene-liner, bentonite, or use of chemical additives to decrease permeability (MDE 2009). Another method of establishing a water level is to dig down to the water table (MDE 2009). While liners help maintain water levels in a constructed wetland, they also reduce possible infiltration into groundwater and baseflow.

Therefore, infiltration may or may not be a large part of constructed wetland hydrology, depending on existing site hydrology and construction methods. This loss of function also reduces wetland volume and peak flow reduction effectiveness. As a result, water volume reduction is generally minimal in most constructed wetlands.

When constructed wetlands are placed in areas allowing for infiltration, they can be very effective at runoff volume reduction. In a study done by Lenhart and Hunt (2011) a monitored stormwater wetland reduced peak flows and volumes by 80 and 54%, respectively. The wetland in the study was situated in sandier soils, allowing for higher volume reductions than wetlands with liners (Lenhart and Hunt 2011). A major drought during the study period may have also affected performance (Lenhart and Hunt 2011). This study indicates that wetlands placed in infiltrative soils can perform well hydrologically.

2.3.2 Water Quality Performance

The Anacostia Restoration Team projected water quality reductions based on typical wetland design. Projected removal rates for TSS were 75%, total phosphorus (TP) 45%, total nitrogen (TN) 25%, and BOD 75% (Schueler 1992). Wetlands designed with deep ponds were projected to have nitrogen and phosphorus removals of 40 and 65%, respectively (Schueler 1992). Constructed wetlands also have natural background concentrations of a number of nutrients and water quality parameters. Therefore, removal is limited by these levels (Kadlec and Knight 1996; USEPA 2000; Mitsch and Gosselink 2007). Table 2-7 shows ranges and typical values for these background levels.

Table 2-7 Background concentrations of relevant water quality constituents in constructed wetlands based on collected data throughout the literature (Kadlec and Knight 1996).

Constituent	**Range (mg/L)**	**Typical (mg/L)**
TSS	2-5	3
BOD	2-8	5
TN	1-3	2
NH_4^+-N	0.2-1.5	1

One study on wetlands treating effluent water from septic tanks found concentration based removal rates of 78% for TSS, 39% for TP, 46% for total kjeldahl nitrogen (TKN), 39% for NH_3, and 69% for BOD (Boutilier et al. 2010). Nitrate values were below the detection limit for both influent and effluent water. Two wetlands were placed in parallel, down gradient from a septic tank, each with a surface area of about 100m^2 and a theoretical hydraulic retention time of about 25 days, which is about 10 times the EPA design HRT (Boutilier et al. 2010). Performance appeared to decrease over time and vary seasonally.

Borin et al. (2001) looked at the nitrogen removal capacity and pathways in an agricultural runoff wetland. Flow into the wetland was almost continuous, but volume fluctuated. Overall, an average of almost 90% load reduction of influent NO_3 was observed, with an influent TN load of 526kg/ha and an effluent of 58kg/ha. About half the reduction was attributed to plant uptake, suggesting that at least initially, plant uptake can play a major role in nitrogen retention. Above-ground plant biomass reached maximum nitrogen levels in the summer and minimum in the winter, while the opposite trend was seen in the below-ground biomass. The plant species studied were Phragmites australis and Typha latifolia. The study wetland area was

3200m^2, which was a little over 5% that of the total drainage area of 60,000m^2. This ratio was higher than the MDE (2009) required range of 1-2%.

Kadlec's (2010) study in Wadsworth, Illinois, found nitrogen mass removal over seven different wetlands to vary from 17-100%, with an average removal of 67%. Water was pumped into the wetlands from the river, which averaged 2.3-mg/L NO_3-N, while the wetland effluent averaged 0.9-mg/L. Pumping allowed Kadlec to analyze wetland performance during both steady-state and dynamic flows, which represent flows found in wastewater treatment wetland and runoff treatment wetlands. Removal rates were much higher during simulated storm events than during steady-state flows. As water pulses into the wetlands increased, nitrate removal decreased, most likely due to a short HRT. While oxygen levels above 1-mg/L are known to inhibit denitrification, such a trend was not found. Kadlec (2010) credited this behavior to an oxygen gradient in the water, with an oxic layer at the surface and an anoxic layer in the bottom sediments, suggesting NH_4^+ may be nitrified in surface waters, diffuse down to the sediment-water interface, and be effectively denitrified. Nitrate levels were effectively lowered and carbon availability was not found to limit denitrification significantly.

Lenhart and Hunt (2011) saw increases in TKN, NH_4, total nitrogen (TN), and TSS concentrations from wetland influent to effluent in a North Carolina urban stormwater wetland. This trend is attributed to the relatively clean water entering the site. When data were converted to load-based reductions, however, all pollutant levels decreased. The resulting load reductions were 35% for TKN, 41% for $NO_{2\text{-}3}$, 42% for NH_4, 36% for TN, 47% for TP, and 49% for TSS. Effluent concentrations were comparable to those of different North Carolina Wetlands and effluent nitrogen concentrations were similar or lower levels in a nearby river. This study suggests load removal rather than concentration removal rates properly represent wetland performance.

Wadzuk et al. (2010) monitored a constructed wetland located on the Villanova University campus. The study wetland was 0.4-ha, serving a drainage area of 18.2-ha, 9.7-ha of which was impervious surfaces. Both baseflow and surface flow hydrology and water quality was monitored. Significant load reductions were seen in TP. TN, and TSS levels. According to this study, design features such as water volume, flow path, and vegetation density and type drive pollutant removal. No significant changes in water quality performance were observed over an 8-year period, suggesting that these facilities can be effective over an extended period of time.

2.3.3 Wetland water quality characterization

2.3.3.1 Stormwater Wetlands

Stormwater water quality is very difficult to characterize due to the variability of drainage area land uses. Runoff, for example, from residential and commercial areas may have much different chemical characteristics from urban areas. Therefore, stormwater wetlands must be designed with the specific characteristics of their drainage area in mind. Table 2-8 summarizes the water quality characteristics of both urban and residential drainage areas.

Table 2-8 Literature values of stormwater concentrations (mg/L) entering treatment wetlands of all water quality constituents relevant to the current study.

Constituent	**Knight and Knight (1996) – Urban runoff (mg/L)**	**Knight and Knight (1996) – Residential/Commercial runoff (mg/L)**	**Lenhart and Hunt (2011) – Residential/Industrial (mg/L)**
BOD	20	3.6-20	---
Soluble BOD	---	---	---
TSS	150	18-140	31.2
TN	2	9.144-32.18	0.73
TKN	1.4	---	0.55
Organic N	---	---	---
NH_3/NH_4	0.582	---	0.05
NO_3	---	---	0.18

Additionally, stormwater wetland performance varies greatly due to the large variability in stormwater characteristics. As a general guide, Schueler (1992) projected that stormwater wetlands in the Mid-Atlantic region of the USA are estimated to achieve removal rates of 75% for TSS, 25% for total nitrogen, and 15% for BOD. These removal efficiencies, however, are projected values and are not based on actual wetland data.

The BMP database (BMPDB) is an online database of thousands of BMPs throughout the USA as well as a few other countries. Leisenring et al. (2012) summarized all stormwater BMP water quality performance from this database within “Chesapeake Bay related areas,” which included 11 wetland basins within the Chesapeake Bay watershed as well as nearby BMPs. The results of this report, summarized in Table 2-9, included stormwater wetland mean influent and effluent event mean concentrations (EMCs) for TSS, TN, TKN, and NO_3^-. Some incongruities in these mean EMC’s existed, as water quality data was sometimes only available for either the influent or effluent for a given storm event. Therefore, the influent and effluent EMC's for each constituent were often based on a different number of data points, where each data point represents a reported EMC. TSS influent and effluent EMC's were both based on 132 data points from 7 wetland basins. TN influent and effluent values were based on 98 and 100 data points from 3 wetland basins. NH_4^+ influent and effluent values were calculated from 111 and 110 data points from 4 wetland basins. And NO_3^- influent and effluent values were calculated from 72 and 79 data points from 3 wetland basins.

Table 2-9 Influent and effluent median concentrations for stormwater wetland basins as reported by the BMP database. The 25% percentile and 75% values are shown in parenthesis (Leisenring et al. 2012).

Constituent	**Leisenring et al. (2012) – Chesapeake Bay area BMPDB**		
	In (mg/L)	**Out (mg/L)**	**Removal (%)**
BOD	---	---	---
TSS	43.2 (21.4-91.8)	15.2 (8.5-33.3)	64.8
TN	1.88 (1.06-2.52)	1.40 (0.84-2.27)	25.5
TKN	---	---	---
Organic N*	1.25	1.07	14.4
NH_3/NH_4	0.13 (0.08-0.24)	0.08 (0.04-0.18)	38.5
NO_3	0.50 (0.28-0.93)	0.25 (0.12-0.67)	50.0

* Organic nitrogen EMC values estimated by subtracting ammonia and nitrate values from total nitrogen assuming all corresponding values are based on the same volumes.

2.3.3.2 Municipal wastewater treatment wetlands

Within the current study, municipal wastewater treatment wetlands were assumed to serve as alternatives to secondary treatment and to be fed primary treated wastewater. Table 2-10 summarizes the water quality characteristics of primary effluent water from municipal WWTPs as reported by a local WWTP (2012), USEPA (2000), and Kadlec and Knight (1996).

Table 2-10 Literature values for municipal wastewater primary effluent concentrations (mg/L) of all water quality constituents relevant to the current study.

Constituent	Local WWTP data (2012) (mg/L)	USEPA (2000) (mg/L)	Kadlec and Knight (1996) (mg/L)*
BOD	94 ± 22	40-200	170
Soluble BOD	---	35-160	---
TSS	60 ± 18	55-230	150
TN	---	20-85	37
TKN	---	---	36
Organic N	18 ± 10	---	14
NH_3/NH_4	22 ± 3	15-40	23
NO_3	---	0	0

* Kadlec and Knight (1996) estimates were calculated by subtracting the mean percent removal values given for primary treatment from the reported typical raw wastewater concentrations.

Because the chemistry of municipal wastewater changes greatly depending on the characteristics of its service area, the overall performance of municipal treatment wetlands also varies. The USEPA (2000) summarized the influent and effluent mean concentrations of BOD, TSS, TKN, and NH_4 for 22 wastewater treatment wetlands in the USA. Results, as well as the corresponding removal efficiencies, are shown in Table 2-11.

Table 2-11 Water quality performance of 22 wastewater treatment wetlands in the United States treating lagoon or primary treated water. Influent and effluent concentrations are given as ranges and the overall mean values are shown in parenthesis. Table adapted from USEPA (2000).

Constituent	USEPA (2000) – lagoon/primary pretreated (mg/L)		
	In (mg/L)	Out (mg/L)	Mean Removal (%)
BOD	6.2-438 (113)	5.8-70 (22)	80.5
TSS	12.7-587 (112)	5.3-39 (20)	82
TN	---	---	---
TKN	8.7-51 (28.3)	3.9-32 (19)	32.8
Organic N	---	---	---
NH_3/NH_4	3.2-30 (13.4)	0.7-23 (12)	10.8
NO_3	---	---	---

2.3.3.3 Agricultural swine wastewater treatment wetlands

The current study focused on modeling the performance of agricultural wetlands treating lagoon-pretreated swine wastewater. Anaerobic lagoons were the most common pretreatment practice for agricultural wastewater treatment wetlands found throughout the literature (Hammer 1992, USEPA 1995, Hunt et al. 2002, Knight et al. 2000). Pretreated swine wastewater characteristics from three sources are summarized in Table 2-12. The values from USEPA (1995) and Hunt et al. (2002) represent lagoon-treated wastewater while those from Kadlec et al. (2000) represent wastewaters entering19 different treatment wetlands, most of which were pretreated by lagoons and the remaining treated by settling basins.

Table 2-12 Literature values of agricultural swine wastewater concentrations (mg/L) entering treatment wetlands of all water quality constituents relevant to the current study.

Constituent	**Knight et al. (2000) (mg/L)**	**USEPA (1995) – lagoon pretreated (mg/L)**	**Hunt et al. (2002) – lagoon pretreated (mg/L)**
BOD	104	45	287 ± 92
Soluble BOD	---	---	---
TSS	128	118	1860 ± 470
TN	407	104	---
TKN	---	---	365 ± 41
Organic N	---	---	---
NH_3/NH_4	366	94	347 ± 52
NO_3	---	---	0.04 ± 0.03

Again, treatment wetlands serving swine wastewater may provide a wide range of water quality performance depending on the swine lot size and management, pretreatment methods, and the design of the wetland system. While some swine wastewater treatment wetlands follow a similar design as the high-low-high-marsh system suggested by the USEPA (2000) for municipal wastewater treatment wetlands, others follow different designs. Knight et al. (2000) summarized the BOD, TSS, TN, and NH_4 average inflow and outflow concentrations for all relevant wetlands within the Livestock Wastewater Treatment Wetland Database. These results are shown in Table 2-13. Each data point used in these estimates represents a single reported data point from one of the 19 swine treatment wetlands within the database. The USEPA (1995) also summarized the results of a wetland treating lagoon-pretreated wastewater in Alabama over a three month study period. Water quality results from this study are shown in Table 2-14.

Table 2-13 Water quality performance of all wetlands treating pretreated swine wastewater in the Livestock Wastewater Treatment Wetland Database (Knight et al. 2000).

Constituent	**Knight et al. (2000)**			
	# Data points	**In (mg/L)**	**Out (mg/L)**	**Removal (%)**
BOD	183	104	44	58
TSS	180	128	62	52
TN	164	407	248	39
TKN	---	---	---	---
Organic N	---	---	---	---
NH_3/NH_4	183	366	221	40
NO_3	---	---	---	---

Table 2-14 Water quality performance of a wetland treating lagoon-treated swine wastewater in Alabama over a three month study period. The wetland flowrate was 1094 gpd (USEPA 1995).

Constituent	**USEPA (1995) – lagoon pretreated**		
	In (mg/L)	**Out (mg/L)**	**Removal (%)**
BOD	45	9	80
TSS	118	10	92
TN	104	6	94
TKN	---	---	---
Organic N	---	---	---
NH_3/NH_4	94	3	97
NO_3	---	---	---

2.4 WETLAND MODELS

A number of studies have been devoted to both understanding and modeling the complex behavior of wetland chemistry and hydrology. The following section will review the current methods used to model constructed wetland behavior as well as the data required to use them.

A review paper written by Kumar and Zhao (2010) summarized the current modeling methods used to predict different constituent concentrations in constructed wetlands. The two main model types cited were black-box models and process-based models (Kumar and Zhao 2010). Black-box models are calibrated using empirically derived based on the relationship between wetland inflow and outflow values. Process-based models attempt to describe wetland behavior by numerically computing the actual processes within the wetland (Kumar and Zhao 2010).

Currently, black-box models are most commonly used to model and design constructed wetlands (Kumar and Zhao 2010). Block-box models that rely on first-order kinetics have been cited as inadequate for wetland models due to their oversimplification of wetland processes (Kadlec 2000). Process-based models, on the other hand, have potential to more accurately mimic wetland behavior, but require detailed data that is often difficult to obtain (Kumar and Zhao 2010). This section looks at wetland modeling methods of current literature as well as their advantages and disadvantages.

2.4.1 Black-Box Models

Black-box models include regression models, first-order models, time-dependent retardation models, Monod models, tanks-in-series models, neural networks, and statistical approaches. All black-box models depend on an empirical relationship between inflow and outflow concentrations of a given constituent.

Regression models relate inflow and outflow concentrations with the hydraulic loading rate with a number of empirically determined regression coefficients:

$$C_{out} = aC_{in}^{b}q^{c} \tag{2-18}$$

where C_{out} is the resulting outflow concentration (M/L^3), C_{in} is the inflow concentration (M/L^3), q is the wetland hydraulic loading rate (M/T); and a, b, and c are regression coefficients. Tang et al. (2009) used a multivariate linear regression to model the effluent concentrations of benzene in a vertical-flow, subsurface constructed wetland. This method allowed the authors to infer benzene concentrations, which is expensive and time-intensive to analyze for, by measuring more easily determined variables such as DO, electric conductivity, pH, temperature, and redox potential (Tang et al. 2009). While this method was effective for a specific wetland, regression coefficients may not be applicable to other wetlands.

Similarly, first-order models use a rate coefficient k to relate inflow and outflow concentrations using an exponential decay equation. These models assume plug-flow behavior in the wetland. Plug-flow describes the overall wetland behavior, assuming pollutant concentrations change with respect to the location of a given section (a plug) of water in the flow path (Kadlec and Knight, 1996). The general form for a first-order transformation can be described by (Kadlec and Knight, 1996)

$$\frac{C_{out}}{C_{in}} = e^{-kt} \tag{2-19}$$

where k is the removal rate (M/T), and t is the hydraulic retention time (T) (Kadlec and Knight, 1996; Kumar and Zhao 2010).

Because first-order models are often inadequate due to their simplicity, the time-dependent retardation model was developed to improve results (Kumar and Zhao 2010). This model accounts for the decrease in removal rate over time by introducing a first-order rate constant *k* that decreases with increasing retention time (Kumar and Zhao 2010).

Carleton et al (2001) used a plug-flow equation to model total phosphorus, ammonia (NH_4^+), and nitrate (NO_3^-) in 49 wetland systems. Outflowing concentrations were calculated accordingly (Carleton et al. 2001; Kadlec 2010):

$$\frac{d(V_i C_i)}{dt} = QC_i - Q(C_i + dC_i) + rdV \qquad (2\text{-}20)$$

where the subscript "*i*" represents water entering the section of interest, and dC_i is the concentration change within that section while dV is the water volume contained in the section. The transformation rate, r, is defined by both the Monod equation and first-order kinetics in the literature (Kadelc and Knight 1996; USEPA 2000; Sykes 2003). This rate can be used to describe the behavior of nitrogen, phosphorus, and BOD species.

The tanks-in-series (TIS) model breaks the wetland into sequential completely mixed flow reactors (CMFR) along the flow path. Each tank is assumed to be completely mixed with first-order kinetics driving constituent degradation (Kadlec and Knight 1996; Kumar and Zhao 2010). The overall expression relating influent and effluent concentrations in a wetland analyzed with TIS is as follows:

$$\frac{C_{out}}{C_{in}} = \frac{1}{(1 + k_{VRC} t / N)^N} \tag{2-21}$$

where k_{VRC} is the first-order volumetric rate constant (T^{-1}), t is the total retention time in the wetland (T), and N is the number of tanks.

Kadlec (2010) used a tanks-in-series (TIS) model, as calibrated by tracer studies, to determine the internal hydrology and chemistry of each wetland; 3 tanks were used for three of the wetlands and 5 tanks were used for the remainder. The TIS model is also covered in Kadlec and Knight (1996). Both steady state and dynamic hydrology was used to model performance.

In another study, denitrification was modeled using a TIS with first order areal uptake equation, and dynamic nitrate balances were used to model the concentrations and flows of nitrate as a function of time (Kadlec 2010). First-order rate constants were found to be much higher during simulated storm events (k_{20}=107) versus periods of steady-state (k_{20}=37). An Arrhenius temperature factor (θ) of 1.09 is also commonly used for wetlands (Kadlec 2010). Both event-based and dynamic equations were used to describe nitrate removal and transformations. Event-based mass removal was represented by

$$\%M \text{ Removal} = \frac{\int (QC)_{in}\, dt - \int (QC)_{out}\, dt}{\int (QC)_{in}\, dt} \times 100 \tag{2-22}$$

where Q represents flowrate (L^3/T), C is concentration (M/L^3), and t is time (T) (Kadlec 2010). The dynamic mass balance for each tank of the wetland (based on TIS) is shown by (Kadlec 2010):

$$\frac{d(V_i C_i)}{dt} = Q_{i-1} C_{i-1} - Q_i C_i + rV \tag{2-23}$$

where V is the wetland tank volume (L^3), r is the transformation rate ($M/L^3/T$), and Q is the wetland tank flowrate (L^3/T). The subscript "*i-1*" represents water flowing into the tank and "i" represents water in the tank and flowing out of the tank.

Monod models estimate effluent concentrations based on bacterial growth rates and the available substrates for decomposition of a constituent. The general form of the Monod equation is shown below (Sykes 2003).

$$u = \frac{u_{\max} S}{K_s + S} \tag{2-24}$$

where u is bacterial growth rate (1/T), u_{max} is the maximum growth rate (1/T), S is the limiting substrate concentration (M/L^3), and K_s is the concentration at $u_{\max}/2$ (M/L^3). If multiple substrates could be limiting, Monod expressions for each can be multiplied in a model, inhibiting growth if one substrate is absent (Sykes 2003). Once a growth rate is established for a wetland, the relative transformation rate of a compound can be determined based on bacterial transformation efficiency and speed (Kadlec and Knight 1996).

Depending on the compound being modeled and available data, one model may be more appropriate than the other. While a number of first-order removal rates have been computed based on actual constructed wetland behavior, Monod variables are generally taken from analogous wastewater treatment facilities (Kadlec and Knight 1996; Sykes 2003). How different types of wetland removal rates compare is also a relevant concern. Carleton et al. (2001) found first-order removal rate constants used for wastewater treatment wetlands to match stormwater wetland performance for total phosphorus, ammonia (NH_4^+), and nitrate (NO_3^-).

Artificial neural networks (ANNs) have also been used to model constructed wetlands (Kumar and Zhao 2010). These models imitate the structure of biological neural networks in order to establish a model that adapts rate constants and other parameters based on patterns recognized by the ANNs. Because these relationships are not based on physical processes within the systems, ANN models are considered black-box models (Kumar and Zhao 2010).

2.4.2 Process-Based Models

Current constructed wetland process-based models and model environments include the FITOVERT model, the constructed wetland two-dimensional (CW2D) model, the structural thinking experimental learning laboratory with animation (STELLA) software, the 2D mechanistic model, and the constructed wetland model No. 1 (CWM1) (Mayo and Bigambo 2005; Kumar and Zhao 2010). HSPF can also be used to create a constructed wetland environment. A number of studies have used such models to predict subsurface constructed wetland behavior. Fewer studies, however, have modeled water-quality performance in surface-flow constructed wetlands.

Mayo and Bigambo (2005) used STELLA to develop a mathematical model of nitrogen in a horizontal subsurface flow (HSSF) constructed wetland. Mechanisms of nitrogen transformation and retention included in the model were mineralization, nitrification, denitrification, sedimentation, plant and microorganism uptake, plant and microorganism decay, and resuspension. Driving factors of nitrogen levels included nitrogen species concentrations, microorganism growth rates (specifically *Nitrosomonas*), dissolved oxygen concentrations, pH, water temperature, plant growth rates, and water flow rate. Monod equations and a number of first-order differential equations were used to describe individual nitrogen mechanisms.

While process-based models have the potential to produce precise results, lack of sufficient data often limits their usefulness and accuracy (Kumar and Zhao 2010). Kumar and Zhao (2010) also cited the need for future research focused on improving process-based water quality models in constructed wetlands through better technical understanding of the processes that control them. Greater knowledge of these processes paired with more detailed and extensive data collection could lead to significantly more accurate process-based models (Kumar and Zhao 2010).

2.4.3 Hydrologic Modeling

Inflows to a wetland could be from runoff, wastewater effluent, streams, or groundwater flow. Water can exit a wetland through groundwater infiltration, baseflow, streamflow, and evapotranspiration (Kadlec and Knight 1996). While the majority of water leaves through streamflow, evapotranspiration has strong diurnal and seasonal variation (Kadlec and Knight 1996). A number of sources first calculated a water balance for modeling purposes (Kadlec and Knight 1996; USEPA 2000). The general form of this balance is.

$$\frac{dV_W}{dt} = Q_{in} + Q_c - Q_{out} + (P - ET - I)A_W \tag{2-25}$$

where A_W represents the wetland water surface area (L^2), V_W is water volume storage in wetland (L^3), t is time (T), ET is evapotranspiration rate (L/T), I is infiltration to groundwater (L/T), P is the precipitation rate (L/T), Q_{in} is the flowrate via the inlet

(L^3/T), Q_c is runoff flowrate into catchment by means of the wetland sides (L^3/T), and Q_{out} represents the flowrate out of the wetland (L^3/T).

If inflowing water is from runoff, the rational method can be used to determine the flowrate for a given rainfall event (McCuen 2005):

$$q_p = CiA \qquad (2\text{-}26)$$

where q_p represents the peak flowrate of the runoff generated by the drainage area (ft^3/s), C is the runoff coefficient, i is the rainfall intensity (in./hr), and A is the drainage area in acres (McCuen 2005). Rainfall models can also be used to simulate rainfall depth and intensity over a period of time based on historical data based on three probability distributions, for rainfall amount, storm duration, and interstorm time periods (Kadlec and Knight 1996).

Internal wetland flow (Q) is often characterized by the relative slope S of water pathway (L/L), bottom roughness n ($T/L^{1/3}$), cross-sectional area A (L^2), and hydraulic radius R_h (L) as demonstrated by Manning's equation (Kadlec and Knight 1996; McCuen 2005):

$$Q = \frac{1}{n} A R_h^{2/3} \sqrt{S} \qquad (2\text{-}27)$$

A number of other studies also estimated the roughness coefficient based on vegetation density and water depth (Reed et al. 1995; Kadlec and Knight 1996; Crites et al. 2006).

Weirs are also used in wetlands at the entrance and outlet as well as at any transition points within the flow path (e.g. a weir may be placed between the forebay and the main wetland). The general form of the weir formula is shown below (McCuen 2005)

$$Q_{out} = C_w L h^{1.5} \tag{2-28}$$

where Q_{out} is the flowrate through the weir (L^3/T), C_w is the weir coefficient, L is the weir length (L), and h is the water height above the weir (L). A general orifice outlet equation can be used to model outflow from a submerged outlet

$$Q_0 = C_d A\sqrt{2gh} \tag{2-29}$$

where Q_0 is the outlet flowrate (L^3/T), C_d is dimensionless discharge coefficient, A is the area of the orifice (L^2), g is gravity (L^2/T), and h is the water height above the orifice (McCuen 2005).

Infiltration and evapotranspiration are also possible exiting routes for water depending on the presence of a liner, water table level, permeability of surrounding soils, surface area, vegetation density, and weather variations. If sediments are assumed to be saturated, Darcy's Law can be used to determine the infiltration rate (Fetter 2001). However, if the wetland is periodically dry and unsaturated flow occurs, the Green-Ampt equation can be used to model infiltration (Fetter 2001). The Penman evaporation method was suggested by Kadlec and Knight (1996) for determining wetland evapotranspiration:

$$ET = K_e[P_w^{sat}(T_w) - P_{wa}] \tag{2-30}$$

Where ET is evapotranspiration (L/T), K_e is the water vapor mass transfer coefficient (L/T/P), P_{wa} is the ambient water vapor pressure (P), $P_w^{sat}(T)$ is the saturation water vapor pressure at T_w (P), and T_w is the water temperature (°C)

(Kadlec and Knight 1996). Evapotranspiration can be estimated by a number of other formulas as well, including the Hammer and Kadlec equation (Mitsch and Gosselink 2007).

A study done on nitrate dynamics in seven wetlands over a 4-yr period developed a number of equations that described the hydrology of wetlands (Kadlec 2010). Equation 2.3-1 was used as an overall water balance for each tank. A modified Penman equation was used to model evaporation and infiltration was deduced by mass balances at steady-state (Kadlec 2010). The dynamic wetland volume was calculated accordingly:

$$V = \int_0^H A\,dh \tag{2-31}$$

where V represents the wetland water volume (L^3), A is the surface area of the water (L^2), and h represents the average water depth in the wetland (L) (Kadlec 2010). This general equation was also suggested by Kadlec and Knight (1996) for modeling wetland water storage.

The hydraulic loading rate q, and hydraulic retention time $\bar{t}$ were also modeled base on the instantaneous flowrate $\overline{Q}_P$, the water surface area at the maximum water level A_{max}, and the maximum water volume V_{max} (Kadlec and Knight 1996):

$$q = \frac{\overline{Q}_P}{A_{\max}} \tag{2-32}$$

$$\bar{t} = \frac{V_{\max}}{\overline{Q}_P} \tag{2-33}$$

2.4.4 Water Quality Modeling

Constructed wetland water quality parameters are most often calculated in the literature using black-box models. The following sections show how constituents specific to the current study have typically been modeled as well as the processes that dictate their removal.

2.4.4.1 Total Suspended Solids (TSS)

The general removal mechanisms for TSS are sedimentation, i.e., settling out, and flocculation (USEPA 2000). Stokes Law is used in a number of sources to model TSS removal and settling out (Kadlec and Knight 1996; USEPA 2000):

$$v_s = \frac{(\rho_p - \rho_f)}{18\mu} g D^2 \tag{2-34}$$

where v_s is the settling velocity (L/T), ρ_p is the particle density (M/L^3), ρ_f is the fluid density (M/L^3), μ is the fluid dynamic viscosity (F-T/L), g is gravity (L^2/T), and D is the particle diameter (L). The use of Stoke's Law assumes plug flow through the wetland and that particles entering the wetland are vertically uniformly distributed and approximately spherical. TSS removal can also be modeled using first-order removal rates. Boutilier et al. (2010) determined a TSS removal rate constant based on a first-order plug flow model of two wetlands treating wastewater from a domestic septic tank. The resulting average rate constant was $0.08d^{-1}$. An assumed TSS background concentration of 2mg/L was also used in this method (Boutilier et al. 2010).

2.4.4.2 Oxygen and Oxygen Demand

Oxygen is transferred to and consumed through a number of pathways in a wetland (Kadlec and Knight 1996; Chapra 1997; USEPA 2000; Bicknell et al. 2001). Kadlec and Knight (1996) modeled oxygen aeration from the atmosphere according to a general mass transfer equation

$$J_{O_2} = K(C_{DO}^{sat} - C_{DO}) \tag{2-35}$$

$$K = \sqrt{\frac{D \cdot U}{h}} \tag{2-36}$$

where J_{O2} is the oxygen flux from air to water ($M/L^2/T$), *K* is a mass transfer coefficient (L/T), C_{DO}^{sat} is the saturation DO concentration at the water surface (M/L^3) and is a function of water tempterature, C_{DO} is the DO concentration in the bulk of the water, *D* is the molecular diffusivity of oxygen in water (L^2/T) and was estimated to be $1.76 \times 10^{-4} m^2/d$ at 20°C, *U* is the water speed (L/T), and h is the water depth (L) (Kadlec and Knight 1996).

Both photosynthesis and plant oxygen transfer were noted by Kadlec and Knight (1996) as additional DO pathways into wetlands. While plant oxygen transfer was not found to be a significant DO source, photosynthesis was estimated to contribute about 2.5g $O_2/m^2/d$ during the day (Kadlec and Knight 1996). Four main wetland oxygen demands were also listed, sediment-litter oxygen demand, respiration requirements, dissolved carbonaceous BOD, and dissolved nitrogenous oxygen demand (NOD) (Kadlec and Knight 1996).

The EPA used the same oxygen transfer equation shown in Equation 2-35 (USEPA 2000). A mass transfer coefficient of $0.43 d^{-1}$ was estimated based on a water velocity of 30 m/d, a depth of 0.3 m, and a temperature of 20°C (USEPA 2000). The resulting oxygen flux into the wetland would then have a range of 0.5 to 0.9 g/m^2/d depending on the background DO levels (USEPA 2000). Surface reaeration would be higher in open water areas versus areas with emergent and floating vegetation, which impede oxygen transfer (USEPA 2000). Emergent zones generally have DO levels close to zero (USEPA 2000). The EPA also notes that phytoplankton, and submerged plants transfer oxygen to the water, contributing 2.5 g O_2/m^2/d during daylight hours. A DO gradient was observed in a number of wetlands, with higher DO levels and the surface, and lower levels at the bottom (USEPA 2000). Oxygen transfer through plant shoots down to the root zone was not deemed a significant source of oxygen in sediments (USEPA 2000).

Wetland oxygen demands defined by the EPA are influent organic matter, endogenous respiration, dead biomass, and influent NH_4^+-N (ammonium) (USEPA 2000). While influent DO demands can be modeled based on NH_4^+ and BOD concentrations, oxygen flux due to photosynthesis and other internal wetland mechanisms are difficult to quantify due to limited data (USEPA 2000).

The HSPF model also accounted for oxygen transfer and consumption in a stream, considering longitudinal advection of BOD, sinking of BOD material, benthal oxygen demand, benthal release of BOD material, surface and plant reaeration, and oxygen depletion due to BOD decay (Bicknell et al. 2001).

Ro et al. (2010) analyzed the oxygen transfer efficiencies of three wetland systems treating swine wastewater. One wetland was a marsh-pond-marsh setup, the second a marsh-floating bed-marsh, and the third only marsh. Marsh areas were 0.15-

m deep with emergent plant species. Pond and floating-bed components were 0.65-m and mechanically aerated from the bottom; the pond had no vegetation and the floating beds consisted of plastic beds with plants in them. Both surface and plant aeration were determined for each wetland setup. An oxygen transfer equation was used to model the oxygen concentration at a given time

$$C = C_{\infty} - (C_{\infty} - C_0)e^{-K_{La}t} \qquad (2\text{-}37)$$

where C in the oxygen concentration (M/L^3) at time t, C_{∞} is the saturation concentration, C_0 is the initial oxygen concentration, and K_{La} is the oxygen transfer coefficient (1/T). The K_{La} value was also adjusted for temperature, T (°C) changes according to Equation A-21. Where $K_{La(20)}$ is the transfer coefficient at 20°C and θ is the temperature coefficient, which is typically 1.024 (Ro et al. 2010). The K_{La} value was further modified based on wastewater conditions:

$$K_{La(w)} = K_{La}\left(\frac{\beta \cdot C_{\infty,T} - C}{C_{\infty,20}}\right)(1.024^{T-20})\alpha \qquad (2\text{-}38)$$

where $K_{La(w)}$ is the oxygen transfer rate in wastewater (1/T), $C_{\infty,T}$ is the clean water saturation DO, at temperature T (M/L^3), α is the K_{La} correction factor (typically 0.4 to 0.8), and β is the C_{∞} correction factor (typically 0.95 to 0.98). Values depend on wastewater type and strength, which affects oxygen demand (Ro et al. 2010).

A BOD removal rate constant of 0.03d^{-1} was found by Boutilier et al. (2010) based on two wetlands serving wastewater from a septic tank. This rate constant was determined using a plug-flow model to describe each wetland, which matches well with the estimated background concentration of 3 mg/L by Kadlec and Knight (1996).

2.4.4.3 Nitrogen

The nitrogen cycle is very complex, however, five main routes of nitrogen transformation are noted throughout the literature, volatilization, ammonification, nitrification, denitrification, and assimilation (Kadlec and Knight 1996; USEPA 2000; Mitsch and Gosselink 2007). These pathways are shown in Figure 2-2. Ion exchange is also a route of nitrogen removal, the EPA, however, found it to be of little significance in wetland systems (USEPA 2000).

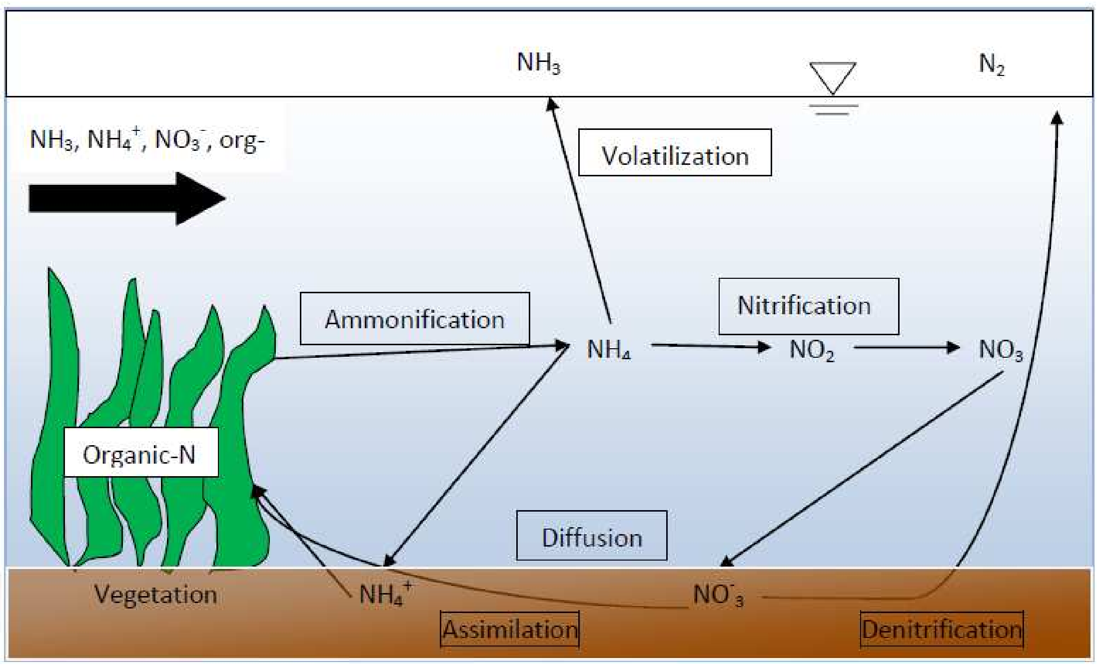

Figure 2-2 Schematic representation of major nitrogen pathways in a wetland.

At a neutral pH and at 25°C, only about 0.6% of the ammonia species is the volatile form, NH_3 (Kadelc and Knight 1996). However, active photosynthesis can increase the pH to up to 8-8.5, and the resulting NH_3 fraction up to 20-25% at 20°C (USEPA 2000). Removal of ammonia species through volatilization is, therefore, dependent on both temperature an pH.

Plant uptake or assimilation is seasonal and is estimated to range from 0.5 to 3.3-gN/m^2/yr (USEPA 2000). When plants are growing, they hold significant amounts of nitrogen in their leaves and stems above the sediments. During senescence, nitrogen moves down to the roots for storage (USEPA 2000). Despite this storage, a significant amount of nitrogen is introduced to the water column from plant decay, often resulting in a net export of nitrogen in the fall and early spring (USEPA 2000). HSPF also allows for both the Monod and first-order equations to be used, but defaults to first-order equations to describe nitrogen movement through water (Bicknell et al. 2001). Boutilier et al. (2010) found a removal rate constant of 0.03d^{-1} for TKN and 0.04d^{-1} for NH_3 for septic tank treatment wetlands. Assumed background concentrations were 0.1mg/L TKN and 0.05mg/L NH_3 (Boutilier et al. 2010).

2.5 WETLAND SUSTAINABILITY

General sustainability is the capacity to endure. As defined by the Brundtland Commission of the United Nations on March 20, 1987: "sustainable development is development that meets the needs of the present without compromising the ability of future generations to meet their own needs." Sustainability with respect to an ecosystem is "'…the ability of an ecosystem to maintain a defined/desired state of ecological integrity over time" (Balmori and Benoit 2007).

Constructed wetlands serve to reduce the impact of development on surrounding ecosystems and could be considered ecosystems themselves. Therefore, wetland sustainability can relate to both downstream effects and their implications for the future, and to their internal ecological integrity. Therefore, depending on perspective, different wetland functions and qualities may contribute more or less to overall wetland sustainability. A number of studies have defined and quantified sustainability for different applications. Costanza et al. (1997) monetarily quantified the sustainability of different ecosystems based on the ecosystem services they provided to humans.

Neuman and Churchill (2011) modeled sustainability as a function of different processes, assigning each process a quantitative value based on the first and second laws of thermodynamics, and the rate process concept. In this study, a sustainable process was defined as one that can maintain a constant rate over time without over-burdening its supporting ecosystem. Neuman and Churchill (2011) also identified that as dictated by the second law of thermodynamics, full sustainability can never be achieved because there are losses in every system. A total of five main rates were used to define a rate-process for sustainability, (1) consumption, (2) production, (3) accumulation, (4) depletion, and (5) assimilation. The interaction of these rates was summed to find an overall sustainability value for a system (Neuman and Churchill 2011).

Kang and Lee (2011) developed a watershed sustainability model called the Water Resources Sustainability Evaluation Model. Using 4 criteria, economic efficiency, social equity, environmental conservation, and maintenance capacity, a number of indicators for each criterion were measured and weighted to determine the overall sustainability of a watershed. Indicators were evaluated based on relevance to a criterion, measurability, transparency, and data availability (Kang and Lee 2011).

Each criterion was defined as a subindex and calculated accordingly (Kang and Lee 2011):

$$SC_j = \frac{wv_1 I_1 + wv_2 I_2 + \cdots + wv_m I_m}{wv_1 + wv_2 + \cdots + wv_m} \tag{2-39}$$

where wv_m represents the indicator *m* weight for the subindex j. The I_m values represent the normalized values of the indicators and *m* is the number of indicators. SC_j is the values of subindex *j* (Kang and Lee 2011). All specified wv_m sum to 1.0. The overall sustainability index was then calculated by adding the weighted subindex values:

$$WRSI = \frac{w_1 EES + w_2 SES + w_3 ECS + w_4 MCS}{w_1 + w_2 + w_3 + w_4} \tag{2-40}$$

where $w_1, w_2, w_3,$ and w_4 are the subindex weights for the economic efficiency subindex (*EES*), the social equity subindex (*SES*), the environmental conservation subindex (*ECS*), and the maintenance capacity subindex (*MCS*). All weights *w* sum to 1.0. Relative indicator and subindex weights were determined by accounting for the preferences of the people affected and evaluated by expert opinion. Additionally, indicator distributions were determined using the nonlinear probability distribution function method (NPDFM), which was found to be more effective than the linear method for data sets with a wide range of values.

A number of other studies also used sustainability indices and indicators to calculate a sustainability value for a given system. Most studies, however, used different methods to quantify each indicator. Availability of relevant data played a role in the use of different indicator measurements as well as the indicators chosen (Manzini et al. 2011).

The NRCS and Deluca et al. (2004) used habitat indices to determine the integrity and sustainability of wildlife survival. The NRCS designed surveys for workers involved in planning and constructing agricultural resource management systems such as drainage and pasture planting. Areas were evaluated based on land use, vegetation type, buffer strip area and location, distance to natural ecosystems, and farming practices (i.e., till versus no-till). Each criterion was associated with a given number of points, adding up to a maximum of 100 points total. The area score was divided by 100 to get a habitat index between zero and 1 (NRCS 2006). A habitat index of ≥ 0.5 was required for constructed habitat wetlands, and a value of ≥ 0.75 was considered to provided excellent habitat (NRCS 2006).

DeLuca et al. (2004) developed an index of marsh bird community integrity (IMBCI). This index was developed by combining two approaches, (1) the guild-based community index and (2) the indicator species approach. Species attributes were measured on a specialist to generalist gradient from 4 to 1 respectively. Specialists depend on specific sources of food/shelter, having difficulty adapting to other sources when the habitat is disturbed. As a result, specialists were suggested to be good indicator species because they are the first affected by change.

All species were scored 1 to 4 for each attribute of interest (ie., foraging habitat, nesting material, etc.). Scores for each attribute were summed to define the overall score for one species. All bird scores were summed and divided by the total number of bird species to determine an average index value for an area. There was no significant relationship between the IMBCI scores and wetland habitat characteristics, suggesting a single plant community did not dictate bird community integrity. The IMBCI was, however, significantly reduced by urban and suburban development within 500m and 1000m of the wetland (DeLuca et al. 2004).

Maes et al. (2011) measured the effects of sustainable forest management on forest composition, structure, and functioning using a criteria and indicator framework. Indicators and their corresponding weights were evaluated based on a panel of 19 experts from relevant fields. Five main selection criteria were used for choosing and weighting appropriate indicators including, suitability of evaluating and quantitating desired aspects, distinguishing power (can this indicator detect significant differences in the forest function/structure?), scientific correctness, measurability, and appropriate scale level.

2.6 UNCERTAINTY ANALYSIS

Due to the unpredictability and complexity of nature, uncertainty is inherent in research. Uncertainty is an attribute of a measurement or result that reflects the lack of exact knowledge of the measure and or the output quantity (Salicone 2007). In relation to modeling, uncertainty analysis is defined as “the study of model output uncertainty as a function of a careful inventory of the different sources of uncertainty present in the model input parameters” (Singh et al. 2007). Traditionally, “uncertainty is defined as the estimated amount by which an observed or calculated value may depart from the true value” (Shirmohammadi et al. 2006). Understanding the uncertainty in a model and its parameters allows for it to be used properly and its limitations known (Salicone 2007).

There are two main types of uncertainty, inherent and epistemic (Salicone 2007). Inherent uncertainty, also called sampling variation, is caused by randomness in nature and is unavoidable in many cases. This randomness can occur in both time and space (Salicone 2007). Epistemic uncertainty is caused by lack of knowledge of the system of study or scarcity of data, and can be reduced through careful research and good judgment (Salicone 2007). Statistical and model uncertainties are types of epistemic uncertainty. Uncertainty in parameter assumptions (often caused by insufficient data or poor parameter estimation methods) as well as in the distributions used to describe variable behavior are the main sources of statistical uncertainty (Singh et al. 2007). Knowledge gaps or difficulty in computing the processes a model simulates may also contribute to error in the form of model uncertainty (Singh et al. 2007).

Two main types of error can cause uncertainty, natural randomness and errors in data and modeling. Errors in data and modeling are either random or systemic (Singh et al. 2007). Systemic error is consistent bias based on measurement method or model. Random error occurs when different results are observed in repeating sampling, showing statistical regularity (standard deviation) around a population mean (Singh et al. 2007). The random variability associated with a parameter can be described by a probability distribution with a central tendency (mean) and its coefficient of variation (standard deviation). The uncertainty associated with this randomness is:

$$CV = \frac{\sigma_x}{\bar{x}} \qquad \textbf{(2-41)}$$

where CV is the coefficient of variation, σ_x is the estimated standard deviation, and $\bar{x}$ is the estimate of the mean for a given sample set (Salicone 2007; Singh et al. 2007). A more general method of defining parameter uncertainty is to compute the difference between an input parameter value and the parameter mean:

$$u(x_i) = x_i - \bar{x}_i \tag{2-42}$$

where $u(x_i)$ is the uncertainty associated with parameter *i*, x_i is a specific value for parameter *i*, and $\bar{x}_i$ is the mean parameter value. The overall uncertainty in an output parameter can be determined by summing the uncertainties associated with all contributing input parameters. The following equation can be used to find this cumulative uncertainty (Salicone 2007):

$$u_c(y) = \sqrt{\sum_{i=1}^{n}\left(\frac{\partial f}{\partial x_i}\right)^2 u^2(x_i) + 2\sum_{i=1}^{n-1}\sum_{j=i+1}^{n}\frac{\partial f}{\partial x_i}\frac{\partial f}{\partial x_j}u(x_i, x_j)} \tag{2-43}$$

where $\frac{\partial f}{\partial x_i}$ is the partial derivative of output *f* with respect to parameter parameter x_i, and $u(x_i, x_j)$ is the estimated difference between input parameters *i* and *j*.

A number of studies also used sensitivity analysis to determine relative parameter importance in a model (van der Peijl and Verhoeven, 1999; Wang and Mitsch, 2000). This analysis assessed how each parameter affected the final model output and can be calculated by

$$S_X = \frac{\delta X / X}{\delta P / P} \tag{2-44}$$

where P is the input parameter, S_X is the model relative sensitivity with respect to P, and X is the model output. Therefore, the sensitivity of a model with respect to a parameter is the ratio of the normalized change in the output over the normalized change in the input parameter (van der Peijl and Verhoeven, 1999). A higher S_X value indicates greater sensitivity to a given parameter.

Park et al. (2011) used three different uncertainty methods to analyze error in modeling stormwater BMPs, the derived-distribution method (DDM), Latin hypercube sampling (LHS), and the first-order second-moment (FOSM) method. Effluent TSS concentrations (C_{out}) were calculated based on an input first-order k value and an influent TSS concentration, C_{in}. Both k and C_{in} values were assumed to follow a lognormal distribution. Resulting outlet TSS concentrations were compared with detention basin data sets from the International Stormwater BMP database. The LHS method was found to be the most efficient and accurate method (Park et al. 2011). Because q, the hydraulic loading rate depends heavily on k, a prediction interval was defined around their linear relationship,

$$Mean \pm t_{0.025} s \sqrt{1 + \frac{1}{n} + \frac{(X - \overline{X})^2}{\sum_{i=1}^{n} (X_i - \overline{X})^2}} \tag{2-45}$$

where t is the t distribution for (n-2) degrees of freedom, n is the number of total data, s is the standard error of the regression, X is the average q at which the confidence interval is calculated, $\overline{X}$ is the mean observed q from observed data, and X_i is an individual observed q from data (Park et al. 2011). A prediction interval, rather than a confidence interval was used because the study was more concerned with performance for individual events than with the average performance for a number of similar events (Park et al. 2011).

Using a first-order plug flow equation to model TSS concentrations, the input variables k and C_{in} were subject to uncertainty analysis. The C_{out} distribution was calculated using three methods, based on the uncertainty in C_{in}, based on the uncertainty in k, and based on the uncertainties in both C_{in} and k. C_{out} values based on both uncertainties using the LHS method matched data with a 95% confidence interval (Park et al. 2011).

Hughes and Mantel (2010) applied uncertainty analysis to model the effects of small farm dams on downstream flow in different climate zones in South Africa. Each known parameter was input to the model along with a defining distribution. Parameters could be fit with a normal, log-normal, or uniform distribution. The Monte Carlo method was used to sample values for a given run from the parameter distributions. Lack of sufficient dam hydrologic data was the main contributor to uncertainty in the model (Hughes and Mantel 2010).

Another study done in the UK modeled nitrate transport and loading for a rural headwater basin (Howden et al. 2011). Using uncertainty analysis allowed Howden et al. (2011) to reliably use historical data on nitrate loads from outside the drainage area for the model. Input variables were given uniform distributions, from which output parameters were computed (Howden et al. 2011). Nitrate loading was most sensitive between the years 1930-1985, when fertilizers became the largest source of nitrate, decreasing after 1985 when fertilizer use declined (Howden et al. 2011). The fertilizer input, was therefore concluded to be one of the most influential factors predicting the overall nitrate load.

Chapter 3: Sustainability Metrics Development

The current study aims:

- To define sustainability as it applies to constructed wetlands and to develop metrics based on sustainability principles that connect wetland design with intended wetland functions.

In order to sum individual wetland metrics (e.g., TSS removal, flowrate discharge reduction, etc.) to determine an overall wetland sustainability index, all metrics were normalized accordingly. After normalization, all metrics followed the same scale of 0 to 1, where 0 implies poor sustainability and 1 suggests high sustainability.

3.1 DEFINITIONS

Wetland Function (WF): a general, intended wetland service (e.g., water quality improvement, water quantity management, wildlife habitat, aesthetics, etc.).

Performance Criteria (PC): specific, quantifiable wetland services that represent the processes and/or conditions contributing to the fulfillment of wetland functions (e.g., TN removal, internal wetland surface water depths, etc.). Multiple performance criteria can be used to fully characterize one WF.

Metric (M): A normalized measure on a scale of 0.0 to 1.0 used to quantify the sustainability of a given PC and/or WF. A value of M of 0 implies poor PC or WF sustainability, while a value of 1 implies optimal sustainability.

Metric Weight (MW): The design-engineer/stakeholder assigned weights given when multiple metrics are used to evaluate performance of one wetland function.

Performance-to-Metric Function (PTM): An established function that relates PC values with corresponding M values. These functions are based on the relative sensitivity of sustainability to a given PC as well as goal PC values.

Wetland Sustainability Index (WSI): The weighted sum of M values for all wetland functions. This index represents the overall sustainability score for a given wetland design.

Sustainability Weight (SW): The stakeholder-assigned weights given to each wetland metric. These weights will be assigned based on relative stakeholder importance of wetland functions.

3.2 WETLAND FUNCTIONS AND PERFORMANCE CRITERIA

The wetland functions that analyzed in the current study are (1) wildlife habitat, (2) flood control, (3) downstream hydrologic regime maintenance, (4) wetland water balance, (5) groundwater recharge, (6) baseflow maintenance, (7) downstream water quality, and (8) aesthetics. Multiple performance criteria were used to fully characterize each general wetland function. Wildlife habitat, for example, was evaluated based on internal wetland water depths, water quality, and flows. Table 3-1 lists all performance criteria (PC) used to assess the sustainability of each wetland function.

3.3 RELATING PERFORMANCE CRITERIA AND METRICS THROUGH PTM FUNCTIONS

Metrics were related to PC values through developed Performance-to-Metric functions (PTMs), with the metrics defined on a scale of 0.0 to 1.0. PTM functions were developed based on the relative effect a given PC had on a WC. For example, the current study, assumed that downstream levels of NO_3^- greater than 0.36 mg/L were exponentially related to downstream benthic community health decline (McNett et al. 2010). However, benthic communities were assumed to be insensitive to changes in NO_3^- concentrations that ranged between 0 and 0.39 mg/L. Therefore, the PTM function for the wetland NO_3^- PC produced a NO_3^- metric of 1 if NO_3^- concentrations were less than or equal to 0.39 mg/L and used an exponential function to determine NO_3^- metrics corresponding to NO_3^- concentrations greater than 0.39 mg/L (see Figure 3-9). The following two sections describe in detail how all PTM functions were determined based on downstream performance goals within the current study. Different PTM functions could result based on different stakeholder needs and goals for a given wetland design.

A general parabolic PTM function was used to relate PC values and metrics (see Figure 3-1) when insufficient data were available in the literature to define a PTM function based on the sensitivity of wetland function sustainability to a given PC value. This general PTM function was defined accordingly:

$$M = \begin{cases} -PC^2 + 2 \cdot PC & \text{for } PC \leq 2 \\ 0 & \text{for } PC > 2 \end{cases} \tag{3-1}$$

where M and PC are the metric and performance criterion represent a given wetland function. For a PC value with a range from 0 to ∞, an M value of 1 would result from a PC of 1, implying that the wetland function was performed optimally by the wetland design. Conversely, an M of 0 would result from a PC equal to 0 or greater than 2, indicating that the wetland design failed to perform the corresponding wetland function. The width of the parabola was chosen so as to produce M values symmetrically at *PC* values of 0 and 2 about the peak *M* of 1, which occurred at a *PC* of 1 (see Figure 3-1). Therefore, this general PTM function assumed that both positive and negative deviations from an optimal *PC* value of 1 had negative effects on the ability of the wetland to perform the corresponding wetland function.

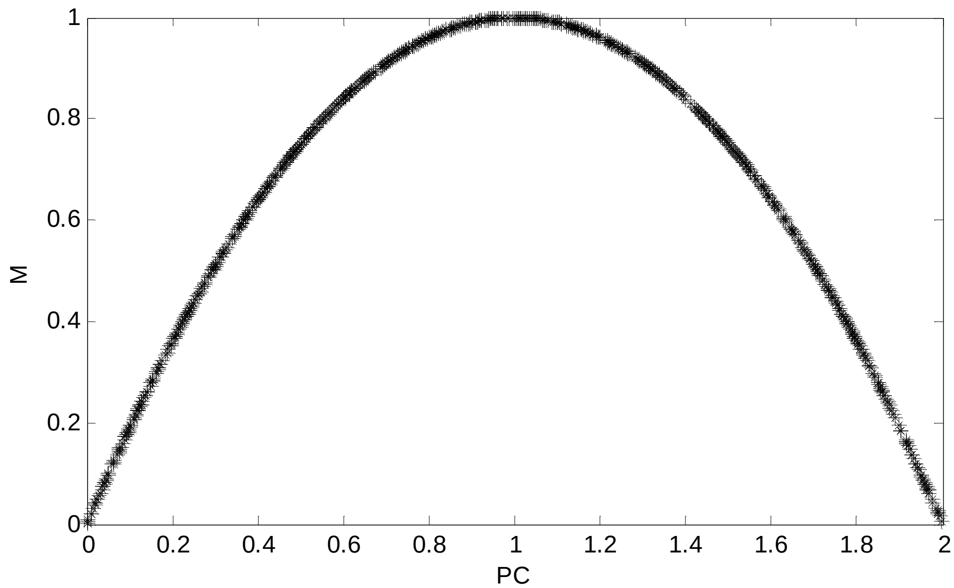

Figure 3-1 General parabolic PTM function used to relate PC values with corresponding metrics.

Table 3-1 Wetland functions and corresponding characteristic functional values.

Wetland Function	Function Goal	Performance Criteria
Wildlife habitat	To provide a wetland habitat suitable for diverse numbers of wetland wildlife, especially those species specified by stakeholders. *The current study used the marsh wren as an indicator species.*	Mean daily high-marsh water depths, fractional wetland coverage of high-marsh area, ratio of number of habitat islands to optimal number of habitat islands.
Flood control	To reduce flooding downstream due to upstream development.	Ratio of pre-development mean annual flow volume to wetland mean annual outflow volume.
Hydrologic regime	To create a wetland effluent hydrology that mimics the estimated pre-development hydrologic regime	Ratios comparing the exceedence of wetland outflow discharge rates for analogous pre-development bankfull and zero flows with those of pre-development conditions.
Wetland water balance	To maintain appropriate seasonal/annual water levels within the wetland based on design specifications.	Ratio of mean daily wetland water depth to design/goal water depth.
Groundwater recharge and baseflow maintenance	To restore groundwater recharge volumes to pre-development values.	Ratio of wetland surface area to impervious area within contributing drainage area.
Downstream water quality	To input healthy water quality levels into downstream natural water body.	Effluent wetland daily average water quality parameters, dissolved oxygen, NO_3^-, NH_4^+, and TSS.
Aesthetics	To create a wetland that is visually appealing to the surrounding community.	Ratios describing perimeter irregularity, total number of wetland types, and total wetland surface area.

3.4 WETLAND PERFORMANCE CRITERIA AND METRICS

This section outlines the specific PC and metric values that were used to assess each WF value. PC values were used to assess the corresponding WF values based on both internal wetland and downstream sustainability. A number of PC values are included in multiple WF definitions because they are measures of performance of multiple wetland functions. For example, dissolved oxygen concentrations are used to assess both wetland wildlife habitat and downstream water quality. Additionally, multiple performance criteria can be used to characterize the sustainability of one wetland function. Downstream water quality, for example, was quantified by a number of criteria including TSS removal, BOD removal, TN removal, and DO concentration. Different performance criteria may also be used in any one design depending on stakeholder objectives, wetland location and intended use, and data availability.

3.4.1 Wildlife Habitat

The aim of the wildlife habitat function is to provide a wetland design that is suitable for healthy wetland wildlife, especially those species specified by stakeholders. In order evaluate this suitability, internal wetland hydrology, water quality, and physical design structures (e.g., habitat islands for waterfowl) were assessed for a given model and compared to specified optimal values. A number of these optimal values would change based on stakeholder-specified wildlife needs including required water depths, internal flowrates, and vegetation inclusion.

As an example, the current study evaluated wildlife habitat by using the marsh wren as an indicator species. Marsh wrens are marsh obligates, which means that they rely solely on marshes for habitat and are, therefore, good indicators of marsh health (DeLuca et al. 2004). A marsh is a wetland characterized by herbaceous, emergent vegetation such as that found within the high-marsh portions of a constructed wetland (Mitsch and Gosselink 1993).

The marsh wren requires a wetland comprised of at least 50% high-marsh areas with emergent vegetation and mean water depths greater than or equal to 0.50 ft both for reproductive and cover purposes (Gutzwiller and Anderson 1987). The marsh wren also requires specific emergent vegetation species including cattails (Typha spp.), bulrushes (Scirpus spp.), and sedges (Crex spp.), all of which are commonly implemented in constructed wetlands (Gutzwiller and Anderson 1987; EPA 2000). One PC value was developed to evaluate the habitat suitability for marsh wrens for a given wetland design:

$$PC_{H1} = \frac{A_V}{A} \tag{3-2}$$

where A_V represents the total wetland surface area covered by emergent vegetation (ft^2), A is the total wetland surface area (ft^2), and PC_{H1} represents the resulting marsh wren habitat PC value. Additionally, a more general PC was developed to evaluate the waterfowl habitat created by habitat islands. According to NRCS (2001) habitat islands must be 400 ft away from the shoreline and placed 300 ft away from each other. The resulting habitat island PC value is a ratio of the number of designed habitat wetlands to the maximum number of appropriately spaced habitat islands within the wetland:

$$PC_{H2} = \frac{H_I}{HI_{IX}} \tag{3-3}$$

where H_I is the number of appropriately spaced habitat islands in the wetland design; HI_{IX} is the optimum number of islands for a given wetland design, and PC_{H2} is the corresponding habitat island PC value. HI_{IX} was estimated based on the NRCS (2001) habitat island spacing rule of thumb of 0.4 islands/acre. Therefore, a PC_{H2} less than 1 indicated that a wetland design did not incorporate a sufficient number of habitat islands while a PC_{H2} value greater than 1 implied habitat islands were overcrowded.

Once all wetland habitat PC values were established, corresponding PTM relationships were developed for both wetland habitat PC values. The PTM for PC_{H1} (the PC value related to marsh wren habitat) was modified from the habitat suitability index developed by Gutzwiller and Anderson (1987):

$$M_{H1} = \frac{1}{1 + 10890\exp(-15 \cdot PC_{H1})} \tag{3-4}$$

$$M_{H2} = \begin{cases} PC_{H2} & \text{for } 0 \le PC_{H3} \le 1 \\ 3 - 2PC_{H2} & \text{for } 1 < PC_{H1} \le 1.5 \\ 0 & \text{for } PC_{H1} > 1.5 \end{cases} \tag{3-5}$$

where M_{H1} and M_{H2} are the respective metric values corresponding to PC_{H1} and PC_{H2}. The PTM relating PC_{H1} and M_{H1} was based on suggested minimum and maximum PC_{H1} values of 0.50 and 0.80, which are required for sufficient emergent vegetation cover for marsh wren nesting and reproductive purposes (Gutzwiller and Anderson 1987). The PTM relating PC_{H2} and M_{H2} was formed around the optimal number of habitat islands in a given wetland. A linear relationship with a slope of 1 was assumed to relate PC_{H2} less or equal to 1 and their corresponding M_{H2} values. A PTM slope of -2 was chosen to calculate all M_{H2} values for PC_{H2} values greater than 1. Both habitat PTM relationships are shown in Figure 3-2.

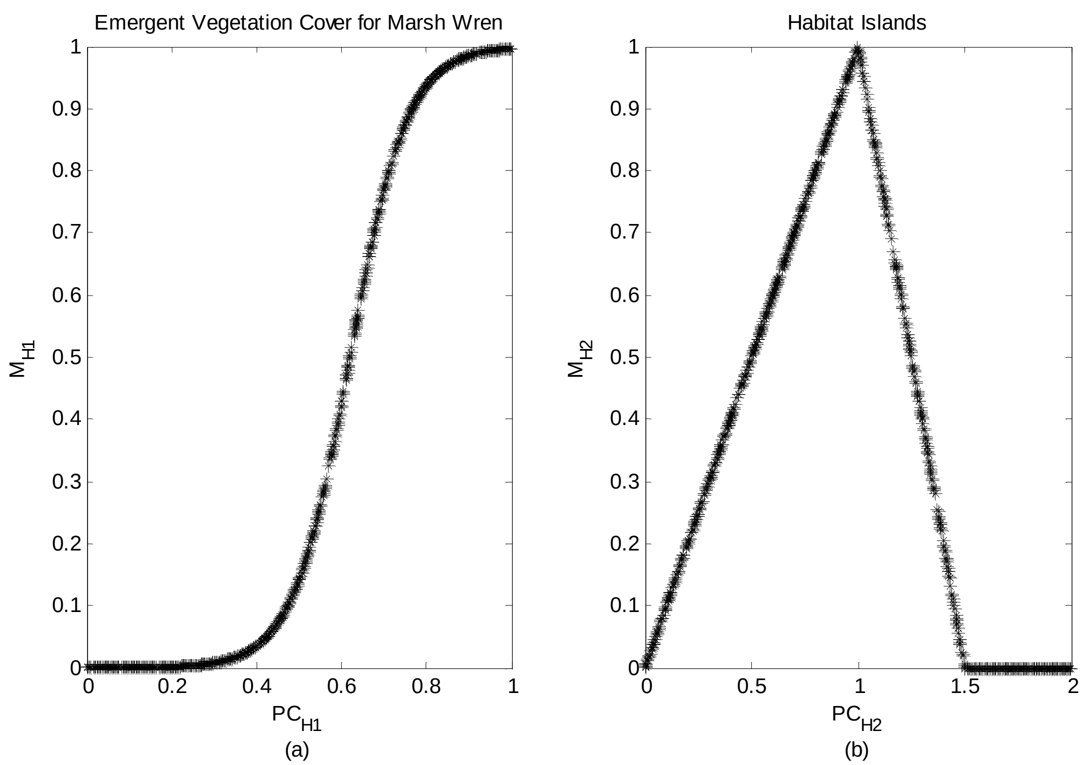

Figure 3-2 PTM relationships of PC_{H1} (a) and PC_{H2} (b); and their corresponding metric values.

3.4.2 Flood Control

The goal of the wetland function for flood control was to evaluate the ability of the wetland to mitigate downstream flooding caused by development in the contributing drainage area. The mean annual wetland effluent runoff volume was compared with simulated pre-development mean annual volume of runoff from the drainage area to assess flood control performance. Within the model all volumes were computed with units of ft^3. A simple mean volume ratio was developed as the volume control PC value:

$$PC_{FC} = \begin{cases} 0 & \text{for } \overline{V}_{OUT} = 0 \\ \dfrac{\overline{V}_{PRE}}{\overline{V}_{OUT}} & \text{for } \overline{V}_{OUT} > 0 \end{cases} \tag{3-6}$$

where $\overline{V}_{OUT}$ is the mean annual cumulative effluent runoff volume (ft^3) over a given simulation period, $\overline{V}_{PRE}$ is the mean annual cumulative pre-development runoff volume (ft^3) over a given simulation period, and PC_{FC} represents the PC value measuring the relative difference between mean cumulative annual outflow and pre-development volumes (dimensionless). A PC_{FC} value of 1 would indicate that the wetland perfectly mimics pre-development flood control characteristics. Additionally, a PC_{FC} value greater than 1 indicates that pre-development volumes are greater while a value less than 1 indicates that pre-development volumes are smaller. Because of its ratio form, the resulting PC_{FC} value allows for normalized comparison between sites with varying drainage areas and , therefore, is a useful model evaluation tool on its own.

The corresponding flood control PTM relationship should be calculated based on specific stakeholder flood control goals for downstream hydrologic characteristics. If outflow volumes lower than estimated pre-development volumes are acceptable (e.g., in highly developed areas where runoff volumes are of great concern or in areas were downstream baseflow is not a concern, etc.), a PTM with a metric value of 1 for all PC_{FC} less than or equal to 1 would be appropriate. However, if baseflow was of concern, metric values should decrease as PC_{FC} values deviate positively and negatively from 1. Additionally, downstream sensitivity to volume changes should dictate the shape of the chosen PTM for flood control. This sensitivity could be based on specific species needs, downstream bank properties, etc.

A simple parabolic PTM relationship was chosen to represent flood control performance in the current study as an example:

$$M_{FC} = \begin{cases} -PC_{FC}^2 + 2 \cdot PC_{FC} & \text{for } PC_{FC} \leq 2 \\ 0 & \text{for } PC_{FC} > 2 \end{cases} \tag{3-7}$$

where M_{FC} is the corresponding metric value for PC_{FC}. For the current study, positive and negative biases were assumed to be equally undesirable downstream and therefore assumed to produce symmetric M_{FC} values. The PTM function used to relate PC_{FC} and M_{FC} in Equation 3-7 (plotted in Figure 3-3) was used to relate a number of PC and corresponding metric values due to its general form. As mentioned earlier, these PTM functions were subject to change given stakeholder goals and wetland site properties.

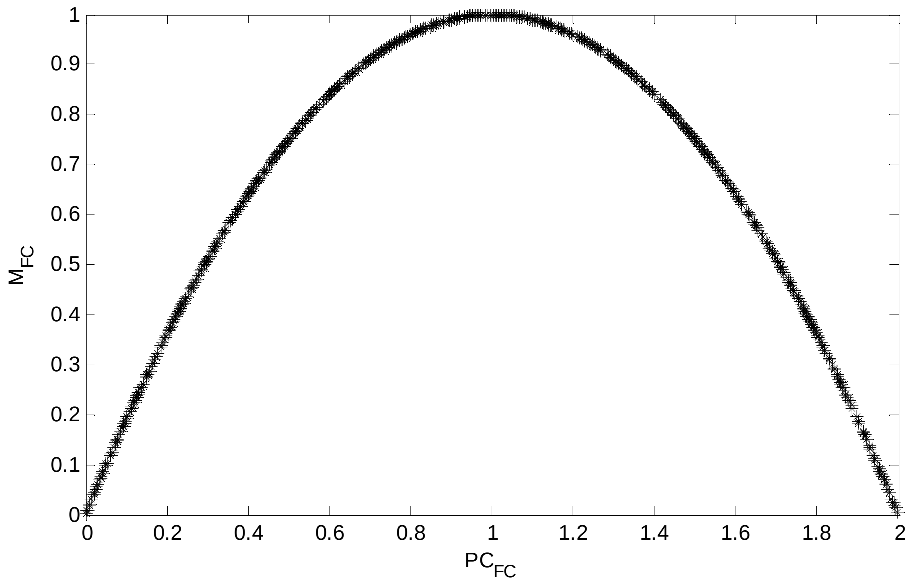

Figure 3-3 PTM function relating PC_{FC} and M_{FC}.

3.4.3 Downstream Hydrologic Regime

The overall flow regime of a waterbody has both direct and indirect impacts on the distribution, abundance, and health of resident aquatic organisms (Konrad and Booth 2002; Poff et al. 1997; Walsh et al. 2005; Stromberg et al. 2007). Therefore, a complete assessment of wetland hydrologic performance should incorporate its entire flow regime. Five components are consistently cited to be most crucial in assessing the hydrologic regime, (1) magnitude, (2) timing, (3) frequency, (4) duration, and (5) rate of change (Richter et al. 1996; Poff et al. 1997; Arthington and Zalucki 1998; DePhillip and Moberg 2010). These components, excluding timing, were incorporated into the downstream hydrologic regime metrics of the current study. Timing was not included in the current study because it generally deals with the seasonality and predictability of flows, which is of major concern when controlling dam flow, but not as much of a concern with BMP design as urbanization does not generally change the seasonality of flows (Poff et al. 1997).

While flow magnitudes are captured by current BMP metric ratios, frequency, duration, and rate-of-change are not. The frequency and duration of flows of different magnitudes are crucial assessment factors of the natural hydrologic regime. Additionally, flow flashiness and reactivity to storm events (i.e., rate of change) greatly impacts downstream ecosystem health.

A number of studies have created in-depth methods of assessing and comparing hydrologic regimes in an effort to incorporate all five hydrologic components with respect to the assessment of water allocation control, irrigation regulations, and river/stream management (Richter et al. 1996; Arthington and Zalucki 1998). Richter et al. (1996) developed the Indicators of Hydrologic Alteration (IHA) method, which uses 32 hydrologic parameters based on the five hydrologic descriptors and 64 corresponding statistics (the mean and coefficient of variance of each parameter) to compare pre and post-development flow regimes with the final goal of developing metrics to aid in ecosystem management and restoration (Richter et al. 1996). A follow-up study (Richter et al. 1997) also addressed the issue of creating flow targets for stream and river management based on the results from IHA analysis, using a "Range of Variability" (RVA) approach. Additionally, Richter (2009) proposed a "Sustainability Boundary Approach" as a method of limiting high and low flow deviations from corresponding baseline or natural flows.

While the IAH program generates a detailed description of both pre and post-development hydrologic regimes, the large number of outputs produced could be overwhelming and not readily adopted by BMP managers and researchers as it requires the use of the IAH software and interpretation of 64 output statistics.

Therefore, the current study aimed to characterize the five hydrologic components using a smaller number of metrics that could more easily be applied in conjunction with existing metrics to the evaluation of BMP hydrologic performance.

To date, environmental flow guidelines analogous to those developed by Richter et al. (1997) have not been developed in the context of wetland or general BMP hydrologic performance despite their relevance. The use of such hydrologic characterization could provide vital information about BMP hydrologic performance. The proposed metrics were developed in an effort to better reflect the actual hydrologic regime created by a given wetland and how this regime compares to that of an estimated analogous pre-developed area.

In order to evaluate all relevant components of the hydrologic regime of the effluent flows of a given wetland design, three downstream hydrologic regime performance criteria were developed. These performance criteria are referred to as (1) the low-flow PC value, (2) the high-flow PC value, and (3) the flow-variation PC value. The low-flow PC value evaluates the frequency and duration over which a user-specified goal low flowrate is exceeded in the wetland effluent discharge as compared with discharge from an analogous pre-developed area. Similarly, the high-flow PC value evaluates the frequency and duration over which a user-specified goal high flowrate (e.g., estimated downstream bankfull flow) is exceeded in the wetland effluent discharge as compared with discharge from an analogous pre-developed area. The flow variation PC value is then used to compare the rate of change or flashiness in wetland effluent discharge rates as compared with analogous pre-development discharge rates. Combined, these three downstream hydrologic regime metrics evaluate the extent to which wetland effluent hydrology mimicks the pre-development hydrologic components: magnitude, frequency, duration, and rate of change.

3.4.3.1 Low-flow and high-flow PC values and metrics

The goal low flowrate was set to zero under the assumption that the analogous pre-developed area did not produce flow between storm events. Therefore, the low-flow PC value was a measure the frequency of effluent flows exceeding zero relative to the frequency that pre-developed flows exceeded zero. This low flow goal could also be set to equal a specific baseflow discharge rate if the model user was concerned with baseflow maintenance.

Similarly, the pre-development bankfull discharge rate, which was estimated to equal the pre-development 2-yr flow, was assumed to equal the goal high-flowrate. Bankfull flow represents the flow at which channel erosion and morphological changes are most effective (Dunne and Leopold 1978). Therefore, if a constructed wetland produces flow contributing to bankfull flow more often than pre-development conditions would, the downstream channel morphology will be impacted. In order to ensure minimal downstream morphological impacts, the wetland effluent bankfull flow frequency and duration should match that of pre-developed conditions.

The goal high flow or bankfull flow was calculated by creating an annual maximum series of pre-development flows over the simulation period. Dunne and Leopold (1978) specified that annual-maximum series should be constructed with instantaneous peak flowrates as opposed to daily mean flowrates. Therefore, the maximum instantaneous (1-min) pre-developed discharge rates from each year of simulation were collected and ranked largest to smallest and assigned a corresponding return period:

$$R_P = \frac{1+M}{r} \tag{3-8}$$

where R_P is the return period in years, M is the number of years of simulation or record, and r is the rank of a given annual maxima flowrate. The resulting flowrate corresponding to a return period M of two yrs was then estimated to equal the pre-developed bankfull flow Q_{bk} (cfs).

Once the goal low and high flowrates were defined, the corresponding low-flow and high-flow PC values could be defined. The low-flow PC value was computed by taking the ratio of the proportion pre-development vs. wetland effluent 1-min flowrates that exceeded the goal low flowrate of 0 cfs:

$$PC_{H(L)} = \frac{p_{PRE(L)}}{p_{OUT(L)}} \tag{3-9}$$

where $p_{OUT(L)}$ is the proportion of 1-min wetland effluent discharge rates exceeding 0 cfs over the simulation period, $p_{PRE(L)}$ is the proportion of 1-min pre-development discharge rates exceeding 0 cfs over the simulation period, and $PC_{H(L)}$ is the resulting low-flow PC value. The high-flow PC value was computed similarly with respect to the proportion of 1-min effluent and pre-development discharge rates exceeding the estimated pre-development bankfull flow Q_{bk}:

$$PC_{H(H)} = \frac{p_{PRE(H)}}{p_{OUT(H)}} \tag{3-10}$$

where $p_{OUT(H)}$ is the proportion of 1-min wetland effluent discharge rates exceeding Q_{bk} over the simulation period, $p_{PRE(H)}$ is the proportion of 1-min pre-development discharge rates exceeding Q_{bk} over the simulation period, and $PC_{H(H)}$ is the resulting high-flow PC value.

The PTM relationship for both $PC_{H(H)}$ and $PC_{H(L)}$ should be dependent on downstream sensitivity to change in flow. Therefore, the current study assumed a simple parabolic PTM relationship for both $PC_{H(H)}$ and $PC_{H(L)}$:

$$M_{H(H)} = \begin{cases} -PC_{H(H)}^2 + 2 \cdot PC_{H(H)} & \text{for } PC_{H(H)} \le 2 \\ 0 & \text{for } PC_{H(H)} > 2 \end{cases} \tag{3-11}$$

$$M_{H(L)} = \begin{cases} -PC_{H(L)}^2 + 2 \cdot PC_{H(L)} & \text{for } PC_{H(L)} \le 2 \\ 0 & \text{for } PC_{H(L)} > 2 \end{cases} \tag{3-12}$$

where $M_{H(H)}$ and $M_{H(L)}$ represent the corresponding hydrologic regime metrics for $PC_{H(H)}$ and $PC_{H(L)}$.

3.4.3.2 Flow variation PC value and metric

In addition to $PC_{H(H)}$ and $PC_{H(L)}$, a third hydrologic regime PC value $PC_{H(CV)}$ was developed to evaluate the variation or rate of change in outflow rates as they compared with corresponding simulated pre-development flow rates:

$$PC_{H(CV)} = \frac{\overline{CV}_P}{\overline{CV}_E} \tag{3-13}$$

where $\overline{CV}_P$ is the mean pre-development daily flowrate coefficient of variation (dimensionless) over the simulation period, $\overline{CV}_E$ is the mean BMP effluent daily flowrate coefficient of variation (dimensionless) over the simulation period, and $PC_{H(CV)}$ is the resulting flow variation PC value. $\overline{CV}_E$ and $\overline{CV}_P$ represented the mean values of the coefficient of variations of all 1440 one minute outflow and pre-development flowrates occurring within each day of record. Zero flows were excluded from these computations in order to avoid the negative skewing that occurred from their inclusion. For a given day of record *i*, $CV_{E(i)}$ and $CV_{P(i)}$ were computed accordingly:

$$CV_{E(i)} = \frac{1}{\overline{Q}_{E(i)}} \sqrt{\frac{\sum_{t=1}^{24} \left(Q_{E(t,i)} - \overline{Q}_{E(i)}\right)^2}{1439}} \tag{3-14}$$

$$CV_{P(i)} = \frac{1}{\overline{Q}_{P(i)}} \sqrt{\frac{\sum_{t=1}^{24} \left(Q_{P(t,i)} - \overline{Q}_{P(i)}\right)^2}{1439}} \tag{3-15}$$

where *t* represents the minute of a given day (min), *i* represents the day of year (DOY) , $Q_{P(t,i)}$ is the pre-development flowrate magnitude at minute *t* for DOY *i* (cfs), $Q_{P(i)}$ is the daily mean pre-development flowrate for DOY *i* (cfs), $Q_{E(t,i)}$ is the wetland effluent flowrate magnitude at minute *t* for DOY *i* (ft^3/s), $\overline{Q}_{P(i)}$ is the daily mean pre-development flowrate for DOY *i* (ft^3/s), $\overline{Q}_{O(i)}$ is the daily mean wetland effluent flowrate for DOY *i* (ft^3/s), $CV_{P(i)}$ is the coefficient of variation of pre-development flowrates for DOY *i* (dimensionless), $CV_{E(i)}$ is the coefficient of variation BMP effluent flowrate for DOY *i* (dimensionless). As stated before, the shape of the PTM relationship should be a function of downstream sensitivity to flow variation. A simple parabolic PTM relationship was used in the current study as an example:

$$M_{H(CV)} = \begin{cases} -PC_{H(CV)}^2 + 2 \cdot PC_{H(CV)} & \text{for } PC_{H(CV)} \leq 2 \\ 0 & \text{for } PC_{H(CV)} > 2 \end{cases} \tag{3-16}$$

where $M_{H(CV)}$ is the resulting flow variation metric. Resulting PTM functions relating all downstream hydrologic regime PC values with their corresponding metrics are plotted in Figure 3-4.

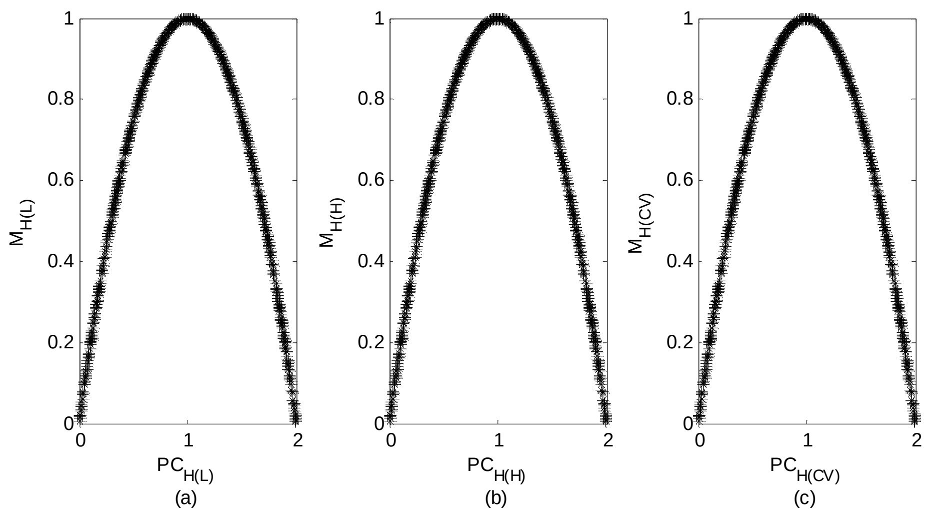

Figure 3-4 PTM functions relating (a) $PC_{H(H)}$ and $M_{H(H)}$, (b) $PC_{H(L)}$ and $M_{H(L)}$, and (c) $PC_{H(CV)}$ and $M_{H(CV)}$.

3.4.4 Wetland Water Balance

The metrics for the wetland water balance function (see Table 3-1) evaluate the extent to which the wetland maintains specified design water depths. Throughout the literature, wetland designs and components are often defined based on their water depth. Therefore, the wetland water balance function was defined in the current study to evaluate the ability of a wetland design to maintain such design depths. Because wetland water levels naturally fluctuate, both seasonal and annual water depth distributions based on daily mean values could be evaluated, with distributions identified for each design based on literature depth specifications, and vegetation and wildlife needs. For example, USEPA (2000) specified for wastewater treatment wetlands that shallow areas with emergent vegetation should have water depths of 2.5 ± 0.50 ft (0.75 ± 0.15 m) throughout the year while open water areas with submerged vegetation should maintain water depths of 4.4 ± 0.50 ft (1.35 ± 0.15 m). MDE (2009) also specified respective high and low-marsh water depth ranges for stormwater wetland design. According to MDE (2009), high-marsh areas should have water depths of ≤ 0.5 ft while low-marsh areas should have water depths between 0.5 and 1.5 ft. Because seasonality in water depths was not specified for constructed wetlands in the literature, only annual water depth trends were used in the development of the wetland water balance metrics. Two PC expressions, one for shallow areas and one for deep areas were developed to assess wetland water balance performance:

$$PC_{WB(S)} = \begin{cases} 0 & \text{for } \overline{SS}_S = 0 \\ \dfrac{SS_{GOAL(S)}}{\overline{SS}_S} & \text{for } \overline{SS}_S > 0 \end{cases} \tag{3-17}$$

$$PC_{WB(D)} = \begin{cases} 0 & \text{for } \overline{SS}_D = 0 \\ \dfrac{SS_{GOAL(D)}}{\overline{SS}_D} & \text{for } \overline{SS}_D > 0 \end{cases} \tag{3-18}$$

where $\overline{SS}_S$ and $\overline{SS}_D$ are the respective mean values of the daily mean surface storage water depths in shallow and deep wetland areas (ft) over the simulation period; and $SS_{GOAL(S)}$ and $SS_{GOAL(D)}$ represent respective mean goal depths (ft) for shallow and deep water wetland areas. $PC_{WB(S)}$ and $PC_{WB(D)}$ are the resulting shallow and deep water depth PC values. Within the context of the current study, it was assumed that neither $SS_{GOAL(S)}$ nor $SS_{GOAL(D)}$ could be set to zero. Therefore, if either $\overline{SS}_S$ or $\overline{SS}_D$ were equal to zero, the resulting PC value was also equal to zero. $\overline{SS}_S$ and $\overline{SS}_D$ were defined accordingly:

$$\overline{SS}_S = \frac{\sum_{i=DOY1}^{n} SS_{S(i)}}{n} \tag{3-19}$$

$$\overline{SS}_D = \frac{\sum_{i=DOY1}^{n} SS_{D(i)}}{n} \tag{3-20}$$

where $SS_{S(i)}$ is the daily mean shallow water depth for DOY *i* (ft), $SS_{D(i)}$ is the daily mean deep water depth for DOY *i* (ft), and *n* represents the total number of days of record. The resulting PTM relationship should depend on the wetland sensitivity to water depths, which can be a function of vegetation needs, wetland aquatic life needs (e.g., target waterfowl or fish species), water quality performance (certain depths may be required for sufficient particle settling, etc.). As an example, a simple parabolic shape was given to both wetland water balance PTMs:

$$M_{WB(S)} = \begin{cases} -PC_{WB(S)}^2 + 2 \cdot PC_{WB(S)} & \text{for } PC_{WB(S)} \le 2 \\ 0 & \text{for } PC_{WB(S)} > 2 \end{cases} \tag{3-21}$$

$$M_{WB(D)} = \begin{cases} -PC_{WB(D)}^2 + 2 \cdot PC_{WB(D)} & \text{for } PC_{WB(D)} \le 2 \\ 0 & \text{for } PC_{WB(D)} > 2 \end{cases} \tag{3-22}$$

where $M_{WB(S)}$ and $M_{WB(D)}$ are the resulting shallow and deep water wetland water balance metrics. Figure 3-5 plots the PTM functions used to relate both of the wetland water balance PC values with their corresponding metrics.

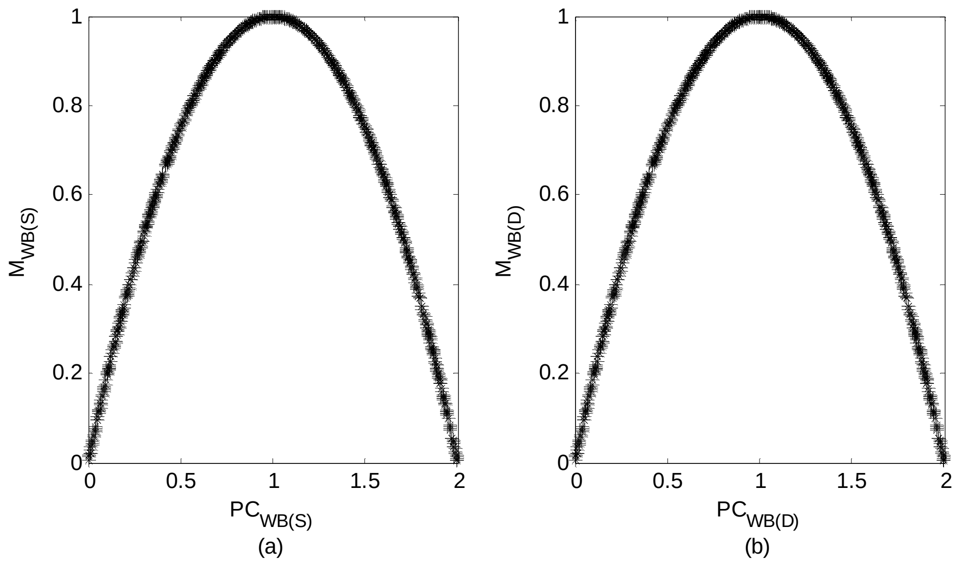

Figure 3-5 PTM functions relating (a) $PC_{WB(S)}$ and $M_{WB(S)}$, and (b) and $PC_{WB(D)}$ $M_{WB(D)}$.

3.4.5 Groundwater Recharge and Baseflow Maintenance

The goal of the groundwater recharge function is to restore groundwater recharge volumes to pre-development levels. Similar to the groundwater recharge function, the baseflow maintenance wetland function aims to restore baseflow rates and volumes to pre-development values. Optimally, the wetland should account for all groundwater and baseflow lost from impervious areas within the contributing drainage area. In other words, the wetland should ideally infiltrate the same volume of water as would the pre-developed drainage area. Given this goal, the total mean annual infiltration volume from the wetland was compared to the estimated mean annual infiltration volume of the simulated pre-developed drainage area. Within the model, pre-development runoff and infiltration for each storm event were separated based on a user-assigned pre-development rational C value. The pre-developed drainage area included both the contributing drainage area to the wetland and the wetland area. In order to develop a final groundwater recharge and baseflow maintenance performance criteria the mean annual pre-development and wetland infiltration volumes were compared in ratio form:

$$PC_{GW} = \begin{cases} \dfrac{\bar{I}_{PRE}}{\bar{I}_{KV}} & \text{for } \bar{I}_{KV} > 0 \\ 0 & \text{for } \bar{I}_{KV} = 0 \end{cases} \tag{3-23}$$

where $\bar{I}_{KV}$ is the mean annual infiltration volume (ft^3) over the simulation period, $\bar{I}_{PRE}$ is the mean annual infiltration volume (ft^3) over the simulation period from the pre-developed drainage area, and PC_{GW} represents the groundwater and baseflow maintenance PC. When PC_{GW} is equal to 1, the wetland fully compensates for all groundwater and baseflow lost to imperviousness in the contributing drainage area. If $\bar{I}_{KV}$ is greater than $\bar{I}_{PRE}$, more groundwater and baseflow will be input to receiving natural areas than under pre-developed conditions. Conversely, if $\bar{I}_{KV}$ is less than $\bar{I}_{PRE}$, and the resulting PC_{GW} is greater than 1, the wetland contributes less water to downstream baseflow and groundwater than would the pre-developed drainage area.

From both hydrologic and economic perspectives, excess groundwater and baseflow may be disadvantageous, costing more and possibly causing problems downstream. Therefore, a simple parabolic PTM function was used to relate PC_{GW} and the M_{GW}:

$$M_{GW} = \begin{cases} -PC_{GW}^2 + 2 \cdot PC_{GW} & \text{for } PC_{GW} \le 2 \\ 0 & \text{for } PC_{GW} > 2 \end{cases} \tag{3-24}$$

A plot of the groundwater PTM is shown in Figure 3-6. In cases where infiltration is not desired or not possible (e.g., areas with high water tables, wetlands in which groundwater contamination is a concern, etc.), the resulting PC_{GW} and M_{GW} will equal to zero. A number of constructed wetland designs actually require the installation of impermeable liners (USEPA 2000; MDE 2009). Due to this restriction, constructed wetlands often do not contribute to the groundwater recharge and baseflow maintenance sustainability function, producing PC_{GW} and the M_{GW} values of zero.

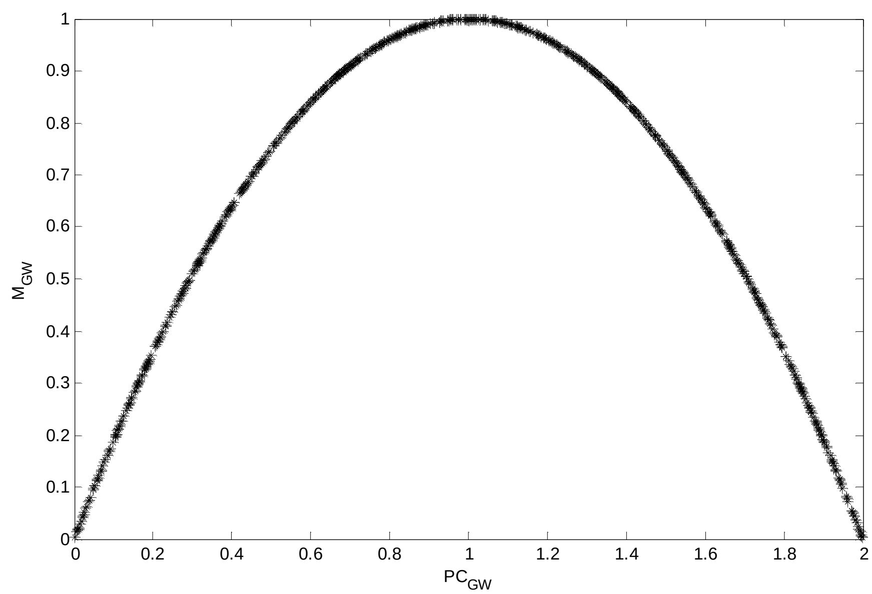

Figure 3-6 PTM relationship relating PC_{GW} and M_{GW}.

3.4.6 Aesthetics

The overall goal of the wetland aesthetics function is to create a wetland that is visually appealing to the surrounding community. The aesthetic appeal of the wetland was assumed to depend on the wetland design parameters. A more irregular wetland perimeter, for example, is more attractive because it creates a more interesting, natural-looking landscape (Smarden 1983). Three wetland design features were used to evaluate wetland aesthetic performance, which included (1) wetland perimeter irregularity, (2) wetland-type diversity, and (3) wetland area. These three characteristics were evaluated using the rating system suggested by Smarden (1983). The corresponding PC values were calculated accordingly (Smarden 1983):

$$PC_{PI} = \frac{PR}{2\sqrt{\pi(43{,}560 \cdot A_w)}} \tag{3-25}$$

$$PC_{WD} = N \tag{3-26}$$

$$PC_A = A_w \tag{3-27}$$

where PR represents the total wetland perimeter (ft.); A_w represents the total wetland surface area (ac); N is the number of different wetland types (defined by vegetation and water depths) present; PC_{PI} represents the perimeter irregularity PC value, which is a ratio of the actual wetland perimeter to the circumference of the wetland if it were perfectly circular; PC_{WD} is the PC value for wetland-type diversity; and PC_A is the PC value corresponding to total wetland surface area. A relationship between the aesthetic PC values and final aesthetic metrics were then adapted from Smarden (1983):

$$M_{PI} = \begin{cases} \dfrac{PC_{PI}}{5} & \text{for} \quad 0 \le PC_{PI} \le 5 \\ 1 & \text{for} \quad PC_{PI} > 5 \end{cases} \tag{3-28}$$

$$M_{WD} = \begin{cases} \dfrac{PC_{WD}}{5} & \text{for} \quad 0 \le PC_{WD} \le 5 \\ 1 & \text{for} \quad PC_{WD} > 5 \end{cases} \tag{3-29}$$

$$M_{A} = \begin{cases} \dfrac{PC_{A}}{10} & \text{for} \quad 0 \le PC_{A} \le 10 \\ 1 & \text{for} \quad PC_{A} > 10 \end{cases} \tag{3-30}$$

where M_{PI}, M_{WD}, and M_{A} represent the corresponding metric values for evaluating wetland perimeter irregularity, wetland-type diversity, and overall surface area. Smarden (1983) established the linear nature of all three aesthetic PTM relationships. Plots of all three aesthetic PTM relationships are shown in Figure 3-7.

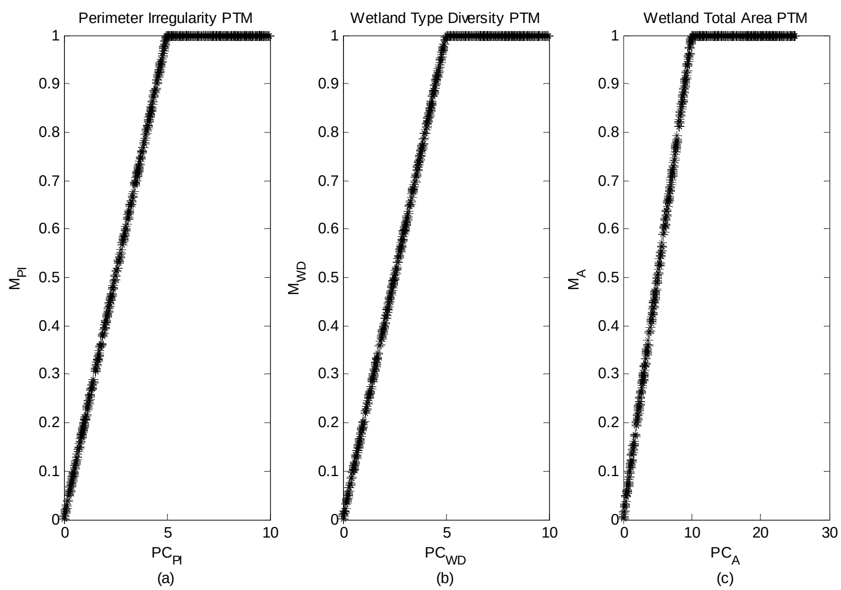

Figure 3-7 Aesthetic wetland function PTM relationships from for perimeter irregularity (a), wetland type diversity (b), and total wetland area (c).

3.4.7 Water Quality

The overall goal of the water quality wetland function was to produce effluent TSS, DO, NH_4^+, and NO_3^- concentrations that met the requirements for a given wetland design. Such water quality requirements were found to vary notably based on a wetland's intended use. For example, wetlands whose outflow feeds directly into natural streams may require different water quality standards than wetlands intended to serve as secondary treatment in a wastewater treatment plant. In the literature, disagreements occur over differences between healthy and detrimental nutrient and TSS concentrations, which makes setting water quality standards difficult. Additionally, TSS, DO, NH_4^+, and NO_3^- concentrations sufficient for one stream could be detrimental to a different stream depending on aquatic life needs and stream properties. In order to address these complicating issues, the current model developed example water quality metrics for (1) stormwater wetlands treating water in the mid-Atlantic region, (2) constructed wetlands used as secondary municipal wastewater treatment, and (3) constructed wetlands treating agricultural wastewater. Given the variability of wetland effluent water quality needs, water quality metrics should be developed for each wetland design individually in order to ensure the best performance.

3.4.7.1 Stormwater Water Quality in the Mid-Atlantic region

The goal of the water quality function in the context of stormwater wetlands in the mid-Atlantic region was to input healthy water quality levels into receiving natural water bodies. These water quality levels should mimic those found in healthy, analogous receiving water bodies. A total of four stormwater water quality performance criteria were developed within the current study to evaluate wetland effluent dissolved oxygen (DO), TSS, nitrate nitrogen (NO_3^--N), and ammonia nitrogen (NH_4^+-N) concentrations. Each of the four resulting performance criteria was related to a corresponding metric based on its relative impact on downstream water quality and aquatic health.

Water quality PC values were set equal to the daily mean effluent TSS, DO, NH_4^+, and NO_3^- concentrations simulated for a given wetland design. These daily mean effluent concentrations were calculated using a daily mean concentration (DMC) method. Water quality loads were summed over each day of simulation and divided by the corresponding total daily outflow volume in order to calculate the DMC for a given day:

$$DMC = \frac{\sum_{i=1}^{24} L_{OUT}}{\sum_{i=1}^{24} V_{OUT}} \tag{3-31}$$

where L_{OUT} represents the minute-by-minute outflow load of a given water quality constituent within a given day (mg) and V_{OUT} is the minute-by-minute outflow volume within a given day (L). All DMC values for each water quality constituent were averaged over the simulation period to determine the final mean outflow concentrations for a given wetland design. Zero-flow days were excluded from these final water quality means in order to avoid a negative skew in effluent concentrations. The DMC, as opposed to the event mean concentration (EMC), was employed in the current study in order to avoid inaccuracies and complications associated with differentiating water quality loads and flows associated with separate events within the model, which had a continuous rather than event-based structure.

Because the model was initially calibrated for the mid-Atlantic region of the United States, all water quality metrics were based on water quality concentrations and associated benthic invertebrate health in North Carolina as calculated by McNett et al. (2010). This study collected both the ambient water quality samples (AWQ) and benthos ratings (BMR) for 106 streams in the Piedmont region of North Carolina over a 30-year study period. Water quality concentrations were taken as grab samples and were only collected during summer months, which were thought to produce conservative values as water quality tends to decline in warmer months (McNett et al. 2010). Within this study, the following five BMR categories were used: excellent, good, good-fair, fair, and poor to evaluate benthic health (McNett et al. 2010). Each category was defined by the sensitivity of benthic species present in a stream; all sensitive species present translated to an excellent BMR while an absence of sensitive species translated to a BMR of poor. McNett et al. (2010) then collected AWQ samples from all 106 streams and compiled the mean AWQ values corresponding to each BMR category; results from this study are shown in Table 3-2.

The current study assigned metric values to each of the five BMR categories, defined by McNett et al. (2010), which were respectively 1, 0.83, 0.67, 0.50, and 0.30 for BMR categories of excellent, good, good-fair, fair, and poor. Therefore, water quality metric values increased with increasing associated stream benthic health. Curves were also generated from the resulting metric values for each BMR category and associated AWQ values. TSS, DO and NO_3^- PTM curves all followed a power model, while the NH_4^+ PTM curve was found to best fit an exponential curve. These developed curves for TSS, DO and NO_3^- are plotted with the AWQ values with which they were fit in Figure 3-8. These curves were further extrapolated to account for pollutant concentrations outside of the ranges observed in Figure 3-8 to develop final corresponding water quality PTM functions.

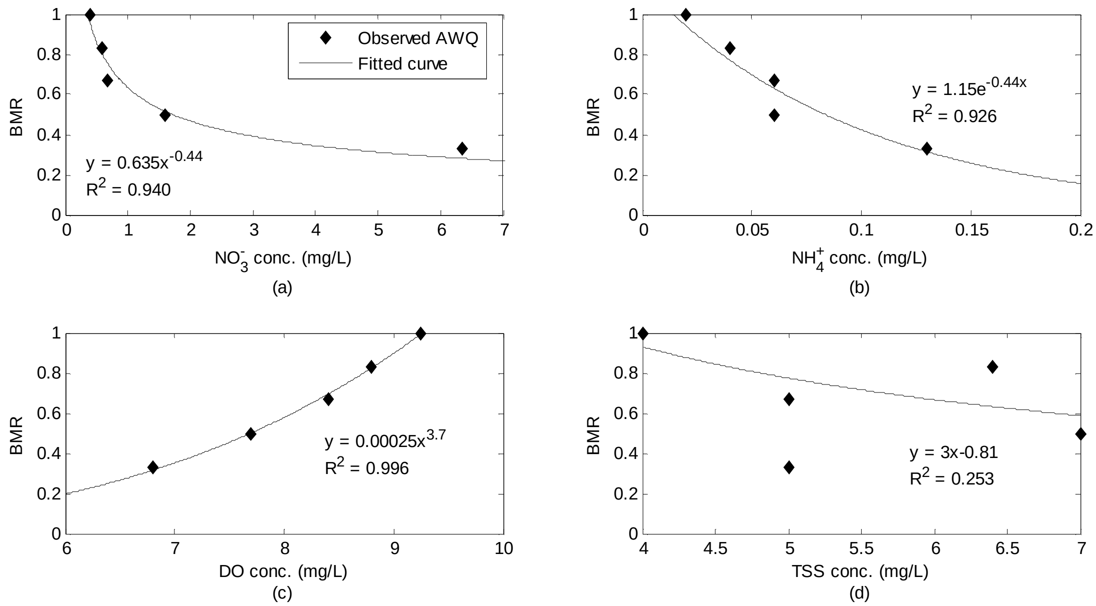

Figure 3-8 Curves fit to the AWQ data reported by McNett et al. (2010) vs, the estimated numerical metrics corresponding to the BMR categories defined by McNett et al. (2010) for (a) NO_3^-, (b) NH_4^+, (c) DO, and (d) TSS. Equations and adjusted R^2 are shown with each curve.

Table 3-2 McNett et al. (2010) BMR levels for DO, TSS, NO_3^-, and NH_4^+ concentrations in streams in the Piedmont region in the mid-Atlantic. Corresponding metric values assigned by the current study to each BMR category are also shown.

Rating	**Assigned metric**	**DO (mg/L)**	**TSS (mg/L)**	**NO_3^-/ NO_2^- (mg/L)**	**NH_3/ NH_4^+ (mg/L)**
Excellent	1	9.25	4	0.39	0.02
Good	0.83	8.8	6.4	0.59	0.04
Good-fair	0.67	8.4	5	0.67	0.06
Fair	0.50	7.7	7	1.6	0.06
Poor	0.33	6.8	5	6.34	0.13

Both NO_3^- and NH_4^+ species were used to assess wetland performance with respect to nitrogen. The following expressions were developed from the results of the McNett et al. (2010) study to model the NO_3^- and NH_4^+ PTM's:

$$M_{NO3} = \begin{cases} 0.635 \cdot NO3^{-0.44} & \text{for NO3} \geq 0.39 \\ 1 & \text{for NO3} < 0.39 \end{cases} \tag{3-32}$$

$$M_{NH4} = \begin{cases} 1.15 \cdot \exp(-9.96 \cdot NH4) & \text{for NH4} \geq 0.02 \\ 1 & \text{for NH4} < 0.02 \end{cases} \tag{3-33}$$

where $NO3$ and $NH4$ respectively represent the final mean outflow concentrations of NO_3^- and NH_4^+ (mg/L) over the simulation period; and M_{NO3} and M_{NH4} are the respective $NO_3^-$and NH_4^+ metrics. These resulting NO_3^- and NH_4^+ PTMs had respective adjusted R^2 values of 0.940 and 0.926, which represented strong agreement with the BMR data provided by McNett et al. (2010) in Table 3-2. The respective NO_3^- and NH_4^+ limits of 0.39 and 0.02 mg/L represented the concentrations corresponding to the Excellent BMR level (see Table 3-2). Therefore, NO_3^- and NH_4^+ concentrations below these levels were assumed to contribute to excellent benthic habitat conditions. Final NO_3^- and NH_4^+ PTM plots are shown in Figure 3-9 and Figure 3-10.

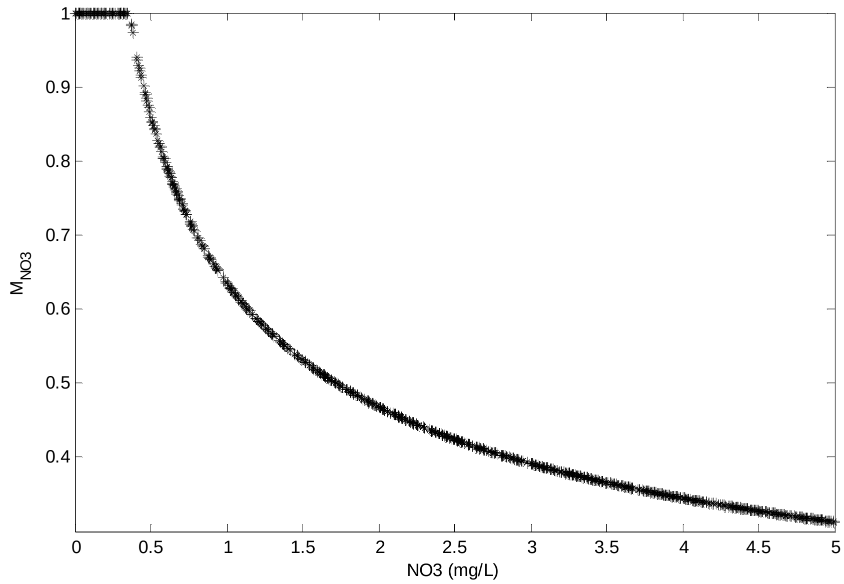

Figure 3-9 PTM function relating NO_3^--N concentration and the final NO_3^- metric M_{NO3}.

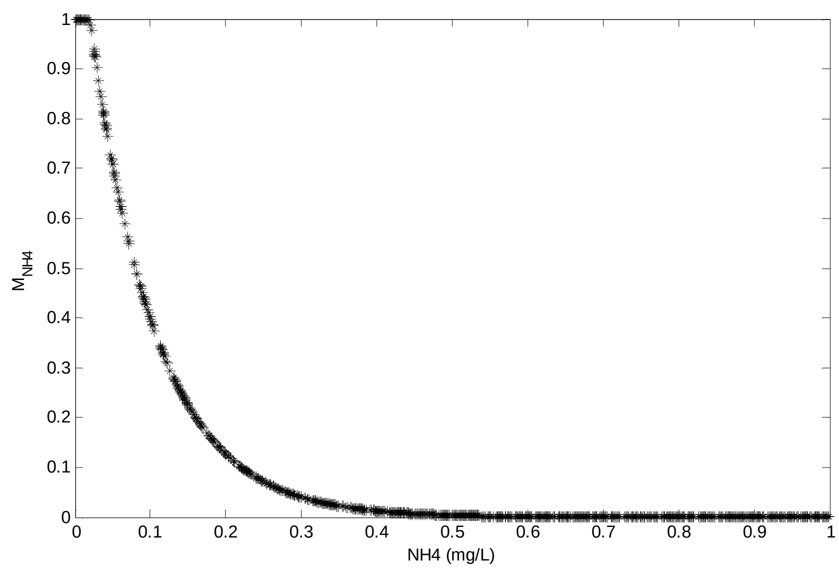

Figure 3-10 PTM function relating NH_4^+-N concentration and the final NH_4^+ metric M_{NH4}.

Dissolved oxygen (DO) is an extremely important water quality parameter. All aquatic aerobic activities, by definition, require DO. Ammonification, nitrification, and BOD degradation also require DO. Without sufficient DO levels, an aquatic ecosystem can degrade rapidly. The following expression was used to evaluate wetland DO performance as based on results obtained from McNett et al. (2010):

$$M_{DO} = \begin{cases} 0.00025 \cdot DO^{3.7} & \text{for DO} \leq 9.25 \\ 1 & \text{for DO} > 9.25 \end{cases} \tag{3-34}$$

where DO represents final mean outflow dissolved oxygen concentration (mg/L) and M_{DO} is the resulting dimensionless DO metric. The upper limit of 9.25 mg/L represented the DO concentration corresponding to an Excellent BMR DO metric (see Table 3-2) under the assumption that all DO concentrations exceeding 9.25 mg/L sustained a healthy benthic ecosystem. Figure 3-11 shows this PTM relationship, which had an adjusted R^2 of 0.996.

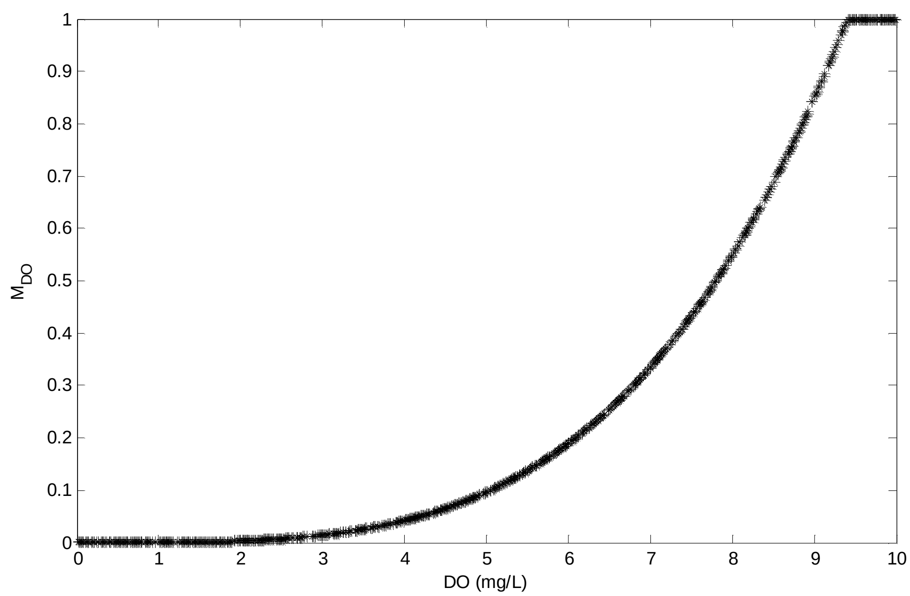

Figure 3-11 PTM function relating DO and the final DO metric M_{DO}.

Elevated TSS concentrations increase turbidity and, therefore, can reduce submerged macrophyte photosynthesis. In addition, a number of pollutants are associated with TSS including BOD and TP. Therefore, as TSS increases, BOD, and TP also potentially increase, which can cause DO to decrease. TSS can also carry beneficial solids downstream. Therefore, some level of TSS reflects good water quality. However, once a certain concentration is reached, TSS begins to degrade water quality. The threshold at which TSS concentrations become detrimental to a stream can vary great depending on a number of factors including stream bed composition, stream flow velocity, stream habitat types, etc. As a result of this variability, TSS concentrations exhibited the weakest relationship with benthic health of all of the water quality constituents analyzed in McNett et al. (2010). Despite this weak relationship, the current study assumed that within an individual stream, increasing TSS concentrations would result in a general trend of worsening water quality. Therefore, despite the large variability in healthy stream TSS concentrations, the final TSS PTM was adapted from McNett et al. (2010) accordingly:

$$M_{TSS} = \begin{cases} 3 \cdot TSS^{-0.81} & \text{for TSS} \geq 4 \\ 1 & \text{for TSS} < 4 \end{cases} \tag{3-35}$$

where TSS represents the final mean outflow TSS concentration (mg/L) and M_{TSS} is the corresponding TSS metric. The limit of 4 mg/L represented the TSS concentration corresponding to an Excellent BMR TSS metric (see Table 3-2) under the assumption that all TSS concentrations below 3 mg/L promoted a healthy benthic ecosystem. This TSS PTM function had an adjusted R^2 of 0.253, which is very poor due to the variability in "healthy" TSS concentrations for different streams. Figure 3-12 shows the resulting TSS PTM.

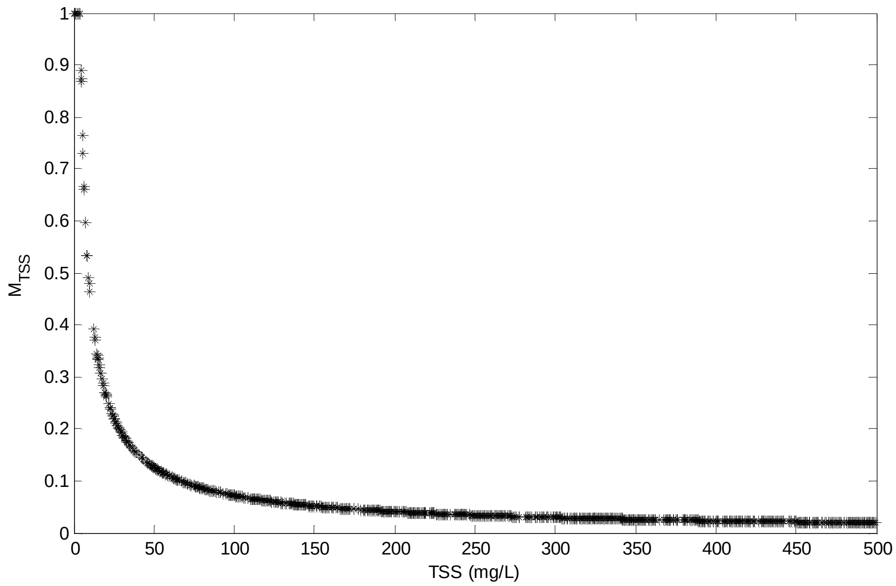

Figure 3-12 PTM function relating TSS and the final TSS metric M_{TSS}.

3.4.7.2 Municipal wastewater treatment effluent water quality

Water quality metrics were defined for wastewater treatment wetlands that perform secondary treatment based on EPA effluent secondary treatment water quality standards. EPA requires respective 30-day mean secondary treatment effluent concentrations for TSS and TKN (Total Kjeldahl Nitrogen) of 30 and 10 mg/L (USEPA 2000; USEPA 2010). Additionally, the USEPA requires that the 7-day mean TSS concentration for secondary treatment are below 45 mg/L. Note that EPA does also list additional standards for constituents not simulated by the current model (i.e., the 30-day average BOD concentration must be under 30 mg/L and the 7-day average BOD concentrations must be under 45 mg/L).

All mean daily effluent concentrations for municipal wastewater treatment wetlands were calculated according to Equation 3-31 within the model. In order to compute 7- and 30- day means, the model averaged the daily concentrations for each consecutive 7- and 30- day period within a given simulation. If the mean effluent concentration for the average 7- or 30-day TSS concentration was found to be greater than the required level, the corresponding metric was related linearly to the resulting TSS concentration. Metric values of zero were assigned to 7- and 30-day TSS effluent concentrations that were equal to or greater than corresponding influent TSS concentrations. The current study assumed a constant influent TSS concentration of 59.5 mg/L, which was based on data provided by a local WWTP. Therefore, the resulting TSS metrics were computed based on an influent TSS concentration of 59.5 mg/L. All resulting concentrations below the required standards were assigned a metric value of 1.0. Therefore, the PTM curve for average effluent 30-day mean TSS concentration took on the following simple stepwise form:

$$M_{T(30)} = \begin{cases} 1 & \text{for } TSS_{30} \leq 30 \text{ mg/L} \\ -0.0339 \cdot TSS_{30} + 2.02 & \text{for } TSS_{30} > 30 \text{ mg/L} \end{cases} \tag{3-36}$$

where TSS_{30} is the average 30-day mean wetland effluent concentration (mg/L) of TSS and $M_{T(30)}$ is the corresponding TSS metric for secondary treatment wetlands. Similarly, a 7-day TSS metric was computed:

$$M_{T(7)} = \begin{cases} 1 & \text{for } TSS_{7} \leq 45 \text{ mg/L} \\ -0.0699 \cdot TSS_{7} + 4.16 & \text{for } TSS_{7} > 45 \text{ mg/L} \end{cases} \tag{3-37}$$

where TSS_7 is the average 7-day mean wetland effluent concentration (mg/L) of TSS and $M_{T(7)}$ is the corresponding TSS metric for secondary treatment wetlands. While linear PTM relationships were defined to relate TSS_{30} and TSS_7 with $M_{T(30)}$ and $M_{T(7)}$, the shape of these functions is subject to change based on user needs as well as the sustainability implications of different effluent TSS concentrations for a given wetland design. The current study chose linear functions to illustrate the general decreasing trend of wetland sustainability with increasing TSS concentrations.

An effluent TKN metric was also developed for secondary treatment wetlands. Because wastewater treatment wetland TKN performance was not as reliable as its TSS performance, strict TKN requirement have not been established for wetland effluents serving as secondary treatment in a municipal WWTP. However, a TKN effluent 30-day average concentration of 10 mg/L was cited by the USEPA (2000) as an ambitious goal. Therefore, a PTM function was developed that returned a TKN metric M_{TKN} of 1.0 for an average effluent 30-day mean TKN concentration less than or equal to 10 mg/L and a M_{TKN} of 0.0 for an average 30-day effluent TKN concentration greater or equal to corresponding influent concentrations. A linear function was used to define M_{TKN} values with concentrations between 10 mg/L and wetland influent concentrations. While a linear function was used in the current study for simplicity, if more data were available to better define the sustainability impacts of increasing TKN concentrations, the TKN PTM function could be altered accordingly.

The current study designed a municipal wastewater treatment wetland with an estimated TKN influent concentration of 47.1 mg/L. This influent TKN concentration was estimated by multiplying the influent NH_4^+ concentration of 28.3 mg/L by a factor of 2.11. This correction factor of 2.11 represented the ratio between the average influent TKN (28.3 mg/L) and NH_4^+ (13.4 mg/L) concentrations reported by USEPA (2000) for 22 wastewater treatment wetlands, which are reproduced in Table 2-11. While a large amount of error can be associated with this TKN estimation method, it was assumed sufficient within the context of the current study given the lack of necessary data throughout the literature. Based on the influent TKN concentration of 47.1 mg/L, the resulting PTM function for M_{TKN} was defined accordingly:

$$M_{TKN} = \begin{cases} 1 & \text{for } TKN_{30} \leq 10 \text{ mg/L} \\ 1.27 - 0.0270 \cdot TKN_{30} & \text{for } 10 < TKN_{30} \leq 47.1 \text{ mg/L} \\ 0 & \text{for } TKN_{30} > 47.1 \text{ mg/L} \end{cases} \tag{3-38}$$

where TKN_{30} is the average 30-day mean wetland effluent concentration (mg/L) of TKN. The resulting TKN PTM function is shown in Figure 3-13. Effluent TKN_{30} concentrations were estimated by multiplying the average 30-day effluent NH_4^+ concentration by a factor of 1.59 as estimated by respective mean TKN (19 mg/L) and NH_4^+ (12 mg/L) effluent concentrations reported by USEPA (2000) for 22 municipal free surface water wetlands treating primary effluent (see Table 2-11). Figure 3-13 shows the PTM functions relating all three municipal wastewater PC values with their corresponding metrics.

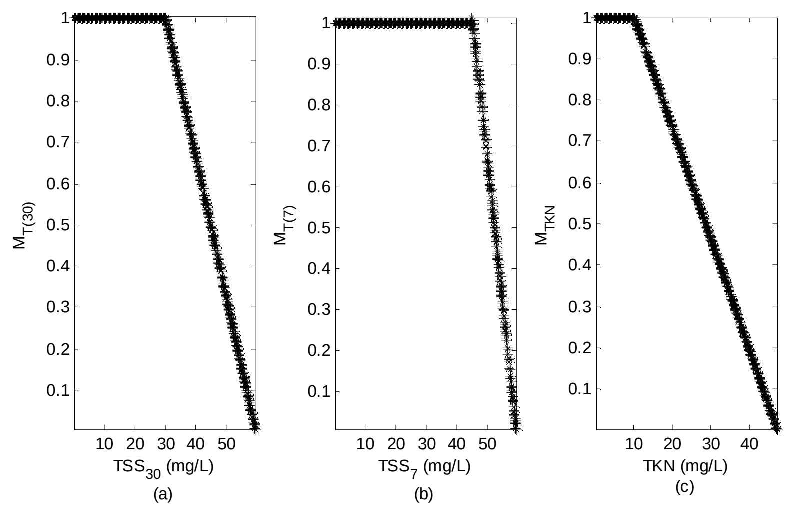

Figure 3-13 PTM function relating (a) TSS_{30} and $M_{T(30)}$, (b) TSS_7 and $M_{T(7)}$, and (c) TKN_{30} and M_{TKN}.

3.4.7.3 Agricultural wastewater treatment effluent quality

Agricultural wastewater quality metrics should be defined for each design based on the nutrient needs of the receiving cropland. From the example NRCS treatment wetland for swine wastewater used in Section 8.1, the receiving cropland required a daily average effluent concentration of 162 mg/L of total nitrogen (TN). Other water quality requirements were not suggested by the NRCS (2002). Therefore, because this nutrient requirement is more flexible, a parabolic shape was given to the final TN metric for agricultural wastewater in this case:

$$M_{TN} = \begin{cases} -3.81\times10^{-5}\cdot TN^2 + 0.0124\cdot TN & \text{for } 0 \le TN \le 324 \text{ mg/L} \\ 0 & \text{for } TN > 324 \text{ mg/L} \end{cases} \tag{3-39}$$

where TN is the mean daily wetland effluent concentration (mg/L) of total nitrogen, and M_{TN} is the resulting agricultural wastewater TN metric. For the purpose of this example, TN effluent concentrations were estimated to be equal to about 1.12 times effluent NH_4^+ concentrations based on values reported by Knight et al. (2000) for 19 swine wastewater treatment wetlands (see Table 2-13). The PTM function relating TN and M_{TN} is plotted in Figure 3-14. Future versions of the model should simulate all nitrogen species as to better evaluate this metric. Additional water quality metrics could be added and altered based on the requirements of a specific design.

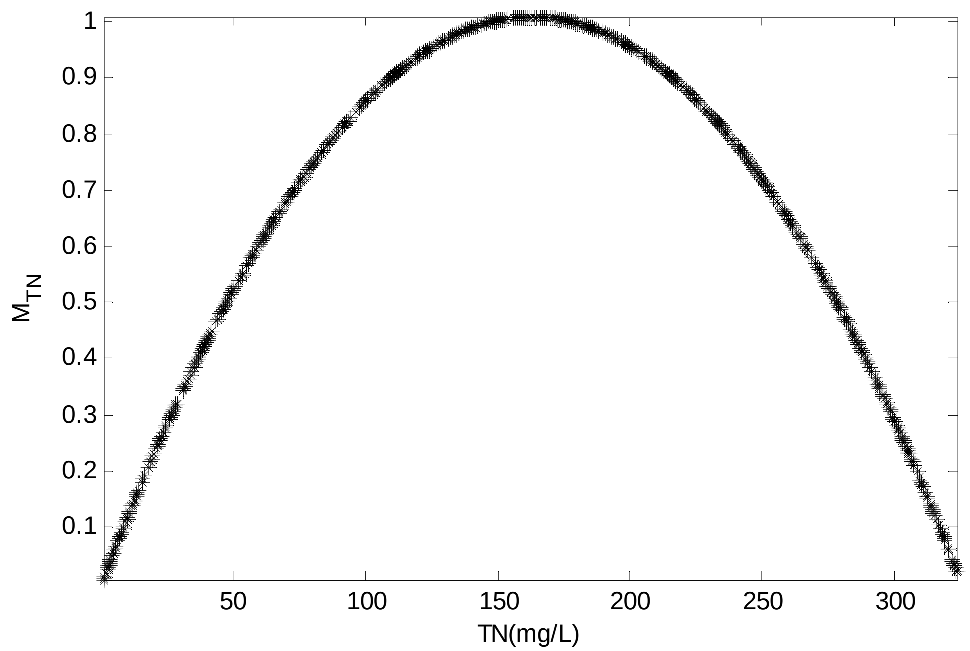

Figure 3-14 PTM function relating TN and M_{TN} .

3.5 FINAL SUSTAINABILITY INDEX

Final metrics were multiplied by stakeholder assigned weights and summed to determine the overall wetland sustainability index (WSI). Metric weights were based on stakeholder judgment. Once all metrics and their corresponding weights were agreed upon, the following equation was used to determine an overall, weighted sustainability value for the entire wetland:

$$WSI = \frac{w_1 M_1 + w_2 M_2 + \cdots + w_n M_n}{w_1 + w_2 + \cdots + w_n} \tag{3-40}$$

where w_n refers to the stakeholder assigned weight corresponding to the metric M_n, and WSI is the resulting wetland sustainability index. The weights are subject to the constraint that they must sum to 1:

$$\sum w_i = 1 \tag{3-41}$$

Metrics were assumed to be independent. However, given that a number of metrics such as those used to quantify the wetland hydrologic and water quality functions were correlated, further applications of these metrics should explore this issue of correlation.

Metrics could also be assigned "design failure" limits where failure to meet a given criteria results in a failed design and an overall sustainability index of 0 (e.g., if ammonia levels are too high, that design is not feasible because downstream aquatic life would be killed). Depending on the criteria that are most important to the stakeholders, these design failure limits could be assigned to different criteria and at different levels of consequence. Extreme 'optimal' wetland designs can result from strong stakeholder bias in chosen metrics. Therefore, careful attention must be paid to reducing such bias in metric development. Final metric values should reflect all stakeholder values while emphasizing sustainability with respect to a given criterion and its corresponding wetland function.

3.6 SUSTAINABILITY METRIC USER INPUTS

A total of six user inputs were required to compute the sustainability metrics discussed in this chapter. All user inputs associated with sustainability metrics are listed in Table 3-3. These inputs allowed the user to directly compute sustainability performance criteria within the model. Corresponding metrics were computed outside of the model as their relationship to performance criteria could be user-defined.

Table 3-3 All sustainability metric user inputs and their associated performance metric.

User input	Units	Associated performance metric
Wetland perimeter	ft	Wetland aesthetics perimeter metric M_{PI}
Number of wetland habitat types	---	Aesthetics wetland-type diversity metrics M_{WD}
Number of habitat islands	---	Habitat island metric M_{H2}
High-marsh design depth	ft	High-marsh wetland water balance metric $M_{WB(H)}$
Low-marsh design depth	ft	Low-marsh wetland water balance metric $M_{WB(L)}$
Pre-developed drainage area Rational C	---	Low- and high-flow metrics $M_{H(H)}$ and $M_{H(L)}$

Chapter 4: Model Development

4.1 WETLAND VEGETATION

Constructed wetlands for treatment typically consist of two main wetland regions, (1) areas with emergent vegetation and shallower water depth, and (2) areas with submerged and/or floating vegetation and deeper water depths (see Figure 2-1). Both areas played important roles in the function of a given wetland design based on their relative water depths and vegetation types. While algae also plays a role in wetland function, it was not simulated within the model due to the complexities introduced by the associated growth, death, and advection cycles.

Emergent vegetation is typically dense and able to stand erect out of the water, requiring water depths of less than 1 m (3.3 ft). Common examples of emergent vegetation include *Typha* species (cattails), *Scirpus* species (bulrush), and *Phragmites* species (common reed). They provide surface area for microbial activity, enhance flocculation and sedimentation, provide cover from wind, and insulate wetland water temperatures during the winter (USEPA 2000). Due to the rigid nature of most emergent species, these areas of high marsh also create a laborious pathway for water flowing through the wetland, resulting in low water velocities. Because emergent vegetation generally reaches above the water surface, it can also provide significant transpiration. The current study used the relative height of emergent vegetation above mean high-marsh water depths z_v (m) for a given design as an input to potential evapotranspiration (PET) calculations (see Section 0). Emergent vegetation was also assumed not to contribute to water oxygen levels via photosynthesis as most of the leaves are above the water surface.

Submerged vegetation is rooted to the bottom of the wetland, generally fully submerged, cannot stand erect in air, and is present in water depths between 0.25 and 3 m (0.80 and 10 ft) (Kadlec and Knight 1996; USEPA 2000). Examples of submerged species include *Potamogen* species (pondweed) and *Elodea* species (water weed). Floating vegetation fits the same niche as submerged vegetation as they require water depths with a typical range of 0.25-3 m (0.80-10 ft), cannot stand erect in air, and can either be free floating or rooted to the bottom of the wetland with additional floating leaves. *Lemna* (common duckweed) and *Nymphea* (water lily) species are examples of common floating vegetation. Both submerged and floating vegetation provide oxygen to wetland water through photosynthesis and surface area for microbial activity. Additionally, floating vegetation may cause problems by blocking surface oxygen transfer as well as blocking sunlight from reaching submerged vegetation (Kadlec and Knight 1996; USEPA 2000). Floating vegetation was not included in the current model because it served the same functions as submerged vegetation with additional detrimental effects on water treatment.

Generally emergent vegetation coincides with shallower water depths while submerged vegetation was restricted to deeper water depths due to species water requirements and typical constructed wetland design guidelines. Therefore, shallow areas with emergent vegetation are generally associated with very low velocities, and lower oxygen levels (lower rates of surface aeration due to low velocities and no macrophyte photosynthesis). As a result of these lower oxygen values, the bottom soils of these shallow areas with emergent vegetation are also thought to be sites of denitrification (USEPA 2000; Bastviken 2006).

Conversely, deeper areas were associated with slightly higher velocities due to less dense vegetation and deeper water, and higher oxygen levels due to increased surface aeration and macrophyte photosynthesis. The aerobic nature of these deep areas made them supposed sites for nitrification. Later model calibration and analyses were used to evaluate the performance of both shallow areas with emergent vegetation and deep areas with submerged vegetation in different wetland designs.

4.2 POTENTIONAL EVAPOTRANSPIRATION (PET)

The Penman Monteith equation was used to simulate potential evapotranspiration (PET) from each wetland cell. PET is the combination of water transpiration through vegetation and direct evaporation from a water surface; both of which occur in a typical wetland system. Within the model, PET was assumed to be constant across all cells regardless of vegetation type or inclusion. This assumption does not distinguish between cells in which plant transpiration dominated and those in which water surface evaporation dominated. However, Kadlec and Knight (1996) pointed out that actual wetland ET is comparable to that of lake evaporation, which suggests that assuming a lump PET term may be a reasonable estimate of both transpiration and evaporation within a cell. PET was simulated on an hourly basis and was turned off during hours when rainfall occurred within the model. Actual evapotranspiration (AET) within a wetland cell was limited to the cell's existing surface water level at a given time interval. The current section outlines in detail the methods used to simulate PET within the model.

4.2.1 Input data for PET component

The following section explains the determination of the input parameters to the PET components. A number of assumptions and simplifications were made to ensure that the resulting model component was both user-friendly and computationally efficient. All such simplifications are discussed.

4.2.2 Constant PET inputs

A number of PET component inputs were assumed to be constant throughout a given simulation including albedo (a), shelter factor (f_s), maximum leaf conductance (C^*_{leaf}), emergent vegetation height above water (z_v), height of wind measurements (z_m), and atmospheric pressure (P = 101.32 kPa). The wetland albedo a represents the composite reflectivity of a wetland area. This collective wetland albedo may vary greatly based on season, water depth, snow cover, vegetation height, vegetation cover, and latitude (Goodin et al. 1996; Dingman 2002). The input z_v value was dependent on both the emergent vegetation height and the corresponding water depths in areas populated with emergent vegetation in a given wetland design. The maximum leaf conductance (C^*_{leaf}) represents the maximum rate (mm/s) at which the leaves of a given plant with will transfer water into the surrounding atmosphere. This maximum rate occurs when the leaf pores (stomata) are completely opened. Different vegetation species can have different C^*_{leaf} values due to varying leaf areas, stomata densities within each leaf, and stomatal opening size (Koch and Rawlik 1993; Morrissey 1993; Dingman 2002). The shelter factor f_s is a measure of how much shading occurs in the vegetation. A f_s value of 0.5 represents a vegetated surface with 50% of the leaves shaded, while a value of 1 represents an area in which no shading effects impede vegetation sun exposure. The variable z_m represents the standard height from which wind speed data are taken. The variable P is the standard air pressure at sea level. Depending on the location of a proposed wetland, P may change.

The leaf area index (LAI) was also a PET component input, and is a measure of leaf area relative to total surrounding area containing a plant or tree. A pine tree, for example, has a much lower LAI than a broad-leafed deciduous tree. The wetland vegetation LAI was assumed to change seasonally in order to simulate leaf loss in the fall and winter seasons. Wetland vegetation was assumed to have no leaves from September 21st through March 20th, which respectively correspond to the days of year (DOYs) 264 and 80. The following simple LAI model was constructed to mimic the resulting reduction of PET during fall and winter months:

$$LAI = \begin{cases} 0 & \text{for } d < 80 \\ \mathrm{LAI_g} & \text{for } 80 \le d \le 264 \\ 0 & \text{for } d > 264 \end{cases} \tag{4-1}$$

where *d* represents the day of year (DOY) and $\mathrm{LAI_g}$ represents the LAI during the growing season, which was assumed to occur during the spring and summer months. $\mathrm{LAI_g}$ was a user input. The piecewise structure of this model was taken from Federer et al. (1996).

4.2.3 Incident Solar Radiation Input

Incident solar radiation was estimated using data from the National Solar Radiation Database (NSRDB) as well as from methods covered in Dingman (2002). Hourly global solar radiation (K_{in}) averaged over each month of record from 1991 to 2010 was downloaded from NSRDB for Baltimore, MD as recorded at the Baltimore-Washington Thurgood Marshall International Airport. Global radiation represents the sum of direct (all radiation hitting the earth's surface from directly above) and diffuse (all radiation reaching the earth's surface from different angles due to scattering in the atmosphere) radiation. All radiation data were collected on a horizontal surface (Wilcox 2012). The resulting mean hourly radiation values for each month over the 20-year period of record are shown in Figure 4-1. These curves were developed by averaging the mean hourly radiation values for each month over the 20-yr period.

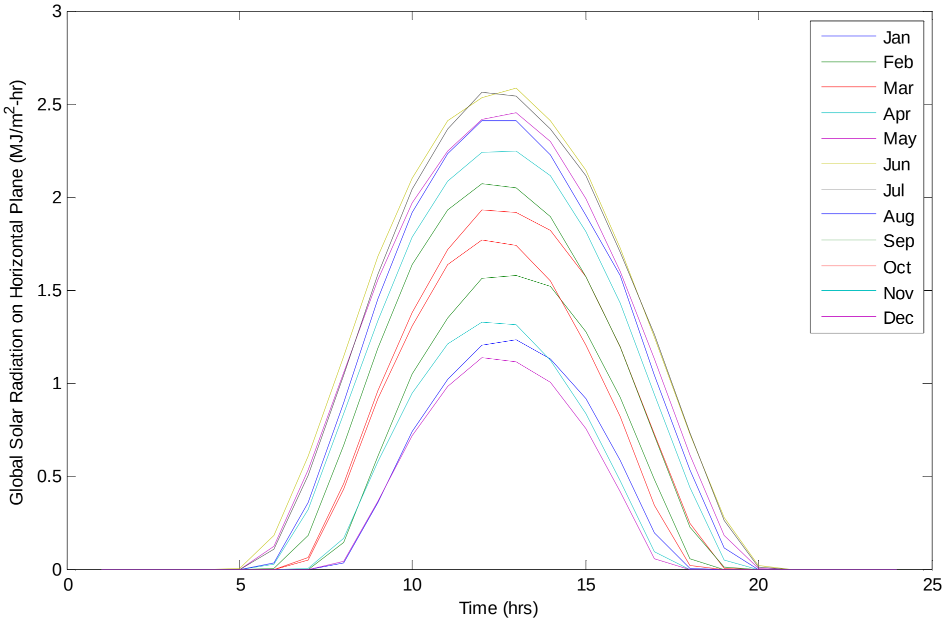

Figure 4-1 Hourly mean global solar radiation on a horizontal plane K_{in} for each month (MJ/m^2-hr) at Baltimore-Washington Thurgood Marshall International Airport over the period of record 1991-2010 (NSRDB)

From these monthly mean hourly values, a daily maximum radiation (K_{in}^{*}) curve was estimated:

$$K_{in}^{*}(d) = 1.92 + 0.72\sin[0.017(d-81)] + 0.072\sin[2 \cdot 0.017(d-30)] \qquad (4\text{-}2)$$

where *d* is the day of year DOY (*d* = 1 on January 1st). The resulting K_{in}^{*} curve (MJ/m^2-hr) estimated the peak hourly solar radiation for each day of the year, which was assumed to occur at solar noon each day. The solar noon for each day at latitude 38° (the estimated latitude of Maryland) was determined using an excel spreadsheet provided by NOAA which is downloadable at <http://www.esrl.noaa.gov/gmd/grad/solcalc/calcdetails.html>.

In order to determine solar radiation curves for each day of the year, as opposed to hourly values averaged over each month of the year as given by NSRDB, the sunset (T_{hs}) and sunrise (T_{hr}) times were calculated for each day of the year (Dingman 2002):

$$T_{hr} = -\frac{\cos^{-1}[-\tan(\delta)\cdot\tan(\Lambda)]}{\omega} \tag{4-3}$$

$$T_{hs} = +\frac{\cos^{-1}[-\tan(\delta)\cdot\tan(\Lambda)]}{\omega} \tag{4-4}$$

where Λ (radians) is the latitude (in this case 38° was used to represent Maryland), δ is the declination angle (radians), ω is the angular velocity of the earth's rotation (0.2618 radian/hr), and T_{hr} and T_{hs} represent the respective number of hours before and after solar noon at which the sun rises and sets on a given day. T_{hr} is negative and T_{hs} is positive and have the same magnitudes. Before sunrise and after sunset, solar radiation was assumed to equal zero.

In order to reduce computational expenses, a curve was fit to T_{hs} daily values:

$$\hat{T}_{hs}(d) = 6.0219 + 1.2836\sin[0.01721d - 1.414] \tag{4-5}$$

where $\hat{T}_{hs}$ represents the curve-predicted sunset time relative to the solar noon (hr) for each day of the year. Figure 4-2 shows the plots that compare the actual calculated T_{hs} values with the predicted curve $\hat{T}_{hs}$.

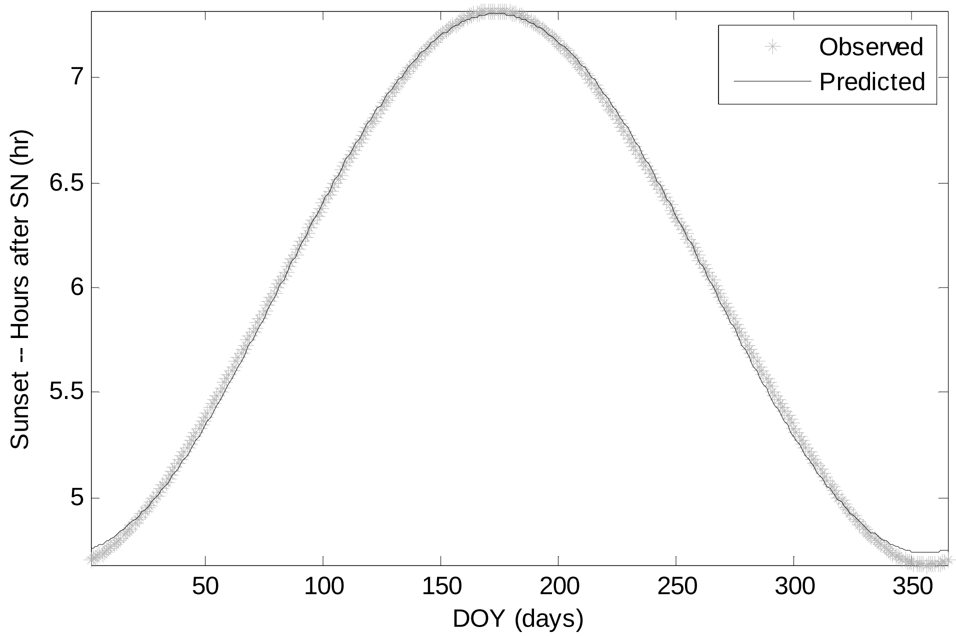

Figure 4-2 Observed T_{hs} (gray) and predicted $\hat{T}_{hs}$ (black) daily values over the course of one annual cycle.

The daily predicted K_{in}^{*} and $\hat{T}_{hs}$ values were then be used to estimate triangular $\hat{K}_{in}$ hourly distributions for each DOY. Because all solar noon values were within 20 min of 12pm and the primary time increment of the model was one hour, the solar noon was assumed to equal 12pm for all days. Sunrise (h_{sr}) and sunset (h_{ss}) times were, therefore, estimated to equal:

$$h_{sr}(d) = 13 - \hat{T}_{hs}(d) \tag{4-6}$$

$$h_{ss}(d) = 13 + \hat{T}_{hs}(d) \tag{4-7}$$

where *d* represents the DOY and 13 presents the hour of the solar noon, which corresponds to 12pm. The annual change in h_{sr} and h_{ss} are plotted in Figure 4-3.

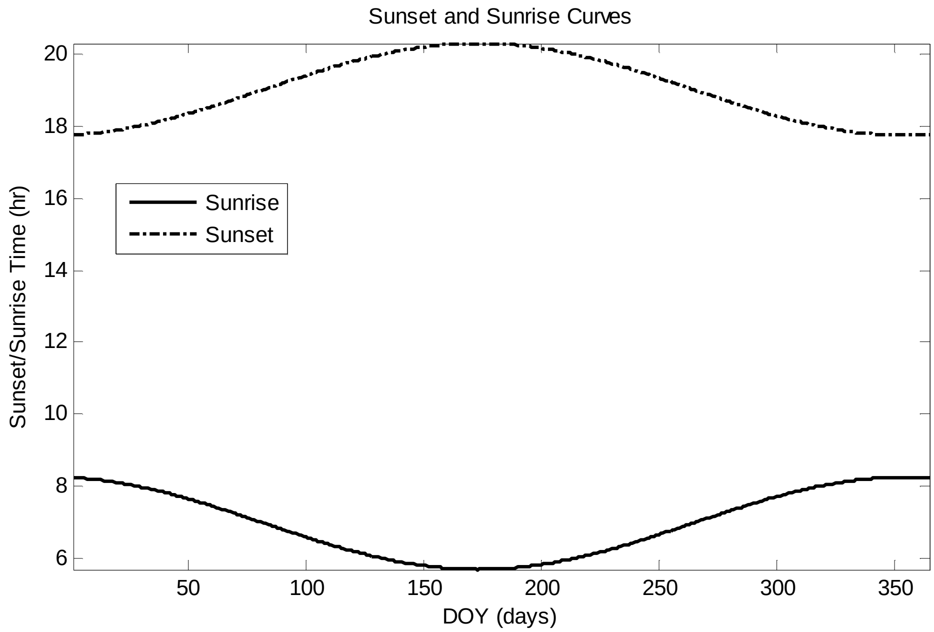

Figure 4-3 Predicted sunrise h_{sr} (solid line) and sunset h_{ss} (dashed line) annual cycles at Latitude 38°.

The following equation was developed to estimate hourly incident solar radiation (*SRI*) values:

$$\hat{K}_{in}(h,d) = \begin{cases} 0 & \text{for } h < h_{sr}(d) \\ 24 \cdot K^*_{in}(d) \cdot \sin\left[\dfrac{h - h_{sr}(d)}{h_{ss}(d) - h_{sr}(d)}\pi\right] & \text{for } h_{sr}(j) \le h \le h_{ss}(d) \\ 0 & \text{for } h > h_{ss}(d) \end{cases} \qquad (4\text{-}8)$$

where $\hat{K}_{in}(h,d)$ is the predicted solar incident radiation (MJ/m^2-d) on hour *h* of day *d*. The final $\hat{K}_{in}$ plot for a given year is shown in Figure 4-4. $\hat{K}_{in}$ represents the total mean incident solar radiation reaching the wetland over a given hour.

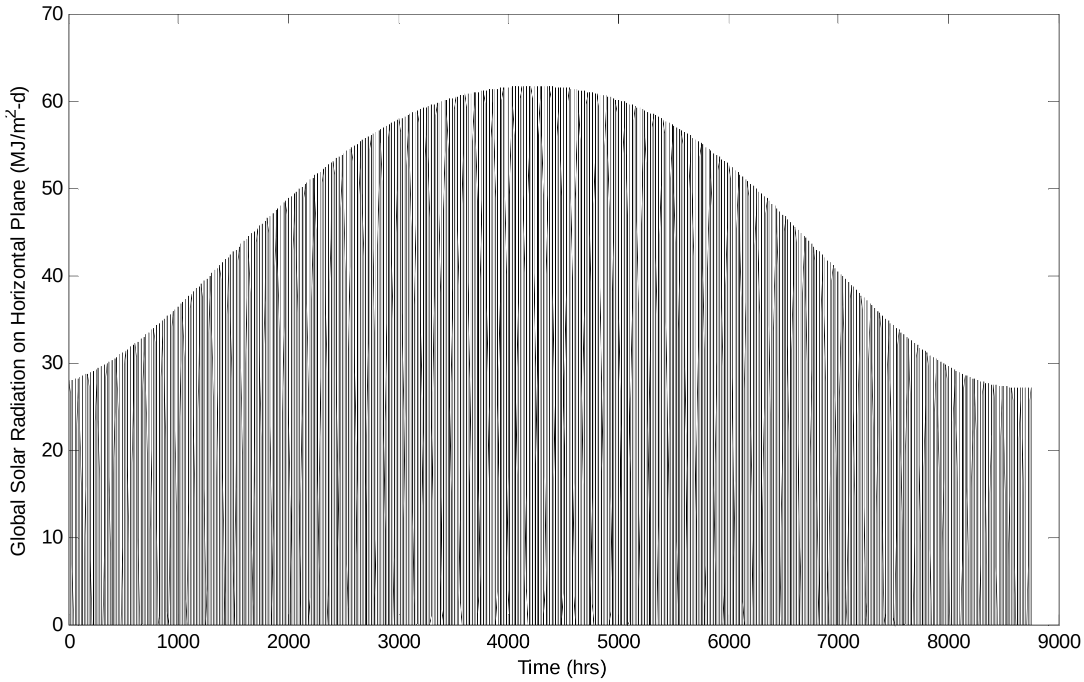

Figure 4-4 Plot of hourly $\hat{K}_{in}$ values generated from the input $\hat{K}_{in}$ curve to the model over the course of one year.

4.2.4 NOAA Climatic Data Inputs

The NOAA-derived inputs used in the ET model are the daily mean air temperature, daily maximum air temperature, daily minimum air temperature, daily dew point temperature, and daily wind speed. The data used in this calibration are NOAA, daily mean data from 1945 through 2011 from Baltimore, MD. Daily mean values are based on the mean of 24 hourly measurements taken each day. Therefore, each mean daily data point represents the mean of 24 hourly values. In order to compile these data into hourly values, sinusoidal curves were used to estimate both annual and daily cycles.

Annual curves were initially estimated for the following four different data inputs: (1) daily mean air temperature T_a (2) daily air temperature range R_a, (3) dew point temperature T_d, and (4) wind speed v_a. The daily temperature range was determined by subtracting the minimum from the maximum temperature for each day of record (21,549 days).

Mean values for all four input parameters were calculated for each day of the year (DOY). DOY refers to the day within a year, with January 1 as 1 and December 31 as 365 in a non-leap-year day. In this project, leap years will be ignored in order to simplify the code. The curve-fitting tool in MATLAB was then used to fit corresponding annual sinusoidal curves. The resulting DOY means represented the mean input value given all 59 years of record of a given DOY (i.e., DOY values for each day were based on 59 data points). The following curves resulted:

$$\hat{T}_a(d) = 13.11 - 12.14\sin\left(0.01713d - 74.15\right) - 0.3474\sin(2 \cdot 0.01713d + 36.5) \quad (4\text{-}9)$$

$$\hat{R}_a(d) = 11.63 - 1.026\sin\left(0.01612d - 79.82\right) - 0.9588\sin(2 \cdot 0.01612d - 4.964) \quad (4\text{-}10)$$

$$\hat{T}_d(d) = 6.5836 - 12.33\sin\left(0.01721d + 1.1487\right) \quad (4\text{-}11)$$

$$\hat{v}_a(d) = 3.869 - 0.6125\sin\left(0.01752d - 81.14\right) - 0.1498\sin(2 \cdot 0.01752d - 1.676) \quad (4\text{-}12)$$

where $\hat{T}_a(d)$, $\hat{R}_a(d)$, $\hat{T}_d(d)$, $\hat{v}_a(d)$ represent the predicted daily air temperature, air temperature range, dew point temperature, and wind speed values for any given DOY *d*. All curves and corresponding mean DOY values are shown in Figure 4-5.

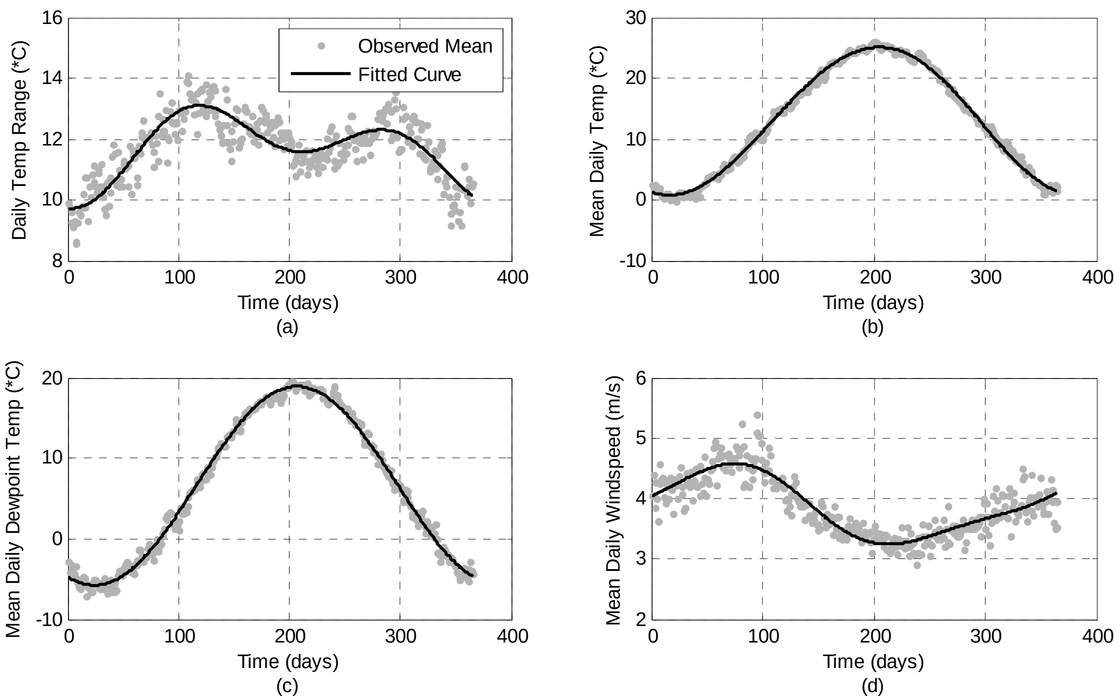

Figure 4-5 Averaged DOY input values (gray dots) and corresponding fitted curves (black) of $\hat{R}_a(d)$ (a)**,** $\hat{T}_a(d)$ (b)**,** $\hat{T}_d(d)$ (c)**,** $\hat{v}_a(d)$ (d).

Once daily values were generated, hourly values for all four input parameters were calculated, which were then input into the PET module. Hourly air temperature and wind speed values were fit to sinusoidal curves over the course of each day, while $\hat{T}_d$ values were kept constant. The following curves were used to define hourly $\hat{T}_a(d)$ and $\hat{v}_a(d)$:

$$\hat{T}_a(h,d) = \hat{T}_a(d) - \frac{\hat{R}_a(d)}{2}\cos\left(\frac{\pi}{12}\left(h - h_{sr}(d)\right)\right) \tag{4-13}$$

$$\hat{v}_a(h,d) = \hat{v}_a(d) - \frac{3}{4}\hat{v}_a(d)\cdot\cos\left(\frac{\pi}{12}\left(h - h_{sr}(d)\right)\right) \tag{4-14}$$

where h represents the hour of the day (1-24hr), and $\hat{T}_a(h,d)$ and $\hat{v}_a(h,d)$ represent the respective air temperature (°C) and wind speed (m/s) values on hour h of DOY d.

A phase shift of $\frac{\pi}{12}$ was also included in both curves to ensure minimum temperature and wind speed values occurred at the estimated sunrise time of each day. If $\hat{T}_d$ values for a given hour were greater than the corresponding $\hat{T}_a(h,d)$, $\hat{T}_d$ was reset to equal $\hat{T}_a(h,d)$. These daily trends were estimated from figures viewed on weatherspark.com for Baltimore, MD via data from the Baltimore-Washington Thurgood Marshall Airport. Unfortunately, this hourly data was not made available by the website. Figure 4-6 shows example results of air temperature hourly distributions for the first days of February, May, July, and November. Both the daily mean air temperature and range vary. The analogous plot for wind speed is shown in Figure 4-7.

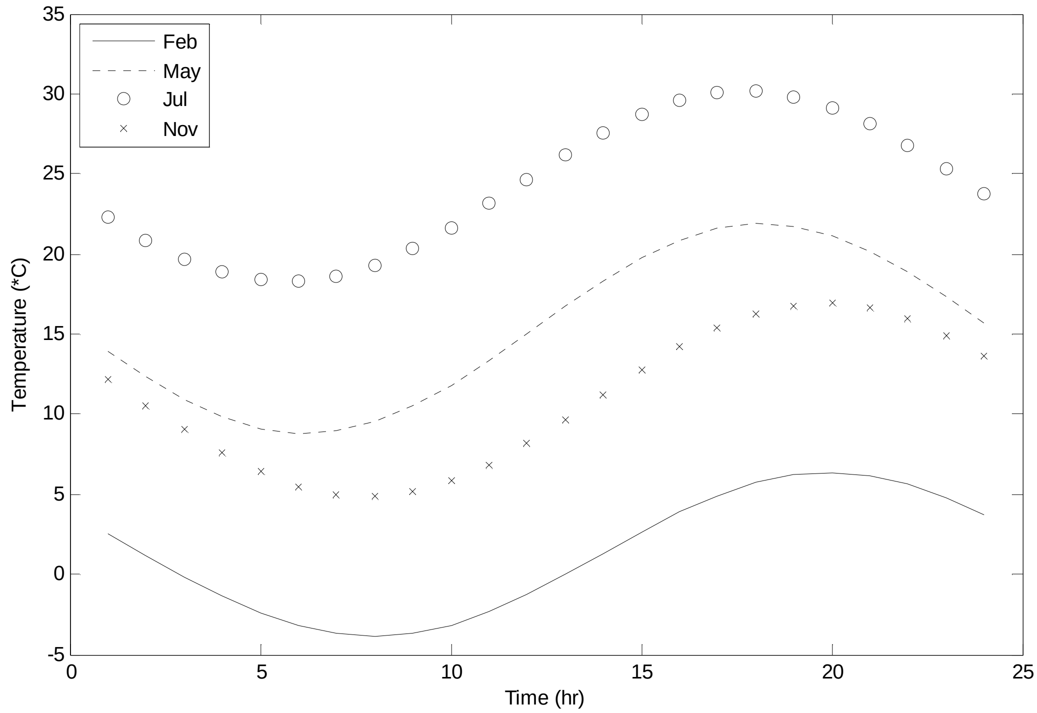

Figure 4-6 Resulting hourly distribution of air temperature over the course of one day. Each line represents generated temperature value for the first days of February, May, July, and November.

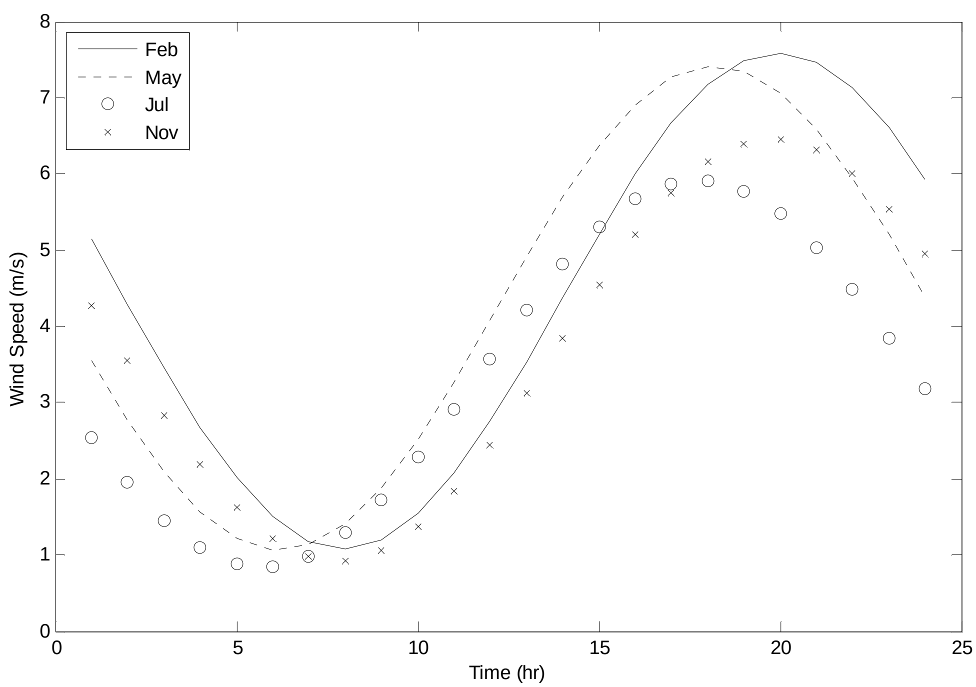

Figure 4-7 Resulting hourly distribution of wind speed over the course of one day. Each line represents generated temperature value for the first days of February, May, July, and November.

4.2.5 The Penman-Monteith ET Method

The general form of the Penman-Monteith equation is:

$$PET = \frac{\Delta \cdot (K_{in} + R_L) + \rho_a \cdot c_a \cdot C_{at} \cdot e_a^* \cdot (1 - W_a)}{\rho_w \lambda_v \cdot [\Delta + \gamma \cdot (1 + C_{at} / C_{can})]} \tag{4-15}$$

where *PET* is potential evapotranspiration (mm/d), K_{in} is net incoming shortwave (solar) radiation (MJ/m-d), R_L is net longwave radiation (MJ/m-d), γ psychrometric constant (kPa/K), λ_v is the latent heat of vaporization of water (MJ/kg), e_a^* is the saturation vapor pressure at the air temperature (kPa), W_a is the relative humidity of the air expressed as a ratio (dimensionless), ρ_w is the mass density of water (kg/m^3), ρ_a is air density (kg/m^3), c_a is the heat capacity of the air (MJ/kg-K), C_{at} is atmospheric conductance (m/d), and C_{can} is canopy conductance (m/d), and Δ represents the slope of the saturation-vapor vs. temperature curve at the air temperature (kg/m-d-K).

4.2.5.1 Mass Transfer Equations

Water and air densities, respectively ρ_w and ρ_a are temperature dependent, with values determined using the calculated hourly air temperature $\hat{T}_a(h,d)$. Water density was approximated using the following empirical thermal-dependent equation (Dingman 2002):

$$\rho_w = 1000 - 0.019549 \cdot \left| \hat{T}_a(h,d) - 3.98 \right|^{1.68} \tag{4-16}$$

where ρ_w has units of kg/m^3. Air density was calculated using a derivation of the Ideal Gas Law (Dingman 2002):

$$\rho_a = \frac{P}{[\hat{T}_a(h,d) + 273.2] \cdot R} \tag{4-17}$$

where P (kPa) is the atmospheric pressure, which was assumed to be 101.3 kPa; R is the gas constant of air, which was set to 0.288; and ρ_a is in units of kg/m^3. The latent heat of vaporization was then calculated (Dingman 2002):

$$\lambda_v = 2.50 - 2.36 \times 10^{-3} \cdot \hat{T}_a(h,d) \tag{4-18}$$

Next, the saturated (e_a^*) and actual (e_a) vapor pressures were calculated given the calculated hourly air $\hat{T}_a(h,d)$ dew point $\hat{T}_d(h,d)$ temperatures (Dingman 2002):

$$e_a^* = 0.611 \cdot \exp\left(\frac{17.3 \cdot \hat{T}_a(h,d)}{\hat{T}_a(h,d) + 237.3}\right) \tag{4-19}$$

$$e_a = 0.611 \cdot \exp\left(\frac{17.3 \cdot \hat{T}_d(h,d)}{\hat{T}_d(h,d) + 237.3}\right) \tag{4-20}$$

Both vapor pressures e_a^* and e_a have units of kPa. With both e_a^* and e_a known, the relative humidity W_a was calculated (Dingman 2002):

$$W_a = \frac{e_a}{e_a^*} \tag{4-21}$$

In order to determine Δ, which represents the slope of the vapor pressure-temperature curve at $\hat{T}_a(h,d)$ and e_a^*, the derivative of Equation 1-34 was taken (Dingman 2002):

$$\Delta = \frac{de_a^*}{d\hat{T}_a(h,d)} = \frac{2508.3}{(\hat{T}_a(h,d) + 237.3)^2} \cdot \exp\left(\frac{17.3 \cdot \hat{T}_a(h,d)}{\hat{T}_a(h,d) + 237.3}\right) \tag{4-22}$$

where Δ is in units of kPa/K. This relationship shows that Δ increases exponentially with increasing atmospheric temperature, which is also seen in the vapor pressure-temperature curve in Figure 4-8.

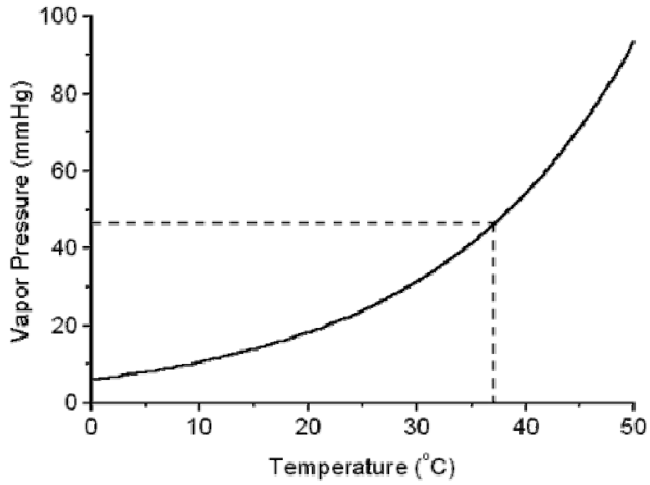

Figure 4-8 Exponential relationship between air temperature and saturated vapor pressure.

The phychrometric constant γ relates the partial pressure of water in air to air temperature. It can be calculated accordingly (Dingman 2002):

$$\gamma \equiv \frac{c_a P}{0.622\lambda_v} \qquad (4\text{-}23)$$

where c_a (MJ/kg-K) is the heat capacity of air and was assumed to be $1\text{x}10^{-3}$ MJ/kg-K, and γ has units of kPa/K.

4.2.5.2 Radiation-Based Equations

Once all mass transfer inputs were calculated, radiation-based inputs were determined. Longwave radiation R_L was determined based partially on the emissivity of the air ε_a (dimensionless ratio), which, in turn, is based on air temperature, vapor pressure, and cloud cover (Dingman 2002):

$$\varepsilon_a = 1.72 \cdot \left(\frac{e_a}{\hat{T}_a(h,d) + 273.2} \right)^{1/7} \cdot \left(1 + 0.22 C_C{}^2\right) \tag{4-24}$$

$$R_L = \varepsilon_w \sigma (\varepsilon_{at} - 1) \cdot \left[\hat{T}_a(h,d) + 273.2\right]^4 \tag{4-25}$$

where ε_w is the emissivity of water and was estimated to be 0.97 for liquid water, σ is the Stefan-Boltzmann constant (σ = 4.90 x 10^{-9} MJ/m^2-d-K^{-4}), C_C is the fraction of sky covered by clouds, and R_L has units of MJ/m^2-d. With both $\hat{K}_{in}$ and R_L known, the net incoming radiation was defined accordingly (Dingman 2002):

$$R_N = \hat{K}_{in}(1 - a) + R_L \tag{4-26}$$

where a is the albedo of wetland surface (vegetation and water surface combined), and R_N is the resulting net incoming radiation to the wetland surface with units of MJ/m^2-d.

4.2.5.3 Vegetation-Related Equations

Transpiration is controlled by the leaf conductance C^*_{leaf} of a given vegetation type as well by atmospheric conductance C_{at}. Leaf conductance, in turn, depends on four main controlling factors related to stomatal opening were considered in this method: light, vapor-pressure deficit, leaf temperature, and leaf water content. CO_2 and O_2 levels were assumed not to play large roles in transpiration rates.

The light factor $f_K(K_{in})$ was based on incoming solar radiation K_{in} values (Stewart 1988):

$$f_K(K_{in}) = \begin{cases} \dfrac{12.78 \cdot K_{in}}{11.57 \cdot K_{in} + 104.4} & \text{for } 0 \le K_{in} \le 86.5\text{MJ/m}^2\text{d} \\ 1 & \text{for } K_{in} > 86.5\text{MJ/m}^2\text{d} \\ 0 & \text{for } K_{in} < 0 \end{cases} \qquad (4\text{-}27)$$

where $f_K(K_{in})$ is dimensionless. Figure 4-9a shows that relative leaf conductance increases with increasing K_{in}, reaching an asymptote at a K_{in} of about 1000 W/m^2 or 86.4 MJ/m^2-d (Stewart 1988). Therefore, $f_K(K_{in})$ values were set to 1 if $K_{in} > 86.5$ MJ/m^2-d. Negative K_{in} should not occur.

Next, the vapor-pressure factor $f_\rho(\Delta\rho_v)$ was determined based on the absolute humidity deficit $\Delta\rho_v$ (Stewart 1988):

$$f_\rho(\Delta\rho_v) = \begin{cases} 1 - 66.6 \cdot \Delta\rho_v & \text{for } 0 \le \Delta\rho_v \le 0.01152\,\text{kg/m}^3 \\ 0.233 & \text{for } \Delta\rho_v \ge 0.01152\,\text{kg/m}^3 \end{cases} \qquad (4\text{-}28)$$

where $f_\rho(\Delta\rho_v)$ is dimensionless. If $\Delta\rho_v$ is sufficiently large, creating a large ET driving force, $f_\rho(\Delta\rho_v)$ steadies to a value of 0.233. A plot of the dependence of relative leaf conductance on $\Delta\rho_v$ is shown in Figure 4-9c. Conductance decreases linearly with increasing $\Delta\rho_v$ until a humidity deficit of about 10g/kg or about 0.012 kg/m^3, after which conductance remains constant with increasing $\Delta\rho_v$ (Stewart 1988).

A leaf temperature factor $f_T(T_a)$ was also calculated (Stewart 1988):

$$f_T(T_a) = \begin{cases} \dfrac{T_a(h,d) \cdot [40 - T_a(h,d)]^{1.18}}{691} & \text{for } 0 \le T_a(h,d) \le 40^\circ\text{C} \\ 0 & \text{otherwise} \end{cases} \qquad (4\text{-}29)$$

Air temperature has a negative quadratic relationship with relative leaf conductance as shown in Figure 4-9b. Conductance increases from temperatures from 0 to 18°C, decreasing for temperatures greater than 18°C (Stewart 1988).

Finally, a leaf water content factor $f_\theta(\Delta\theta)$ was defined by Stewart (1988):

$$f_\theta(\Delta\theta) = 1 - 0.00119 \cdot \exp(0.81\Delta\theta) \quad \text{for } 0 \le \Delta\theta \le 8.4\,\text{cm} \qquad (4\text{-}30)$$

where $\Delta\theta$ is the soil-moisture deficit. Figure 4-9d shows the dependence of relative leaf conductance on $\Delta\theta$. Conductance remains at a maximum from $\Delta\theta$ values of 0 to about 45 mm, after which conductance decreases steeply, reaching zero at about 80 mm. Within the current study, the soil-moisture deficit was set to zero, producing a constant $f_\theta(\Delta\theta)$ of 1.0.

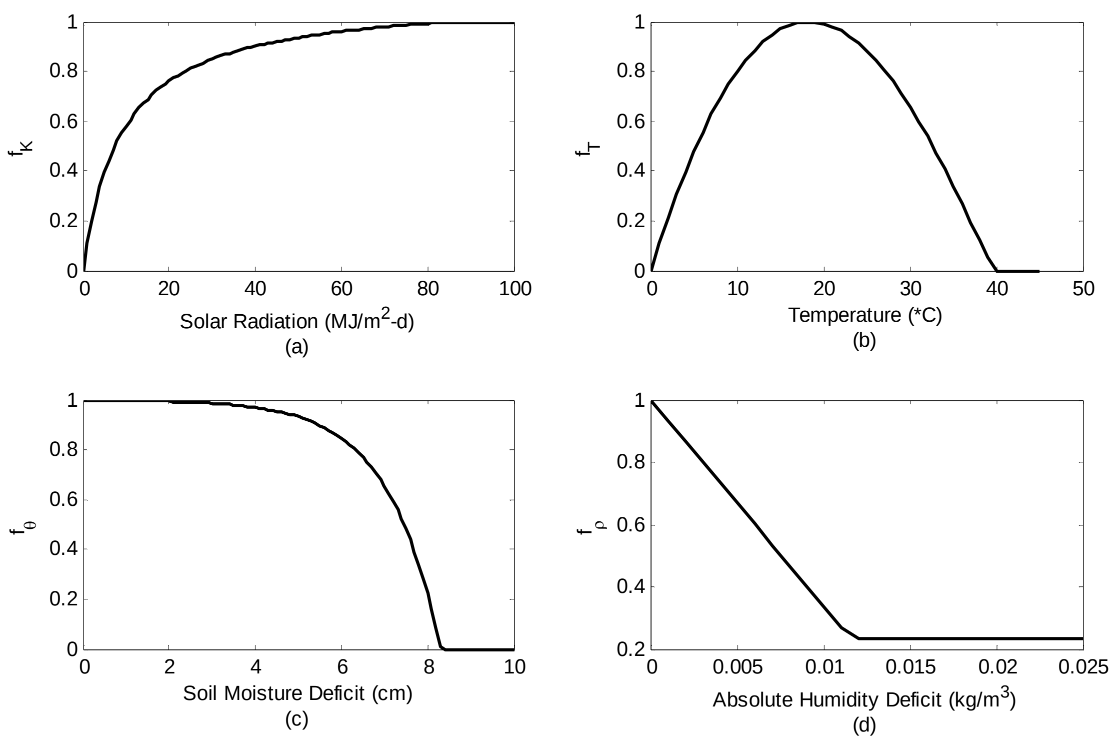

Figure 4-9 Dependence of relative surface conductance on solar radiation K_{in} (a), temperature T_a (b), specific humidity deficit $\Delta\rho_v$ (c), and soil moisture deficit $\Delta\theta$ (d). After Stewart (1988) and Dingman (2002).

With all vegetation factors defined, an overall leaf conductance C_{leaf} value could be determined (Dingman 2002):

$$C_{leaf} = C^{*}_{leaf} \cdot f_k(K_{in}) \cdot f_\rho(\Delta\rho_v) \cdot f_T(T_a) \cdot f_\theta(\Delta\theta) \qquad (4\text{-}31)$$

where C^{*}_{leaf} is the maximum value of leaf conductance. All vegetation factors have values between 0 and 1. Therefore, if all factors are at a maximum 1 value, the final C^{*}_{leaf} will be equal to C^{*}_{leaf}. A C^{*}_{leaf} value of 6.6 mm/s was used to represent tundra/nonforest wetland vegetation (Dingman 2002). Finally, a final canopy conductance C_{can} was calculated to represent the total vegetation conductance from a wetland area (Dingman 2002):

$$C_{can} = f_s \cdot LAI \cdot C_{leaf} \qquad (4\text{-}32)$$

where f_s is a shelter factor that accounts for some leaves sheltering others from the sun and LAI is the leaf-area index. Atmospheric conductance C_{at} was also calculated (Dingman 2002):

$$C_{at} = \frac{v_a}{6.25 \cdot \left[\ln\left(\frac{z_m - 0.7z_v}{0.1z_v}\right)\right]^2} \qquad (4\text{-}33)$$

where v_a is the wind speed (m/s) and z_v is the vegetation height (m). Given that only vegetation above the water surface would be available for transpiration, this vegetation height was assumed to equal the height of the emergent vegetation minus the mean water depth of the wetland. The current study also found it necessary to restrict z_v values to be greater than zero and less than or equal to z_m in order to insure rational C_{at} values.

4.2.6 PET Component Results

Monthly and annual PET output values from the PET component were compared with values reported in the literature. MDE (2009) reported monthly evaporation depths for Maryland stormwater ponds. These evaporation depths were used by MDE (2009) to assess the feasibility of a wetland based on its water balance and corresponding water depths during extended dry periods. Additionally, monthly Class A Pan evaporation data for Beltsville, MD was recorded by NOAA in Farnsworth and Thompson (1982) over the 38-year period from 1941 to 1979. These monthly evaporation depths corresponded to a pan annual evaporation depth of 41.4 in. with an associated error of ±11.2%. Dunne and Leopold (1978) also showed a pan evaporation map of the US, from which the Eastern Shore of MD was estimated to have a pan evaporation of 47.5 in.

Because pan evaporation does not necessarily represent wetland ET, the current study used correction factors to estimate wetland ET depths from reported class A pan evaporation depths. Kadlec and Knight (1996) suggested that wetland total ET can be estimated by multiplying Class A pan evaporation by a correction factor within the range of 0.70 and 0.80. These correction factors produced an estimated annual wetland ET range of 25.7 (pan correction factor of 0.70) to 36.8 in. (pan correction factor of 0.80) based on estimated NOAA annual pan evaporation depth of 41.4 in. ±11.2%. Wetland ET depths based on the Dunne and Leopold

(1978) pan evaporation of 47.5 in. ranged from 33.3 (pan correction factor of 0.70) to 38 in. (pan correction factor of 0.80) with a mean depth of 35.6 in. (pan correction factor of 0.75). Therefore, a literature wetland ET range of 25.7 to 38 in. was assumed for the Maryland region. Additionally, the NOAA Class A pan monthly evaporation depths were multiplied by 0.70 and 0.80 in order to estimate a range of wetland ET depths for the Beltsville, MD region (see Table 4-1).

In order to compare the resulting monthly PET values output by the PET component, the average monthly PET depths (in.) were calculated over a simulation period of 100 years. It was also assumed that because wetlands generally have sufficient water to fulfill PET, that estimated PET depths were comparable to actual wetland ET depths. The rainfall generator developed by Gupta (2013) and discussed in detail in Section 4.4.1.1 was used to generate hourly rainfall within this test simulation. PET was set equal to zero when rainfall occurred within a given hour of simulation. Additionally, mean literature values for all constant inputs (see Table 4-2) were used in this PET component test. The resulting values as well as the corresponding PET depths from the literature are compiled in Table 4-1.

The PET component monthly values match the adjusted pan evaporation data from NOAA (Farnsworth and Thompson 1982) for the months of April through November fairly well. However, while MDE (2009) and Farnsworth and Thompson (1982) data measurements are not reported for the winter months (and are assumed to be zero), the PET Model does produce a small amount of PET during March as leaves were assumed to regain activity on March 21st (the spring equinox and assumed start of spring). The PET module also assumes that transpiration does not occur after the fall equinox (September 21st), which agrees with PET values reported by MDE (2009), but disagrees with the pan data collected by Farnsworth and Thompson (1982) data, as fall senescence did not affect pan evaporation. Overall, the simulated monthly PET depth agree with both the MDE (2009) and corrected Farnsworth and Thompson (1982) depths. Resulting root mean square error (RMSE), relative standard error (S_e / S_y), bias ($\bar{e}$), and relative bias ($\bar{e} / \bar{y}$) existing between the monthly model and MDE PET depths were 0.538 in., 0.197, 0.0964 in., and 0.0361. Similarly, the resulting statistics calculated from the monthly model and corrected NOAA pan evaporation PET depths with a correction factor of 0.80 were an RMSE of 1.18 in., an S_e / S_y of 0.539, a $\bar{e}$ of 0.007 in., and a $\bar{e} / \bar{y}$ of 0.0025. Finally, the resulting RMSE S_e / S_y, $\bar{e}$, and $\bar{e} / \bar{y}$ compared to the NOAA pan evaporation with a correction factor of 0.70 were respectively 1.31 in., 0.682, 0.352 in., and 0.146 in.

Table 4-1 Monthly Maryland stormwater pond evaporation depth (MDE 2009), NOAA Class A pan evaporation (Farnsworth and Thompson 1982), NOAA Class A pan evaporation adjusted with a wetland correction factor of 0.75 (Farnsworth and Thompson 1982), and the PET module output depths.

	Mar.	**Apr.**	**May**	**Jun.**	**Jul.**	**Aug.**	**Sep.**	**Oct**	**Nov**	**SUM**
MDE (2009) (in)	**---**	4.32	5.28	6.24	6.48	5.52	4.2	---	---	**32.0**
NOAA (1982) Class A (in.)	**---**	5.13	5.66	6.57	7.31	6.19	4.75	3.34	2.44	**41.4**
NOAA (1982) 0.8*Class A Pan (in)	---	4.104	4.528	5.256	5.848	4.952	3.800	2.672	1.952	**33.1**
NOAA (1982) 0.7*Class A Pan (in)	---	3.591	3.962	4.599	5.117	4.333	3.325	2.338	1.708	**28.9**
PET Model (in)	1.29	4.88	5.97	6.15	6.09	5.55	3.27	0	0	**33.2**

Table 4-2 Literature mean and ranges of input parameters used in the PET component of the model. Mean values were used to generate the initial PET estimates within the model.

Model parameter	**Units**	**Mean**	**Estimated range**	**Sources**
Wetland albedo *a*	dimensionless	0.159	0.05-0.333	Rouse and Bello (1983), LaFleur et al. (1987), Federer et al. (1996), Goodin et al. (1996), Dingman (2002)
Leaf area index LAI	dimensionless	6.5	2.5-23	Boyd (1987), Koch and Rawlik (1993), Federer et al. (1996), Xu et al. (2011),
Shelter factor f_s	dimensionless	0.75	0.5-1	Federer et al. (1996), Dingman (2002)
Maximum leaf conductance C_{leaf}^{*}	mm/s	9.7	3-21	Federer et al. (1996), Koch and Rawlik (1993), Morrissey et al. (1993)
Mean vegetation height above water z_v	m	1.65	0.3-3	Federer et al. (1996), Kadlec and Knight (1996)

4.3 WATER TEMPTERATURE

Hourly water temperature values were determined based on incident solar radiation (K_{in}), calculated daily mean air temperature ($\hat{T}_a$), and calculated daily mean wind speed ($\hat{v}_a$) data. In addition, model-derived values of actual evapotranspiration (*AET*) and longwave radiation (R_L) were also used to compute hourly water temperatures.

The current study used an adapted version of the theory-based water temperature method used in HSPF, which treats water temperature as a thermal concentration. The variables Q_{SR}, Q_B, Q_H, and Q_E were used to represent the respective energy fluxes of solar radiation (K_{in}), longwave radiation (R_L), conductive-convective forces (a function of the difference between air ($\hat{T}_a$) and water temperature ($\hat{T}_w$) as well as wind speed $\hat{v}_a$), and actual evapotranspiration (*AET*). HSPF additionally allowed the user to define heat fluxes for precipitation and the wetland ground. Both precipitation and ground contributions to the wetland water temperature flux were assumed negligible in the current study. Q_{SR} was defined accordingly (Bicknell et al. 2001):

$$Q_{SR}(h,d) = 9.96 \cdot K_{in}(h,d) \cdot (1-a) \qquad (4\text{-}34)$$

where a represents the albedo of the wetland, which was set to 0.10 according to model calibration and suggested values given in Dingman (2002); K_{in} is the incident solar radiation reaching the wetland (MJ/m^2-d) on hour *h* of DOY *d*, and Q_{SR} is the heat input of solar radiation to the water (kcal/m^2-hr.) on hour *h* of DOY *d*. The value 9.96 represents the conversion factor from MJ/m^2-d to kcal/m^2-hr. Next, the longwave radiation heat flux was calculated (Bicknell et al. 2001):

$$Q_B(h,d) = 9.96 \cdot R_L(h,d) \qquad (4\text{-}35)$$

where R_L is the longwave radiation in MJ/m^2-d and Q_B is the associated heat flux (kcal/m^2-hr.) on hour *h* of DOY *d*. Q_B represents the heat loss from the water. Similarly, the latent heat flux associated with evapotranspiration (Q_E) was defined as a heat loss (Bicknell et al. 2001, Dingman 2002):

$$Q_E(h,d) = -AET(h,d) \cdot \rho_w \cdot \lambda_{vap}(h,d) \cdot 239 \qquad (4\text{-}36)$$

where AET represents the actual evapotranspiration rate (m/hr.), ρ_w is the density of water as calculated in the evaporation module (kg/m^3), λ_{vap} is the latent heat of vaporization (MJ/kg), and Q_E is the resulting evaporative heat loss (kcal/m^2-hr.) on hour *h* of DOY *d*. Actual evapotranspiration AET was restricted to the water depth available in the wetland for a given time interval, which distinguished it from PET. The term 239 represents the conversion factor from MJ to kcal. Finally, the conductive-convection heat flux Q_H was calculated (Dingman 2002):

$$Q_H(h,d) = -C_{HT} \cdot \hat{v}_a(h,d) \cdot [(T_w(h-1,d) + 273.15) \\ - (\hat{T}_a(h,d) - 273.15)] \cdot 3600 \cdot 239 \tag{4-37}$$

where C_{HT} is the heat transfer coefficient (MJ/K-m^3), $\hat{v}_a(h,d)$ is the calculated hourly mean wind speed (m/s) for day *d*, T_w is the water temperature (°C) from the previous hour (*h-1*), $\hat{T}_a(h,d)$ is the calculated temperature (°C) for hour *h* of DOY *d*, and Q_H is the conductive-convection heat flux (kcal/m^2-hr.) on hour *h* of DOY *d*. C_{HT} was additionally calculated (Dingman 2002):

$$C_{HT} = c_a \cdot \rho_a \cdot \frac{1}{6.25 \cdot \left[\ln\left(\frac{z_m - 0.7z_v}{0.1z_v}\right)\right]^2} \tag{4-38}$$

where c_a (MJ/kg-K) is the heat capacity of air and was assumed to be 1x10^{-3} MJ/kg-K, ρ_a is the air density in units of kg/m^3, z_m is the elevation at which wind measurements are taken (m), z_v is vegetation height (m), and C_{HT} is the heat transfer coefficient (MJ/K-m^3). The current study also found it necessary to restrict z_v values to be greater than zero and less than or equal to z_m in order to insure rational C_{HT} values. Once all contributing heat fluxes were calculated, they were summed into one term Q_T (Bicknell et al. 2001):

$$Q_T = Q_{SR} + Q_B + Q_H + Q_E \tag{4-39}$$

where Q_T represents the total heat flux to the water (kcal/m^2-hr.). A positive Q_T implies heat is being transferred to the water, while a negative value implies heat is leaving the water. In order to convert the heat flux to a temperature change ΔT_w, the following equation was used (Bicknell et al. 2001):

$$\Delta T_w = \frac{3.28 \cdot Q_T}{\overline{SS} \cdot \rho_w} \quad (4\text{-}40)$$

where $\overline{SS}$ is the mean surface storage depth (ft) of the wetland at time *h*, and ΔT_w represents the resulting water temperature change for a time interval (°C). A $\overline{SS}$ value of 2.15 ft for initial water temperature calibration, which was estimated from literature wetland depth specifications (Kadlec and Knight 1996; USEPA 2000; NRCS 2002; MDE 2009). The final water temperature for a given hour of record *h* was calculated accordingly:

$$T_w(h) = T_w(h-1) + 0.385\Delta T_w \quad (4\text{-}41)$$

where 0.385 represents a calibration coefficient initially designed as a Taylor series first-order term that is dependent of the relative change of Q_H, Q_B, and Q_E with respect to temperature at the beginning and end of a time interval. A corresponding calibration coefficient of 0.385 was found sufficient to produce temperatures with a daily range of 5°C as specified by Kadlec and Reddy (2001) for free water surface wetlands and mean daily water temperatures within the range of USGS mean daily water temperature data for the Paint Branch Stream in College Park, MD (http://waterdata.usgs.gov/nwis/nwisman?site_no=01649190) while avoiding the additional computational expense of the Taylor expansion.

HSPF also specified that this water temperature method does not work for water levels under 2 in. (0.167 ft) (Bicknell et al. 2000). Therefore, if wetland surface water levels reached 0.167 ft or less, the water temperature was set equal to the air temperature. Figure 4-10 compares the resulting air and water temperature hourly results over the course of one year. Water temperatures vary much less than air temperature over the course of one day. This behavior reflects the higher heat capacity of water versus air.

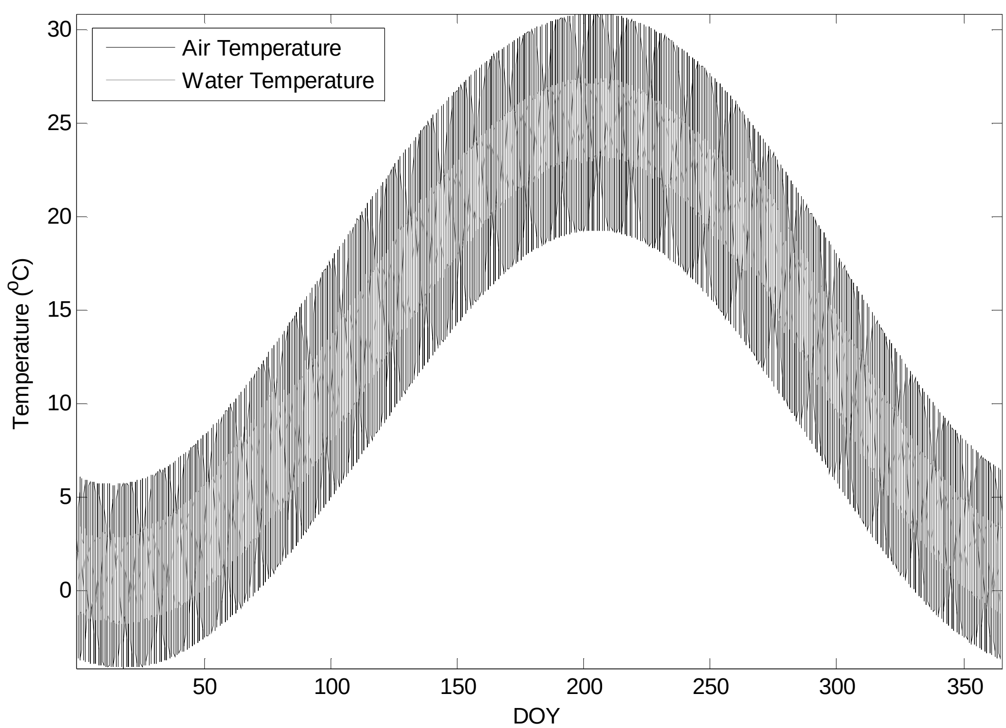

Figure 4-10 Comparison of model-generated hourly air temperature (black) and water temperatures (grey).

4.3.1 Water temperature component output

Water temperatures obtained from the water temperature component were compared with stream data. Figure 4-11 shows the comparison of the mean daily water temperatures output by the Water temperature component and values from 2008 from the USGS gaged site on Paint Branch stream in College Park, MD (http://waterdata.usgs.gov/nwis/nwisman?site_no=01649190). The water temperature component produced a similar trend and similar results as the USGS data curve.

In general, wetland water temperatures should be expected to be higher than those at the Paint Branch due to the higher velocities associated with streams. Wetland water temperature data was not found to perform a more useful analysis. Therefore, this water temperature comparison was used as a guide rather than a fitting method. Despite this drawback, the resulting water temperature values were rational and followed the expected trend.

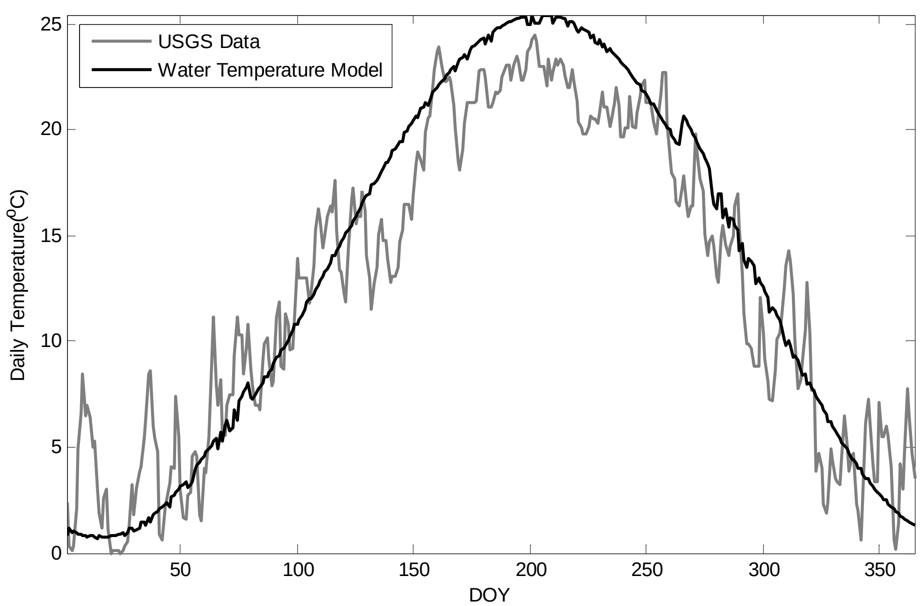

Figure 4-11 Daily mean water temperature predicted by the Water Temperature Model (solid black line). The grey line represents USGS downloaded mean daily water temperature data for the Paint Branch stream (gray) near College Park, MD.

4.4 WETLAND-CELL MODEL

4.4.1 Hydrologic Simulation

The current section outlines the overall methods used to route water into and through the wetland model. Figure 4-12 shows all influent and effluent fluxes calculated for each wetland cell, which include precipitation, evapotranspiration (ET), infiltration, inflow, and outflow.

Before simulations were made, a wetland design was established, which included defining all cell bottom elevations and slopes, water depths, infiltration rates, exiting berm heights, and any vegetative properties. A cell flowpath was then determined from the characterized wetland grid. This flowpath, which is an input into the wetland simulation model, is used to direct water flow through the cells. Within this user-defined flowpath any cell can receive flow from multiple up-gradient cells (e.g., cells 2 and 3 both flow into the outlet cell 1). Each cell, however, can only flow into two downgradient, adjacent cells. To allow for this flowpath structure, the user is required to input a primary and a secondary flowpath. The primary flowpath of the wetland should define the main flowpath within a given wetland design. The secondary flowpath serves two main purposes, which are (1) to supplement the primary flowpath and (2) to simulate the smoothing of water levels in the wetland after water is routed through the primary flow path for a given time interval. Within each of the primary and secondary flowpaths, any given cell is only allowed to flow into one downgradient, adjacent cell. Therefore, by inputting both primary and secondary flowpaths, each cell was allowed to flow into up to two adjacent cells.

This flowpath structure allowed the user route water through a given wetland design with sufficient detail and complexity.

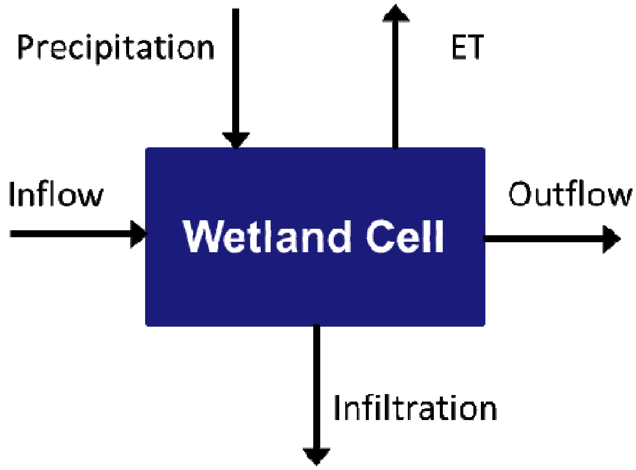

Figure 4-12 Schematic of all hydrologic components calculated within a given wetland cell.

Wetland cells are numbered according to their order in the defined primary flowpath. The outlet cell is always designated as cell 1. Flow Identification Direction (FID) values are assigned to each cell. These FID values identify the cell number into which flow is discharged from a given cell within the primary flowpath. The secondary flowpath vector, referred to as FID2, is also defined in terms of these primary flowpath FID values. If cell 3, for example, flows into cell 2 in the secondary flowpath, it would be assigned an FID value of 2. However, if cell 3 did not flow into any other cells in the secondary flowpath, it would be assigned an FID2 value of 3. These referencing vectors FID and FID2 allow the model to define the path that water follows through the wetland. Figure 4-13 shows an example wetland flowpath divided into cells labeled according to the design primary flowpath. As shown in Figure 4-13 and summarized in Table 4-3, runoff from the watershed or primary treatment storage flows into cell 12 and is routed through the wetland according to the FID and FID2 vectors. Additionally, Figure 4-14 shows the same wetland with different primary and secondary flowpaths (also see

Table 4-4). The outlet cell always has an FID value of 0. Within a given time interval, flow is routed through all cells according to the primary flowpath FID and then routed again according to the secondary flowpath FID2.

Figure 4-13 Example wetland flowpath diagram with numbers in each cell representing its primary flow FID values. Black arrows indicate direction of primary flow while grey arrows indicate the secondary flowpath. The darkly shaded cells highlight the main flowpath through the wetland.

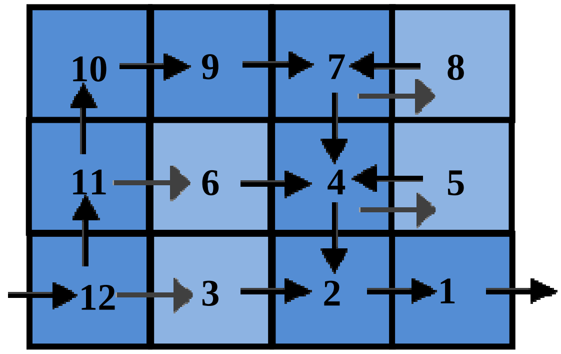

Table 4-3 Constructed wetland cell specifications for the primary (FID) and secondary (FID2) flowpath input vectors for the wetland design shown in Figure 4-13.

Cell	FID	FID2
1	0	1
2	1	2
3	2	3
4	2	5
5	4	5
6	4	6
7	4	8
8	7	8
9	7	9
10	9	10
11	10	6
12	11	3

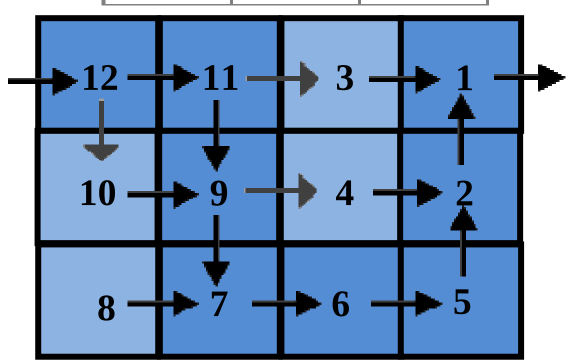

Figure 4-14 Same wetland depicted in **Figure 4-13** with different primary and secondary flowpaths. Black arrows indicate direction of primary flow while grey arrows indicate the secondary flowpath. The darkly shaded cells highlight the main flowpath through the wetland. Cells are numbered according to their primary flowpath FID values.

Table 4-4 Constructed wetland cell specifications for the primary (FID) and secondary (FID2) flowpath input vectors for the wetland design shown in Figure 4-14.

Cell	FID	FID2
1	0	1
2	1	2
3	1	3
4	2	4
5	2	5
6	5	6
7	6	7
8	7	8
9	7	4
10	9	10
11	9	3
12	11	10

The model calculates precipitation, evapotranspiration, and outflow for each cell, beginning with the outlet cell. Once all of the fluxes are calculated for a given cell, the model calculates the same fluxes for the upstream adjacent cell(s) in the defined flowpath. Calculations proceed in the order of the assigned FIDs. All outflow fluxes from an upstream cell are then added to the receiving downstream cell(s) as inflow. For example, flow from cell 12 (see Figure 4-13) discharges into cells 11 and 3.

Within each wetland cell, the following flux calculation order was used: (1) precipitation added; (2) outflow to the downgradient cell; (3) evapotranspiration; and (4) infiltration to the aquifer. Once all five fluxes are calculated for a cell within a given time interval, the model moved to the adjacent, up-gradient cell in the flowpath for which the same five fluxes are calculated. Outflow from this up-gradient cell is then added to the receiving, down-gradient cell. Given these hydrologic components, a water balance for each wetland cell was defined accordingly:

$$SS(t+1) = SS(t) + IN - OUT - AET - I + P \tag{4-42}$$

where $SS(t+1)$ represents the surface storage depth (ft) of a given cell at time *t+1*, $SS(t)$ is the surface storage depth (ft) of a given cell at time *t*, IN is the total inflow water depth into a given cell (ft) at time *t* from up-gradient cell(s) or the forebay, OUT is the total outflow water depth from a cell (ft) at time *t*, AET is the total depth of actual evapotranspiration removed from a cell (ft) at time *t*, I is the depth of water lost from the cell due to infiltration (ft) at time *t*, and P is the total depth of precipitation falling on the cell (ft) at time *t*. A time interval of 1-min was used within the model.

4.4.1.1 Precipitation

Precipitation was generated using a method developed by Gupta (2013), who used storm depth and duration distributions generated by Kreeb (2003) for the Baltimore-Washington area based on 15 years of rainfall record collected from 15 rainfall gages within the state of Maryland. The resulting depth-duration distributions are summarized in Table 4-5.

Before simulating individual storm events, the number of rainy days was simulated for each year. Because the model was calibrated for Baltimore, MD, it was assumed that, on average, rain occurred on 90 days out of the year. Therefore, rainfall was only simulated for an average of 275 days out of the year. The model generated by a binomial distribution B (1, 0.25) for each day of the year, where a probability of a day being rainy was 0.25 (90 days/365 days). Therefore, when a value of 1 was generated for a given day, it was assumed to be rainy. Conversely, a value of 0 implied rain did not occur on that day. If a day was simulated to be rainy, the method developed by Gupta (2013) was used to generate a storm event for that day based on the Depth-Duration table shown in Table 4-5. Storm events were always assumed to occur at the beginning of the day.

Table 4-5 Depth-Duration table showing the fraction of actual storms for each interval of depth and duration in Baltimore, MD (Kreeb and McCuen 2003).

Event Duration	**Rainfall Depth (in.)**					**Sum**
	0.01-0.1	***0.1-0.25***	***0.25-0.5***	***0.5-1***	***> 1***	
0-1 hr	0.2857	0.0214	0.0167	0.0043	0.0008	0.3289
1-2 hr	0.0164	0.0257	0.0221	0.0089	0.0025	0.0756
2-3 hr	0.0085	0.0223	0.0198	0.0083	0.0038	0.0627
3-6 hr	0.0099	0.0351	0.0475	0.0221	0.0087	0.1233
6-12 hr	0.0058	0.0337	0.0629	0.0528	0.0266	0.1818
12-24 hr	0.0024	0.007	0.0397	0.0611	0.0515	0.1617
>24 hr	0	0.0009	0.0043	0.0172	0.0435	0.0659
Sum	0.3287	0.1461	0.213	0.1747	0.1374	1.0

In order to generate a random storm event, Gupta (2013) first used Monte Carlo simulation using a uniformly distributed variate (u_D) to generate a random storm duration given the discrete cumulative probabilities of storm durations summarized in

Table 4-6. Once a storm duration was generated, a second random uniform variate (u_s) was generated to determine the total rainfall depth for the storm event. If this probability corresponded to a rainfall depth of less than or equal to 1 in., (u_s) was input into the following gamma distribution to generate the corresponding rainfall depth P_T (Gupta 2013):

$$p_s = \frac{P_T^{C_2 - 1} \exp - \dfrac{P_T}{C_1}}{C_1^{\,C_2} \Gamma_d (C_2)} \tag{4-43}$$

where p_s is the probability that the simulated total rainfall depth will be less than P_T (in.), C_1 is the shape parameter, C_2 is the scale parameter, and Γ_d is the gamma distribution function.

If a rainfall depth was greater than 1 in., a third uniform variate (u_b) was generated and input into an exponentially distributed rainfall model that described rainfall depths greater than 1 in. (Gupta 2013):

$$P_T = 1 - \frac{\ln(1 - u_b)}{\lambda} \tag{4-44}$$

where λ is a coefficient and P_T represents the resulting simulated rainfall depth for a given storm event (in.). Once storm durations and depths were generated, triangular, center-loaded design storms were generated and used to distribute rainfall intensity with the peak intensity occurring at the midpoint of the storm duration (Gupta 2013).

Table 4-6 Cumulative probabilities corresponding to storm durations derived from a study done by Kreeb (2003) with the Baltimore-Washington area (Gupta 2013).

D (hrs)	Cumulative Probability	D (hrs)	Cumulative Probability
1	0.352	13	0.852
2	0.433	14	0.875
3	0.500	15	0.896
4	0.551	16	0.914
5	0.594	17	0.931
6	0.632	18	0.946
7	0.669	19	0.958
8	0.704	20	0.969

D (hrs)	Cumulative Probability	D (hrs)	Cumulative Probability
9	0.738	21	0.979
10	0.769	22	0.987
11	0.799	23	0.994
12	0.827	24	1.00

4.4.1.2 Cell flow

Manning's Equation was used to compute the velocity of flow in a given cell. Surface runoff through a cell is dependent on the length of the cell, the flow velocity of the water in the cell, and the 1-minute time increment:

$$v_i = \left(\frac{1.486}{n}\right) R_{h(i)}^{2/3} \sqrt{S_L} \qquad (4\text{-}45)$$

where v_i represents the velocity (ft/s) in cell i, n is the roughness coefficient, $R_{h(i)}$ is the hydraulic radius (ft) in cell *i*, and S_L is the bottom elevation slope (ft/ft) from i to the receiving cell. The roughness coefficient n was determined by vegetative properties of a given cell as explained in the following section. The S_L was computed accordingly:

$$S_L = \frac{(EL_i + SS_i) - (EL_{i-1} + SS_{i-1})}{LC} \qquad (4\text{-}46)$$

where SS_i and SS_{i-1}, respectively, represent the surface water depth (ft) in cells i and i-1; EL_i and EL_{i-1} are the respective bottom elevations (ft) for cells i and i-1; and LC was the user-defined cell length, which was assumed to approximate the horizontal distance between cells i and i-1. Cell i-1 represented the cell receiving flow from cell i. Typical non-zero S_L values ranged from 1×10^{-8} to 1×10^{-4} ft/ft.

Given these small S_L values, it was reasonable to assume the velocity head within the conservation of energy equation to be zero. Therefore, only the hydraulic and elevation heads were incorporated in the movement of water through the wetland. $R_{h(i)}$ was assumed to equal the flow depth, or mean of adjacent cell surface storage water depths:

$$R_{h(i)} = \frac{SS_i + SS_{i-1}}{2} \tag{4-47}$$

The proportion (p_d) of water stored in a cell that is discharging flow to the next downgradient cell was defined accordingly:

$$p_d = 60 \cdot \frac{\Delta t \cdot v_i \cdot}{SS_i} \tag{4-48}$$

where Δt represents the time interval of 1 minute and SS_i is the depth of water (ft) in. If p_d is greater than 1, it was set to 1, which means that all water in the cell drains to the next downgradient cell. The outflow depth is the product of the proportion p_d and the surface storage depth SS_i in the cell:

$$dSS_i = p_d \cdot SS_i \tag{4-49}$$

where dSS_i is the outflow depth or change in storage (ft) and SS_i is the water depth in cell i at the start of the time interval.

4.4.1.2.1 Vegetated Flow

Wetland data have proven traditional methods of Manning's roughness coefficients (*n*) estimation to be inaccurate measures of wetland vegetated flow (Kadlec and Knight 1996). Data-derived Manning's roughness coefficients values generally range from 0.2 to 0.7 s/ft$^{1/3}$ with greater values of 0.7 to 1.3 s/ft$^{1/3}$ in water depths of 0.65ft or less (Hall and Freeman 1994; Kadlec and Knight 1996; USEPA 2000). These roughness values have not be found to accurately model the slow flows that occur in constructed wetlands (Kadlec and Knight 1996). Additionally, because wetland flow is generally found to occur in the transitional zone between turbulent and laminar flow, it is difficult to characterize (Kadlec 1990). Vegetated wetland flow has been estimated using a number of methods and equations based on turbulent, laminar, and both turbulent and laminar flows (Kadlec 1990; Reed et al. 1995; Kadlec and Knight 1996).

Kadlec and Knight (1996) developed a simple method for estimating the Manning's roughness coefficient based on water depth and vegetation density given literature wetland *n* values. Reed et al. (1995) also used a similar method to relate the wetland *n* value to an estimated vegetation resistance factor. The current study modified the method outlined by Kadlec and Knight (1996) as it incorporated data from a number of studies. In order to use this model, areas with emergent vegetation (e.g., *Typha* spp.) were assumed to be densely vegetated while areas with submerged vegetation (e.g., *Elodea* spp.) were assumed to be sparsely vegetated.

The following general equation was proposed to describe the roughness coefficient of densely vegetated areas of the wetland. The center equation was derived from Kadlec and Knight (1996) based on literature values. Limits were placed around this equation in order to create a complete model of the roughness coefficient in areas with emergent vegetation:

$$n_D = \begin{cases} 33.8 & \text{for } SS < 0.328\,\text{ft} \\ 0.673 \cdot (SS/3.28)^{-1.7} & \text{for } 0.328 \le SS \le 3.28\,\text{ft} \\ 0.673 & \text{for } SS > 3.28\,\text{ft} \end{cases} \qquad (4\text{-}50)$$

where n_D represents the roughness coefficient value (s/ft$^{1/3}$) for densely vegetated wetland areas, and *SS* represents the surface storage depth (ft). The following model, adapted in the same manner from Kadlec and Knight (1996), was used to characterize flow through areas with submerged vegetation in the wetland:

$$n_S = \begin{cases} 21.9 & \text{for } SS < 0.164\,\text{ft} \\ 0.673 \cdot 0.2 \cdot (SS/3.28)^{-1.7} & \text{for } 0.164 \le SS \le 3.28\,\text{ft} \\ 0.135 & \text{for } SS > 3.28\,\text{ft} \end{cases} \qquad (4\text{-}51)$$

where n_S represents the roughness coefficient value (s/ft$^{1/3}$) for sparsely vegetated wetland areas. Finally, if vegetation is not present in a wetland cell, the corresponding n is set equal to 0.10 s/ft$^{1/3}$. The relationships between surface storage depth, and both n_D and n_S values are shown in Figure 4-15. Given normal water depths with emergent vegetation ranging from 1-2.5 ft, n_D should range from 4.8 down to 1.1 s/ft$^{1/3}$. With a typical range of submerged depths of 2.5 to 5 ft, the n_S should be greater than or equal to 0.135 under normal wetland conditions.

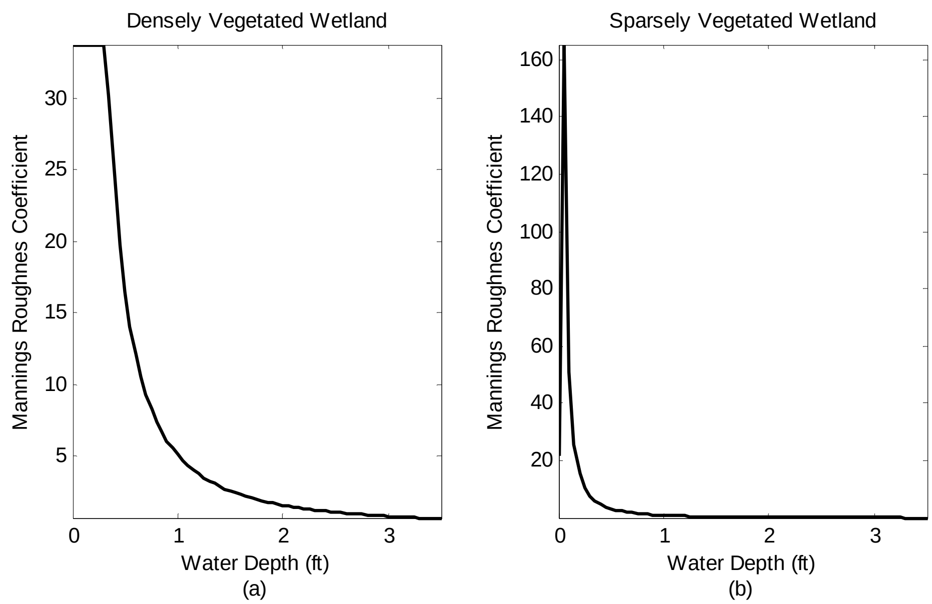

Figure 4-15 Resulting n_D (a) and n_S (b) values for varying surface storage depths.

4.4.1.3 Potential evapotranspiration

Hourly PET depths computed from the PET model component discussed in Section 4.2, were divided by 60 min/hr, converted to units of ft and subtracted from each wetland cell over each minute of simulation. These resulting 1-min potential evapotranspiration depths PET_m were compared with the existing water depth SS_i a given in cell *i* at minute *m* in order to ensure sufficient water was available to remove all PET_m. If PET_m was greater than SS_i, PET_m was set equal to SS_i and SS_i was set equal to zero for the next time interval. Therefore, the final value of PET_m represented the actual ET (AET) occurring based on water availability.

4.4.1.4 Infiltration

A constant infiltration depth was subtracted from each cell over 1-min each time step based on soil permeability and hydraulic conductivity in each cell:

$$I = \frac{1}{1440} \cdot K_V \cdot \Delta t \tag{4-52}$$

where K_V is the vertical hydraulic conductivity of the wetland media (ft/d), I represents the resulting depth of water lost to infiltration in a wetland cell during a given 1-min time interval (ft), 1440 represents the conversion factor from days to minutes. Corresponding 1-min infiltration volumes (ft^3) were computed for a given cell and time interval by multiplying I by the corresponding cell area. The cumulative annual wetland infiltrated volume for a given year of simulation I_{KV} was computed by summing all 1-min infiltration volumes from all wetland cells over that year. A final average annual infiltration volume $\bar{I}_{KV}$ was then computed over the simulation period for the calculation of PC_{GW} (see Equation 3-22).

K_V was a user input and was dependent on the media chosen for the wetland bottom. A number of wetland designs require a liner to be installed in order to avoid contamination of groundwater and/or to prevent drying out of the wetland via infiltration, which are simulated with a K_V of 0. Wetland media K_V requirements will depend on site conditions such as rainfall frequency, existing soil hydraulic conductivities, water table height, and influent water pollutant levels. This simple infiltration module assumes vertical infiltration rates to be equal to that of the wetland media saturated hydraulic conductivity. Future versions of the model may include a more complex representation of infiltration. However, given the saturated nature of wetlands, the use of the hydraulic conductivity was assumed a sufficient estimate of infiltration given the scope of the current study.

4.4.1.5 Wetland Inflow

Inflow into stormwater treatment wetlands was estimated by the Rational formula while inflow into municipal wastewater treatment wetlands was simulated by input curves that followed the basic shape of municipal water demand. Agricultural wastewater flow was also assumed to be constant over a given simulation period. The following subsections describe in detail how both inflows were generated within the model.

4.4.1.5.1 Inflow from a drainage area

The Rational method was used to compute the peak flow for a triangular hydrograph that corresponded to the generated rainfall that was used to represent flow into the inlet cell(s):

$$q_p = C \cdot i_R \cdot DA \tag{4-53}$$

where q_p is the peak flow (cfs), C is the runoff coefficient, i_R is rainfall intensity (in./hr), and DA is the drainage area (acres). The corresponding runoff volume V_R produced by the drainage area over one storm event was also computed by multiplying the rational C by the corresponding storm rainfall volume over the contributing drainage area:

$$V_R = 3630 \cdot C \cdot P_T \cdot DA \tag{4-54}$$

where P_T is the rainfall depth (in.) for a given storm event (see Section 4.4.1.1) and V_R is the resulting runoff volume (ft^3) from the drainage area DA. The value 3630 was a conversion factor. The storm time of concentration t_c was then computed based on V_R and q_p:

$$t_c = \frac{1}{60} \cdot \frac{V_R}{q_p} \tag{4-55}$$

where the 1/60 term was a conversion factor from seconds to minutes. As defined by the Rational method, the q_p occurs at the end of the rainfall (at the time of concentration t_c), and runoff ends at $2t_c$. With these parameters defined, 1-min flows for a given influent hydrograph were calculated with the following equation:

$$Q_{IN}(t) = \begin{cases} \left(\frac{q_p}{t_c}\right)t & \text{for t} \le t_c \\ 2q_p - \left(\frac{q_p}{t_c}\right)t & \text{for t} > t_c \end{cases} \tag{4-56}$$

where $Q_{IN}(t)$ is the flow rate (cfs) at time t (min) and t_c is the storm time of concentration (min). The resulting hydrograph $Q_{IN}(t)$ was assumed to reach the wetland through a pipe where it would flow into a level spreader, over a weir, and into the forebay or inlet cell(s) to the wetland.

4.4.1.5.2 Inflow from a WWTP (Method 1)

Inflow rates from a municipal WWTP can be input into the model based on estimated seasonal, daily curves. Due to its complexity, municipal water demand can be difficult to predict (Tchobanoglous et al. 2003; Viessman et al. 2009). A number of methods have been employed to predict municipal water demand and resulting flows, which include time-series analysis, artificial neural networks, and autoregressive modeling (Zhao et al. 2001; Zhou et al. 2002; Alvisi et al. 2007;

Herrera et al. 2010). Such complex models require data that may not always be readily available to the model user and are often computationally expensive. The current study used simple seasonal curves to estimate WWTP inflow based on location and the service area population.

Literature values were used to estimate influent municipal wastewater discharge rate magnitudes and trends. An average consumption of 180 gallons per capita per day (gpcd) was assumed for all domestic water use in the United States according to the estimate of Viessman et al. (2009) for the year 2000. The USEPA (2004) reported an average consumption rate of 171 gpcd for the mid-Atlantic area over the years 1975-1993. Viessman et al. (2009) estimated that the average winter flow was about 80% of the annual average while summer demand was 125% that of the annual average. It is also documented in the literature that two peaks in domestic water consumption occur within the daily cycle (Viessman et al. 2009). These peaks were estimated to occur between 7am and 1pm, and 5pm and 9pm (Viessman et al. 2009). Additionally, the second peak increases in magnitude during the summer months due to the increased water demands of lawn irrigation, pools, etc.

The most recent estimate of mean domestic water consumption 180 gpcd was used to calculate mean winter and summer consumption rates of 144 and 225 gpcd. The mean per capita consumption for the fall and spring months was estimated to equal the annual average of 180 gpcd. Given these average daily consumption rates, maximum and minimum hourly values were estimated in order to develop input WWTP hourly flow curves for each season (winter, spring, summer, and fall). According to Davis (personal communication 2009), the maximum daily flow was estimated to be 1.8 times the daily mean and the maximum hourly flow was estimated to equal 1.8 times the maximum daily flow. Therefore, hourly maximum rates were 3.24 times the daily mean rate. This concept was applied to each seasonal flow estimate, resulting in peak hourly winter, spring, summer, and fall consumption rates of 467, 583, 729, and 583 gpcd.

Once the peak flow magnitudes were estimated, the time of these peaks was defined. According to Viessman et al. (2009), the first peak was estimated to occur at 10am while the second was assumed to occur around 8pm. In addition, the first (smaller) peak for summer months was estimated based on a figure given in Viessman et al. (2009). Winter morning and afternoon peaks were assumed to be equal. From the figure in Viessman et al. (2009), the afternoon summer peak was estimated to be 1.25 times that of the morning peak. Therefore, a morning summer peak of 583 gpcd was estimated. The afternoon peak for fall and spring months was also assumed to be slightly larger than the morning peak and was estimated to equal 1.15 times that of the morning peak, which was the mean of the ratio of afternoon to morning peaks for winter (1) and summer (1.25) months. Therefore, the spring and fall, and summer morning peaks were estimated to equal 518 and 583 gpcd. Again, from the figure given in Viessman et al. (2009), a minimum water consumption rate of 50 gpcd was assumed for all seasons.

From the estimate flows above, three curves were generated to represent hourly flows for winter, summer, and spring and fall months. WWTP flow required the input of the service area population. The input curves were multiplied by the population and a conversion factor in order to generate inflow flows with units of cfs into the wetland. The following curves resulted:

$$Q_W = \frac{0.13P_n}{24 \cdot 3600T_n}\left[250 + 220\sin(0.55h - 4)\right] \tag{4-57}$$

$$Q_F = \begin{cases} \dfrac{0.13P_n}{24 \cdot 3600T_n}\left[275 + 250\sin(0.55h - 4)\right] & \text{for } 0 \le h \le 19 \\ \dfrac{0.13P_n}{24 \cdot 3600T_n}\left[275 + 300\sin(0.55h - 4)\right] & \text{for } h > 19 \end{cases} \tag{4-58}$$

$$Q_S = \begin{cases} \dfrac{0.13P_n}{24 \cdot 3600T_n}\left[300 + 280\sin(0.55h - 4)\right] & \text{for } 0 \le h \le 19 \\ \dfrac{0.13P_n}{24 \cdot 3600T_n}\left[300 + 450\sin(0.55h - 4)\right] & \text{for } h > 19 \end{cases} \tag{4-59}$$

where Q_W is the winter WWTP wetland inflow (cfs), Q_F is the spring and fall WWTP wetland inflow (cfs), Q_S is the summer WWTP wetland inflow (cfs), P_n is the service area population (# of people), T_n is the total number of parallel trains desired in the wetland, and *h* is the hour of the day (hr). WWTP are often divided into parallel trains in order reduce the flow entering one given wetland train as well as to allow for continuous flow during maintenance (USEPA 2000). An example set of input curves are shown in Figure 4-16 for an WWTP serving a population of 25,000 people with a treatment wetland composed of two parallel flow trains. The inflow curves represent flow entering one train (or half of the total flow entering the WWTP). Seasons were specified by DOY in the model based on the dates of solstices and equinoxes and are compiled in Table 4-7.

Table 4-7 Assigned day-of-year (DOY) ranges for each season within the model.

Season	**DOY Range (days)**
Winter	1-78 and 355-365
Spring	79-171
Summer	172-265
Fall	266-354

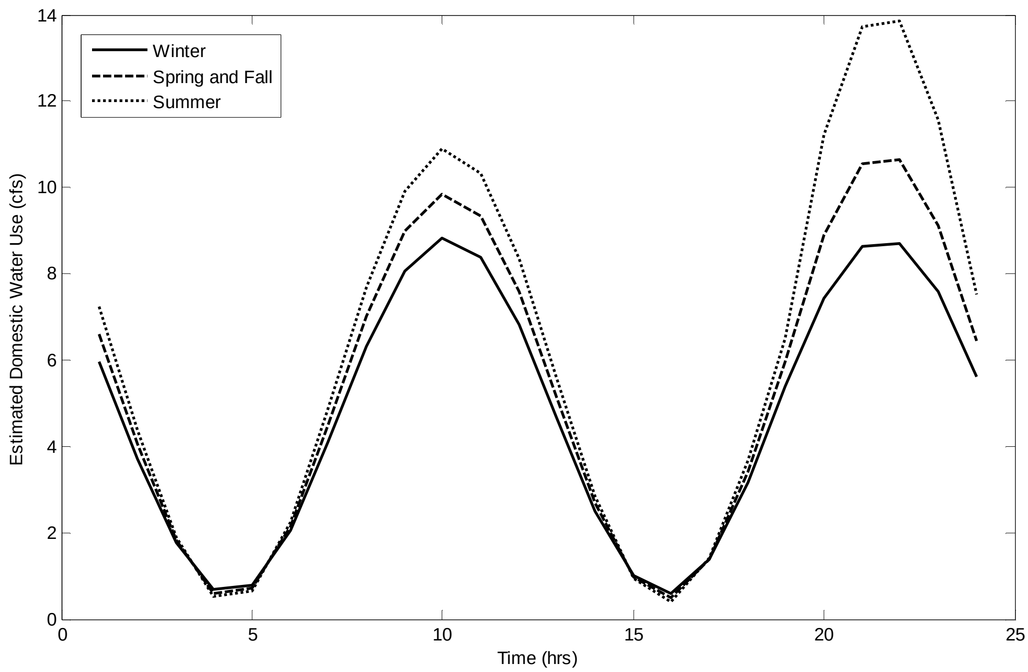

Figure 4-16 Shows the generated inflow winter (solid), summer (dotted), and spring and fall (dashed) curves (cfs) into one wetland train serving a population of 25,000 people.

4.4.1.5.3 Inflow from a WWTP (Method 2)

Six and a half years (2007 through June 2013) of inflow data was obtained from a local WWTP, which serves about 1.8 million residents and treats and average of 30 million gallons per day (MGD) of wastewater. It would not be reasonable to design a treatment wetland to serve such a large WWTP. The current study, however, used the data from this large WWTP to estimate inflow to a smaller, hypothetical

WWTP, for which a constructed wetland would be a rational option for secondary treatment. It was suggested via personal communication with a Process Control Engineer at the WWTP that the diurnal curve may be dampened in the data due to the large distance between the plant and many service regions. Extended flow records of zero MGD (extending over at most 2 days) were cited by the Proccess Control Engineer as either meter failure or as missing data. All zero-flow values were set equal to the corresponding hourly value of the previous day. The full record of corrected hourly and mean daily data is plotted in Figure 4-17.

Curves of hourly WWTP flow were generated and scaled down with respect to flow magnitude from the flow data plotted in Figure 4-17in order to estimate reasonable wastewater inflow to a secondary wastewater treatment wetland. In order to develop a simple input curve for the model, hourly values from the period of record were averaged for each month. Figure 4-18 shows resulting averaged hourly flows for each month. While all curves followed a similar two-peak form, their magnitudes varied over the course of the year. Late winter (February) and early spring (March through May) had the highest flowrates, which is most likely due to larger amounts of infiltration into the delivery pipes to the WWTP from rainfall.

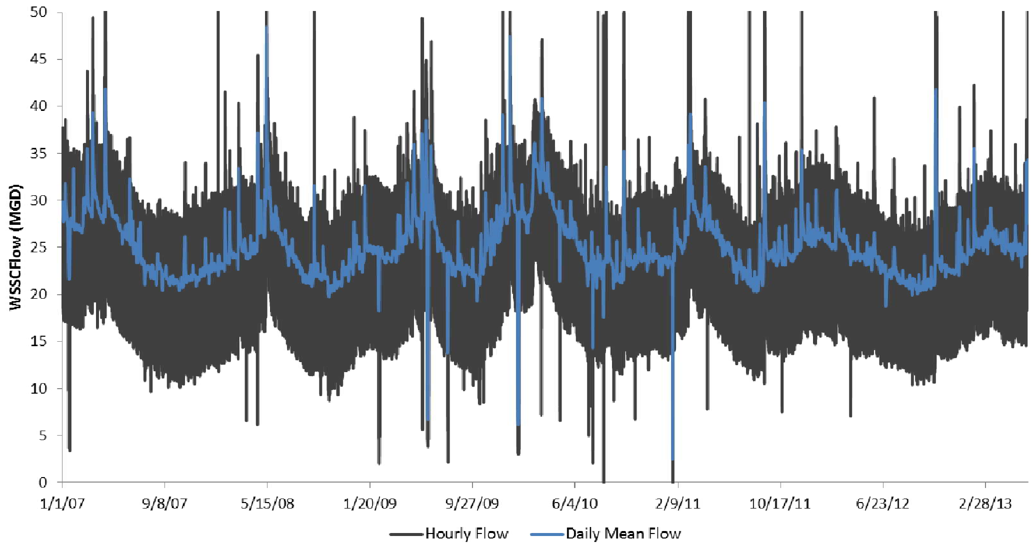

Figure 4-17 Hourly and mean daily flows entering the WWTP from 2007 through June 2013. All flows are in MGD.

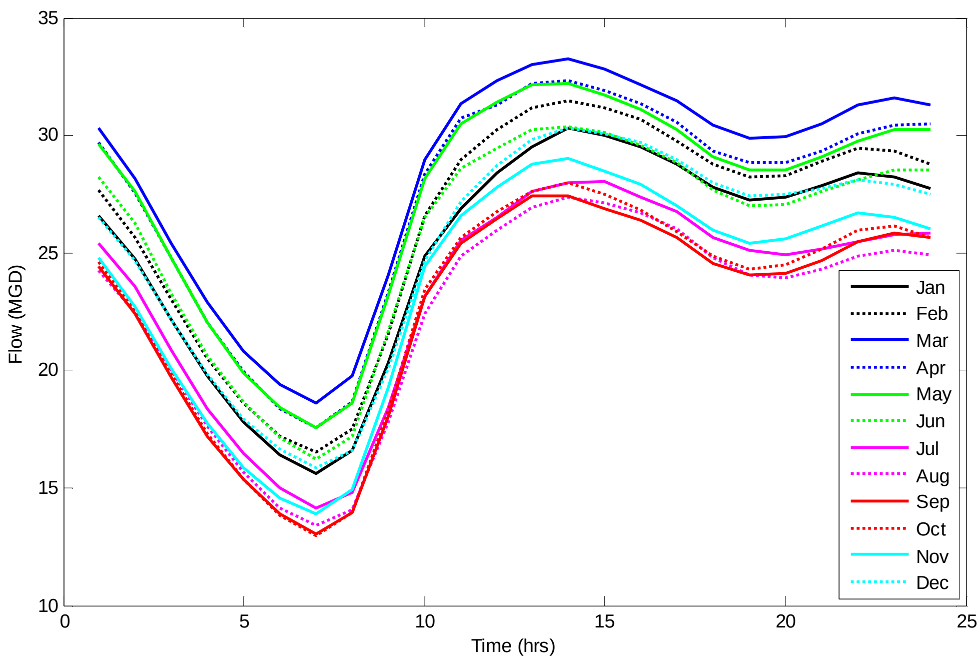

Figure 4-18 Hourly flows into the WWTP as averaged monthly.

From these hourly curves, three seasonal curves were derived based on relative flow magnitude, 1) from February through June, 2) from July through November, and 3) from December through January. These curves were derived by averaging all corresponding hourly values for all months within a given seasonal group. Composite sinusoidal models were fit to each of the three curves:

$$Q_S = \begin{cases} 24 + 6.6\sin(0.53h + 1.4) - 1.2\sin(1.07h + 5.7) & \text{for } 1 \le h < 12 \\ 30 - 1.1\sin(0.35h + 6.3) + 0.94\sin(0.7h + 10) & \text{for } 12 \le h < 24 \end{cases} \quad (4\text{-}60)$$

$$Q_W = \begin{cases} 23 + 6.6\sin(0.45h + 1.9) - 0.91\sin(0.9h + 0.824) & \text{for } 1 \le h < 12 \\ 28 + 1.3\sin(0.35h + 2.4) + 1.2\sin(0.7h + 17) & \text{for } 12 \le h < 24 \end{cases} \quad (4\text{-}61)$$

$$Q_F = \begin{cases} 20 + 6.3\sin(0.53h + 1.4) - 1.2\sin(1.07h + 5.8) & \text{for } 1 \le h < 12 \\ 26 + 1.1\sin(0.35h + 2.9) - 1.1\sin(0.7h + 7.6) & \text{for } 12 \le h < 24 \end{cases} \quad (4\text{-}62)$$

where Q_S, Q_W, Q_F are the respective resulting flows entering WSSC during the previously defined flow seasons of spring (February through June), winter (December through January), and fall (July through November); and h is the hour of day (1-24 hr). Goodness-of-fit statistics were then computed on hourly flowrates yielded by the above equations with respect to the original hourly data plotted in Figure 4-17. The resulting RMSE, $\bar{e}$, $\bar{e}/\bar{y}$, and S_e/S_y were respectively 3.94 MGD, 0.161 MGD, 0.0065, and 0.639. Based on the low bias and reasonable S_e/S_y produced by the simulated hourly flowrates, the Q_S, Q_W, and Q_F curves were assumed to be sufficient estimates of hourly the inflow flow to the WWTP. The final Q_S, Q_W, and Q_F curves are plotted with the averaged data values in Figure 4-19.

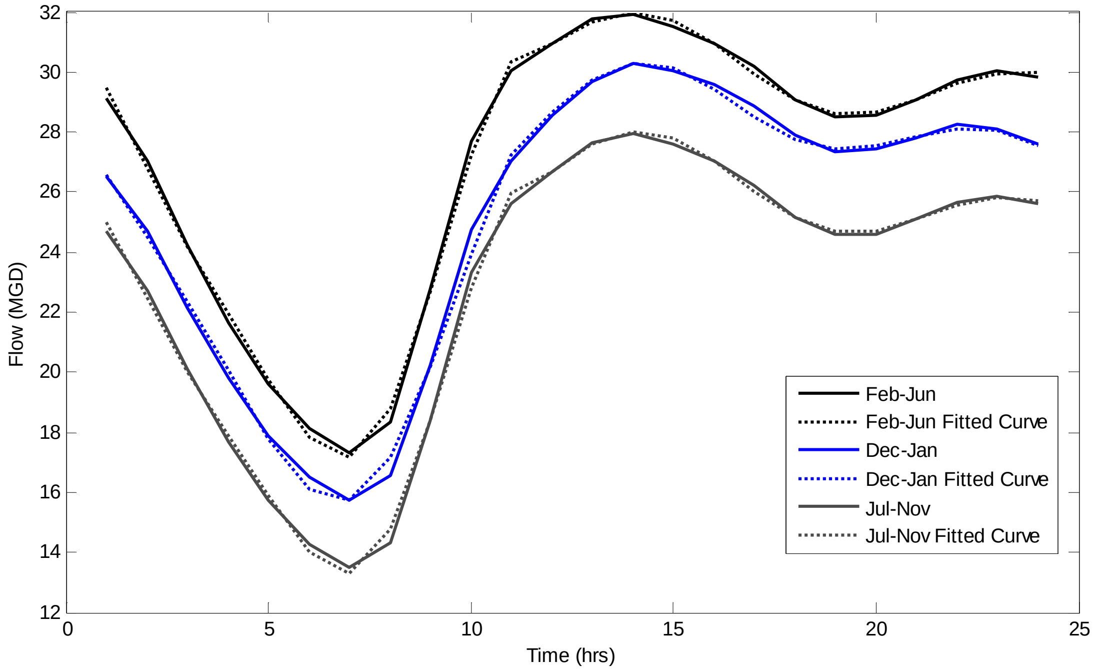

Figure 4-19 Resulting averaged hourly values for the three designated seasons, Feb-June (solid black), December-January (solid blue), and July-November (solid gray); and the corresponding generated composite curves input into the model (dotted black, blue, and gray lines).

Once seasonal flow curves were established, all curves were linearly scaled-down in order to simulate wastewater flows feasible for secondary wetland treatment. USEPA (2000) cites that treatment wetlands require between 4 and 25 acres of area for each million gallon of flow per day depending on the wastewater inflow quality as well as the water quality requirements of the wetland effluent. Therefore, facilities are limited by area when using treatment wetlands. A WWTP with mean flowrates of 30 MGD would require a treatment wetland anywhere from 120 to 750 acres depending on water pollutant content. 90% of all wastewater treatment wetlands are less than 250 acres, while the majority of wastewater treatment wetlands have an area of 25 acres or less (USEPA 2000). Similarly 82% of all WWTPs using treatment wetlands have mean flowrates of 1 MGD or less (USEPA 2000).

Scaling down wastewater flow and the corresponding water consumption of a service area can be very complicated. Due to the shifts in population characteristics, this process is not linear. Additionally, each WWTP experiences flow unique to the population it serves. Therefore, in order to properly model WWTP flow, data from the specific WWTP is needed. Since the purpose of the case study was only to demonstrate the value of the model, the hourly flow curves were scaled down linearly according to the desired mean flowrate. Additionally, the WWTP influent curves derived in Figure 4-19 were assumed to equal the influent curves into secondary treatment as the Process Control Engineer at the WWTP stated that the flow removed from primary treatment via primary sludge withdrawal was "rather insignificant" (personal communication 2013). This scale-down method also assumed that the scaled-down service area was characteristically identical to that of the original WWTP area, the only difference being the population. This method is not suggested for real-world treatment wetland design. Flow curves should always be generated from the actual facility in order to obtain accurate wetland designs. The current study only aimed to show the performance of a constructed wetland for secondary treatment in an example WWTP and did not aim to directly design a wastewater treatment wetland.

4.4.1.5.4 Input from Agricultural Wastewater treatment storage

Sufficient data were not found to generate hourly inflow values to agricultural wastewater wetlands. A majority of agricultural wastewater treatment wetlands are preceded by pretreatment storage facilities (Cronk 1996; Knight et al. 2000). Therefore, it was assumed that wastewater entered these wetlands at a relatively constant flowrate. If additional information about inflow rates was available for a given agricultural wastewater treatment wetland design, variation could also be added to inflow rates to generate an inflow rate distribution.

4.4.1.6 Berm and weir flow

Flow was simulated using the same method over both internal berms and outlet weirs. Internal berms were assumed to have a length equal to that of the cell width and a user-defined elevation above the datum. Both outlet weir lengths and elevations were user-defined inputs to the program. Flow over a berm or weir was initially calculated using the weir equation:

$$Q_w(t) = C_w \cdot L_w \cdot d_w(t)^{1.5} \tag{4-63}$$

where $Q_w(t)$ is the flowrate over the weir (cfs) at time t (min), C_w is the weir coefficient (3.0 for English units), L_w is the length (ft) of the weir, and $d_w(t)$ is the depth (ft) of flow above the invert of the weir (ft) at time *t*. The resulting outflow depth of water exiting the berm or weir $d(t)$ (ft) during the same time interval was calculated by multiplying $Q_w(t)$ by the time interval Δt (1-min) and dividing by the outflowing cell area A_C (ft^2):

$$d(t) = \frac{60 \cdot Q_w(t) \cdot \Delta t}{A_C} \tag{4-64}$$

A water balance was performed during each time interval to check if the volume of water exiting the berm or weir $d(t)$ was greater $d_w(t)$. If $d(t)$ was greater than $d_w(t)$, it was set to equal $d_w(t)$. Once $d(t)$ was established for a given time interval, the new cell depth $SS_i(t)$ in ft was defined:

$$SS_i(t) = SS_i(t-1) - d(t) \tag{4-65}$$

This final value of $SS_i(t)$ then became the initial $SS_i(t-1)$ storage depth (ft) for cell *i* for the next time *t+1*. A time interval of 1 min was found to be optimal for berm and weir flow. The model collected irrationally large depths of water on the level spreader if longer time intervals were used. As a result, the model used a time increment of 1-min for all hydrologic calculations. The proportion p_d of water exiting the cell relative to the initial cell depth $SS_i(t-1)$ was calculated for berm and weir flow for later movement of water quality constituents:

$$p_d = \frac{d(t)}{SS_i(t-1)} \tag{4-66}$$

4.4.1.7 Outflow from the wetland

Outflow from the wetland can be controlled using either a weir or an orifice. For orifice flow, the width and length of the orifice, and the invert elevation must be specified by the user. For weir flow, a weir length and the invert elevation are required inputs. Before subtracting the volume of outflow from the first cell, a check is made to ensure that sufficient water is in storage above the invert.

4.4.1.7.1 Orifice Flow

Within the model, orifices were assumed to have a rectangular shape. Orifice flow from the outlet cell could also be calculated using the orifice equation:

$$Q_O(t) = \begin{cases} 0 & \text{if } EB(t) + SS(t) < E_i \\ C_w \cdot W_O \cdot (EB(t) + SS(t) - E_i)^{1.5} & \text{if } E_i < EB(t) + SS(t) \le E_i + \mathrm{H_o} \\ C_o A_O \cdot \sqrt{2g(EB(t) + SS(t) - E_i)} & \text{if } EB(t) + SS(t) > E_i + \mathrm{H_o} \end{cases} \tag{4-67}$$

where $Q_O(t)$ is the flowrate (cfs) of the outflowing water from the orifice at time *t*, C_O is the discharge coefficient of the orifice (dimensionless), A_O is the orifice area (ft^2), H_O is the orifice height (ft), W_O is the orifice width (ft), g is gravity in English units (ft/s^2), $SS(t)$ is the surface water storage depth (ft) in a given cell at time *t*, and *EB* is the distance (ft) from the datum to the wetland bottom, E_i is the distance from the datum to the bottom of the orifice or the invert elevation (ft). If *SS* was less than $E_i + H_O$, the orifice behaved as a weir, and the weir equation was used to calculate outflow discharges, where the diameter of the orifice was estimated to equal the weir length and the depth of water above the weir was assumed to equal. Figure 4-20 shows an example of orifice flow.

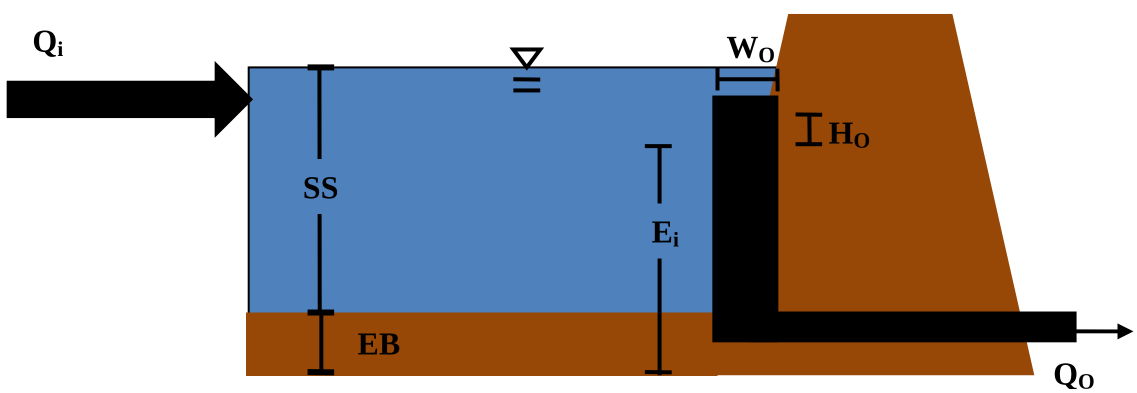

Figure 4-20 Flow through an orifice in the outflow cell (cell 1) and the resulting velocity Q_i (ft/s). Q_O (ft/s) represents the velocity water flowing out of the orifice, SS is the surface storage depth (ft), W_O is width of the orifice, H_O is the orifice opening height, *EB* is the depth of water from the datum to the wetland bottom (ft), and E_i is the invert elevation above the datum(ft).

4.4.1.8 Pre-development runoff simulation

A number of the sustainability metrics developed in Chapter 3 required the simulation of flow over the wetland drainage area under pre-developed conditions for wetlands treating stormwater runoff. Therefore, pre-development flows were simulated for a given wetland site within the model. The pre-developed drainage area was assumed to have an area equal to that of the contributing drainage area to the wetland plus the area of the wetland itself. Flow from the pre-developed drainage area was simulated based on the same rainfall events affecting the wetland and its contributing drainage area. Additionally, pre-developed flows were computed with the rational method in the same manner as were influent flows to the wetland the only difference being the input rational C value:

$$q_{p(PRE)} = C_{PRE} \cdot i_R \cdot DA_{PRE} \tag{4-68}$$

where $q_{p(PRE)}$ is the peak pre-development flow (cfs), DA_{PRE} is the pre-development drainage area (ac), and C_{PRE} is the runoff coefficient. C_{PRE} like C was a user-defined input that was based on the land use characteristics of the estimated pre-developed drainage area. The corresponding runoff volume $V_{R(PRE)}$ produced by the pre-developed drainage area over one storm event was also computed by multiplying C_{PRE} by the corresponding storm rainfall volume over the contributing drainage area:

$$V_{R(PRE)} = 3630 \cdot C_{PRE} \cdot P_T \cdot DA_{PRE} \tag{4-69}$$

where P_T is the rainfall depth (in.) for a given storm event (see Section 4.4.1.1) and $V_{R(PRE)}$ is the resulting runoff volume (ft^3) from the drainage area DA_{PRE}. The value

3630 is a conversion factor. The pre-development flow time of concentration t_c was then computed based on $V_{R(PRE)}$ and $q_{p(PRE)}$:

$$t_c = \frac{1}{60} \cdot \frac{V_{R(PRE)}}{q_{p(PRE)}} \tag{4-70}$$

where the 1/60 term was a conversion factor from seconds to minutes. Pre-development 1-min discharge rates were also computed in the same manner as were wetland inflow discharge rates:

$$Q_{PRE}(t) = \begin{cases} \left(\dfrac{q_p}{t_c}\right)t & \text{for t} \le t_c \\ 2q_p - \left(\dfrac{q_p}{t_c}\right)t & \text{for t} > t_c \end{cases} \tag{4-71}$$

where $Q_{PRE}(t)$ is the flow rate (cfs) at time t (min) and t_c is the storm time of concentration (min).

In addition to pre-development discharge rates and volumes, the model also computed annual pre-development drainage area infiltration for the computation of the groundwater recharge and baseflow maintenance performance criterion PC_{GW} (see Section 3.4.5). Pre-development infiltration volumes for each storm event were computed by subtracting the storm runoff volume $V_{R(PRE)}$ from the corresponding precipitation volume $V_{P(PRE)}$ over the pre-developed drainage area DA_{PRE}. Precipitation volume over DA_{PRE} was first calculated accordingly:

$$V_{P(PRE)} = 3630 \cdot P_T \cdot DA_{PRE} \tag{4-72}$$

With $V_{P(PRE)}$ computed, the corresponding storm infiltration volume $V_{I(PRE)}$ could be computed:

$$V_{I(PRE)} = V_{P(PRE)} - V_{R(PRE)} \tag{4-73}$$

where $V_{I(PRE)}$ is the total volume of water (ft^3) infiltrated over DA_{PRE} for a given storm event. Resulting $V_{I(PRE)}$ values for all storm events within a given year were summed to compute the corresponding annual pre-development infiltration volume I_{PRE} for that year. All annual I_{PRE} values for a given simulation period were then averaged to compute a final $\bar{I}_{PRE}$ value for the site.

4.4.2 Water Quality Simulation

The water quality portion of the model simulates ammonium (NH_4^+), nitrate (NO_3^-), total suspended solids (TSS), and dissolved oxygen (DO) dynamics throughout a given wetland design. In order to characterize these chemical concentrations, wetland photosynthesis, respiration, and transformation rates were estimated and calibrated based on relevant, geographically appropriate data.

All reactions were discretized based on a 1-min time interval using the Euler's method, assuming each wetland cell behaved like a completely mixed flow reactor (CMFR), which implied that all constituent concentrations were vertically and horizontally uniform within each cell. Figure 4-21 shows an example CMFR cell given a general constituent concentration C and flow Q. The CMFR method assumes that the concentration within a cell is uniform and equal to the concentration leaving the cell at the end of a given time interval. A mass balance of each constituent was calculated at the end of each year of simulation to ensure that each water quality constituent followed the law of conservation of mass.

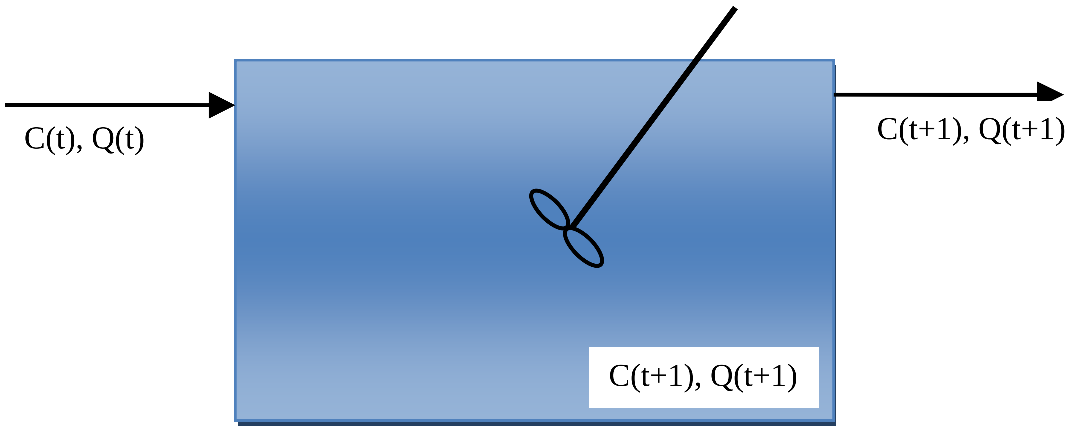

Figure 4-21 Schematic of a CMFR cell, where the concentration and flow entering the cell are C(t) and Q(t); and the concentration and flow values within and leaving the cell are C(t+1) and Q(t+1).

Due to the complexity of wetland water quality, a number of simplifications and assumptions were made. Sufficient amounts of carbon, nitrifying and denitrifying bacteria, and phosphorus were assumed to be present and available for all relevant reactions within the wetland. A circumneutral pH was also assumed throughout the wetland. Additonally, wetland primary productivity was simplified; once particles or dead vegetation settled on the wetland bottom, they were assumed to be inactive, becoming a permanent portion of the anoxic wetland sediment. Resuspension of particles was not modeled, which is a reasonable assumption due to the slow velocities, around 1.25 m/hr (4.10 ft/hr), typically observed in constructed wetlands (USEPA 2000).

The following procedure was followed to determine water quality values for surface water for each time interval. The + and – signs in parenthesis indicate whether a given step contributes (+) or demands (-) DO. Additionally, DO levels were recalculated after each step that creates or consumes DO:

1. Calculate TSS settling/trap efficiency
2. Calculate initial DO concentration for time interval
    a. Photosynthesis (+)
    b. Surface Aeration (+/-)
    c. Respiration (-)
3. Calculate nitrification given DO levels $NH_4^+ \rightarrow NO_2^- \rightarrow NO_3^-$ (-)
4. Calculate denitrification $NO_3^- \rightarrow N_{2(g)}$

Concentrations of water quality constituents were determined after each time interval using discretized mass balance equations following the Euler's method. The following sections define the mass balances used for each constituent. A number of the water quality constituents are interrelated, and the resulting code reflects this. Additionally the reactions dictating nitrification required aerobic conditions, while those controlling denitrification required anaerobic conditions. As suggested by USEPA (2000) and Sykes (2003) aerobic water was defined by a DO concentration of greater than or equal to 2 mg/L and anaerobic water was distinguished by a DO concentration less than 2 mg/L.

4.4.2.1 Total Suspended Solids (TSS)

TSS reduction for each wetland cell was directly simulated by settling as modeled by Stoke's Law:

$$v_s = 3.28 \cdot \frac{(\rho_p - \rho_w)}{18\mu} g_m \cdot D^2 \quad (4\text{-}74)$$

where v_s is the settling velocity (ft/s), ρ_p is the particle density (kg/m^3), ρ_w is the water density (kg/m^3), μ is the fluid dynamic viscosity (N-s/m^2), g_m is gravity (m/s^2), and D is the particle diameter (m). Corresponding trap efficiencies were then calculated for each cell for a given time interval assuming completely mixed conditions:

$$TE = 60 \cdot \frac{v_s \cdot \Delta t}{SS} \quad (4\text{-}75)$$

where Δt represents the time interval (min), *SS* is the cell surface storage (ft), and *TE* is the resulting cell trap efficiency of TSS. The total settling depth d_s within a given cell over the time interval was computed by multiplying Δt and v_s:

$$d_s = v_s \cdot \Delta t \quad (4\text{-}76)$$

This settling depth d_s was the total depth TSS particles fell within a time interval of 1-min for a given cell (see Figure 4-22). If the resulting *TE* was computed to be greater than 1.0, the model restricted it to equal 1.0. Therefore, the TSS load removed from settling from a cell was calculated according for each time step:

$$SET = TE \cdot TSS \quad (4\text{-}77)$$

where *TSS* represents the initial TSS load in the cell (mg) and *SET* represents the load of TSS removed from the cell by settling within the time interval. This method of modeling TSS simulation assumes that TSS particles redistribute evenly through a cell after each 1-min increment. While this assumption may not represent reality given that wetland water velocities are generally slow, it was the most reasonable method given CMFR (completely mixed flow reactor) cell structure of the model. Calibration of the TSS particle diameter may also help to correct discrepancies between the model and reality. "Filtration" of suspended solids by vegetation was also assumed to be negligible in the model as Kadlec and Knight (1996) found such removal efficiencies to be "vanishingly small." TSS levels were not allowed to go below a user-specified background concentration. The corresponding background TSS load in each cell was defined as TSS_o.

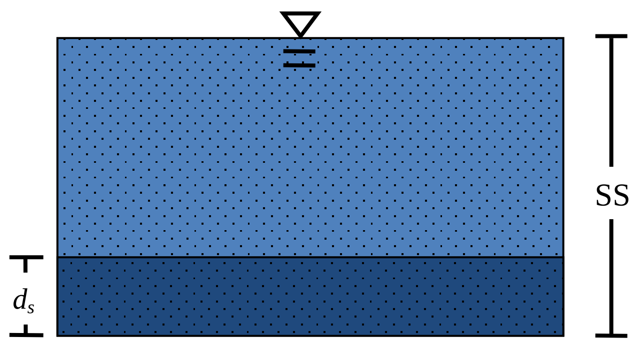

Figure 4-22 Depiction of TSS settling in a given cell within a 1-min time interval where SS is the total water depth in the cell and d_s is the vertical distance travel by TSS particles within 1-min. The darkly shaded region corresponding to d_s was settled out of the cell within that time interval.

4.4.2.2 Nitrogen Transformations

Nitrification and denitrification were both simulated within each model cell as they were cited to be the most important mechanisms responsible for long term nitrogen removal in treatment wetlands (Kadlec and Knight 1996; USEPA 2000; Bastviken 2006). All simulated nitrogen species were assumed to be in a dissolved form. Because wetlands have been found to have circumneutral pH values, volatilization of NH_3 was considered negligible (Kadlec and Knight 1996; USEPA

2000). Additionally, at a pH of 7, about 99.4% of ammonia nitrogen is in the ionized form NH_4^+ (Kadlec and Knight 1996; USEPA 2000). Therefore, the current study simulated only NH_4^+ within the model. As the carbon cycle was not simulated within the model, it was also assumed that sufficient carbon sources were available to promote both nitrification and denitrification within the wetland. The assumption was reasonable given that constructed wetlands are generally highly productive ecosystems (Mitch and Gosselink 1993; Kadlec and Knight 1996).

Nitrification is defined as the oxidation of NH_4^+ to NO_2^- to NO_3^-:

$$NH_4^+ + 1.5O_2 \longrightarrow NO_2^- + 2H^+ + H_2O \tag{4-78}$$

$$NO_2^- + 0.5O_2 \longrightarrow NO_3^- \tag{4-79}$$

Because the transformation of NH_4^+ to NO_2^- is generally the limiting reaction (meaning it occurs at a slower rate than the transformation of NO_2^- to NO_3^-), it is valid to assume all NH_4^+ is converted to NO_3^- for modeling purposes (Chapra 1997). Overall nitrification can then defined by the following chemical expression (Chapra 1997; Kadlec and Knight 1996):

$$NH_4^+ + 1.5O_2 \longrightarrow NO_3^- + 2H^+ + H_2O \tag{4-80}$$

This transformation was modeled by volume-based first-order kinetics:

$$TAMNIT = V \cdot K_{NIT} \cdot \left[NH4 - NH4_o\right] \cdot \theta_{NIT}^{T_W - 20} \tag{4-81}$$

where *NH4* represents the NH_4^+ concentration within a cell at a given time interval (mg-N/L), $NH4_o$ is the background NH_4^+ concentration in the cell (mg-N/L), θ_{NIT} is the nitrification temperature correction factor (defaulted as 1.07 in HSPF), K_{NIT} is the first-order nitrification reaction rate (hr^{-1}), V is the water volume (L) in the cell, and

$TAMNIT$ is the resulting NO_3^- load (mg-N/L) generated from nitrification. $TAMNIT$ was then converted into terms of mg-O_2 using a ratio of 4.3 mg-O_2/mg-N, as derived by Equations 2-15 and 2-16 (Kadlec and Knight 1996; USEPA 2000; Bicknell et al. 2001):

$$DODEMD = 4.3 \cdot TAMNIT \qquad (4\text{-}82)$$

where $DODEMD$ represents the load of O_2 consumed from nitrification within a given cell and time interval. Nitrification was simulated only if DO levels were greater or equal to 2 mg/L.

Nitrate was removed from wetland cells via denitrification. Denitrification was assumed to only occur in the anoxic (DO concentration < 2 mg/L) sediments of wetland cells containing emergent vegetation (USEPA 2000; Bastviken 2006). All denitrification was also assumed to result in the generation of nitrogen gas ($N_{2(g)}$), which was, in turn, assumed to leave the wetland system permanently. The general chemical expression for denitrification can be defined as follows (Sykes 2003):

$$2NO_3^- + 10e^- + 12H^+ \xrightarrow{\text{denitrification}} N_2 + 6H_2O \qquad (4\text{-}83)$$

Within the model, denitrification was simulated using volume-based first-order kinetics:

$$DENIT = V \cdot K_{DNT} \cdot \left[NO3 - NO3_o\right] \cdot \theta_{DNT}^{T_W - 20} \qquad (4\text{-}84)$$

where $NO3$ represents the NO_3^- concentration within a cell at a given time interval (mg-N/L), $NO3_o$ is the background NO_3^- concentration in the cell (mg-N/L), θ_{DNT} is the denitrification temperature correction factor (defaulted as 1.07 in HSPF), K_{DNT} is the first-order denitrification reaction rate (hr^{-1}), and *DENIT* is the resulting $N_{2(g)}$ load (mg-N) generated from denitrification.

4.4.2.3 Dissolved Oxygen Transfer

DO levels in each cell were estimated based on surface aeration, photosynthesis, nitrification, and respiration. The primary sources of oxygen in a wetland are photosynthesis and surface aeration. The primary sinks can vary depending on influent water characteristics, but generally include BOD (biological oxygen demand) and NOD (nitrogenous oxygen demand) degradation and associated microbial respiration, and general wetland respiration.

4.4.2.3.1 Wetland Primary Productivity

Photosynthesis rates into water are a function of available light, temperature, nutrients, water depth, and vegetation type. A sinusoidal curve was used to simulate both diurnal and seasonal photosynthesis variation (Chapra 1997):

$$PT = \begin{cases} PMAX \cdot \sin[\omega(h - h_{sr})] & \text{for } h_{sr} \le h \le h_{ss} \\ 0 & \text{otherwise} \end{cases} \tag{4-85}$$

where $PMAX$ represents the maximum possible photosynthesis rate (mg-O_2/m^2-hr), which occurs at noon each day; PT is the photosynthesis rate at time t (mg-O_2/m^2-hr), ω is the angular velocity (hr^{-1}), h_{ss} refers to the time of sunset (hr.) for a given day of the year (DOY), and h_{sr} refers to the time of sunrise (hr.) for a given DOY. The maximum photosynthesis rate $PMAX$ was estimated based on wetland literature values. The variable ω was further defined accordingly (Chapra 1997):

$$\omega = \frac{\pi}{fT_P} \tag{4-86}$$

$$f = \frac{h_{ST} - h_{SR}}{T_P} \tag{4-87}$$

where T_P represents the total daily time period (24 hr.), and f represents the ratio of sunlight hours (assumed to be hours between sunrise and sunset) to total hours (24 hrs) within a given day. In order to account for the effects of water temperature, additional temperature and solar radiation terms were also added to the photosynthesis expression (Chapra 1997):

$$PT = \begin{cases} PMAX \cdot \dfrac{SRI}{KSRI + SRI} \cdot \theta_{PT}^{TW-20} \sin[\omega(t - h_{SR})] & \text{for } h_{SR} \le \mathrm{t} \le h_{ST} \\ 0 & \text{otherwise} \end{cases} \tag{4-88}$$

where SRI is the solar radiation (MJ/m^2-d), $KSRI$ is the half saturation constant for solar radiation (MJ/m^2-d), and θ_{PT} represents the photosynthesis rate temperature correction factor, which was estimated to be 1.066 for general phytoplankton by Chapra (1997). The EPA treatment wetlands manual suggested an estimated net primary production of 4 g total biomass/m^2/d and 1g O_2/g net biomass is produced by photosynthesis in a wetland and that, on average, 2.5 g O_2 are produced per g-C of biomass produced within a wetland (USEPA 2000). Tian et al. (2010) found that an estimated 668 ± 5 g C/m^2-yr was produced on average by vegetation in the southern United States. Studies summarized by Mitsch and Gosselink (1993) also estimated primary productivity rates of 1000-3000 g-above ground biomass/m^2-yr. Given these literature values and the USEPA (2000) suggested conversion factors, annual photosynthesis rates with the units of g-O_2/m^2-yr were estimated for each source and are summarized in Table 4-8. From these annual photosynthesis rates a mean value of 1710 g-O_2/m^2-yr was calculated. A simple, linear relationship was found to relate this annual photosynthesis rate and $PMAX$:

$$PY = 1.88 \cdot PMAX \tag{4-89}$$

where PY is the annual wetland photosynthesis rate (g-O_2/m^2-yr) and $PMAX$ is the maximum hourly photosynthesis rate (mg-O_2/m^2-hr). A final value of 910 mg/m^2-hr for $PMAX$ could then be estimated from Equation 3-88 based on this annual rate.

Table 4-8 Shows the estimated annual photosynthesis rates from throughout the literature and corresponding calculated $PMAX$ values.

Source	**Photosynthesis Rate (g-O_2/m^2-yr)**	**Calculated $PMAX$ (mg-O_2/m^2-hr)**
USEPA (2000)	1460	777
Tian et al. (2010)	1670	888
Mitsch and Gosselink (1993)	2000	1063
Mean	1710	910

Once photosynthesis was determined, respiration was calculated as a fraction of PT. Chapra (1997) suggested general “rule of thumb” estimation of vegetation respiration:

$$RESP = 0.01 \cdot PT \tag{4-90}$$

where $RESP$ represents the corresponding net respiration of all vegetated biomass associated with PT (mg-O_2/m^2-hr). This respiration approximation excluded oxygen demand of BOD as well as other organisms within a wetland cell given the complexity that they added to the model. Therefore, in order to produce more accurate DO levels, future versions of the model should incorporate more complex respiration processes, especially those related to BOD degradation.

4.4.2.3.2 Surface Aeration

Surface aeration is the process by which oxygen from the atmosphere diffuses down into the surface layer of water. If water has a greater oxygen concentration with respect to its partial pressure in the atmosphere, diffusion can also occur in the upward direction, causing oxygen to bubble out of the water. However, if the opposite is true, diffusion occurs in the downward direction with oxygen from the atmosphere diffusing into the water. The general equation used to model surface aeration is (Kadlec and Knight 1996; Bicknell et al. 2001):

$$SURF = V \cdot K_{La} \cdot (DO_{\max} - DO_{i-1}) \cdot \Delta t \qquad (4\text{-}91)$$

where $DO_{\max}$ is the steady-state saturated DO concentration in a given cell (mg-O_2/L), DO is the DO load (mg-O_2/L) in a given cell and time interval, K_{La} is the mass transfer coefficient (1/hr.), V is the water volume (L) in the cell, and $SURF$ is the resulting addition of DO to a cell within a given time interval due to surface aeration (mg-O_2). Therefore, the difference between the actual and saturated concentrations of DO in a cell serve as the gradient that dictates the surface aeration rate.

DO can be determined through sampling and data collection while K_{La} and DO_{max} must be calculated based on environmental conditions. DO levels are very sensitive to water temperature and saturated DO levels were estimated using the following equation (Bicknell et al. 2001, Kadlec and Knight 1996):

$$DO_{\max} = 1000 \cdot LC^2 \cdot SS \cdot \left[14.652 - 0.41022 T_W + 0.007991 T_W{}^2 - 0.00007777 T_W{}^3\right] \qquad (4\text{-}92)$$

where T_W is the water temperature (°C). This equation will be used under the assumption that water temperature and DO levels are constant within a given cell. In reality, a DO gradient associated with water depth would exist. DO levels should be highest at the surface where surface aeration occurs in non-vegetated areas or areas with emergent vegetation. However, this gradient was assumed negligible given the shallow water depths generally found in constructed wetlands.

With DO_{max} and DO_i known, the K_{La} is the only unknown left. USEPA (2000) used the following equation to estimate the K_{La} in constructed wetlands:

$$K_{La} = \left(\frac{\sqrt{D_L \cdot v_i}}{SS^{3/2}} \right) \cdot \theta_{DO}^{(T_W - 20)} \tag{4-93}$$

where v_i is the horizontal flow velocity (m/hr.), D_L is the oxygen molecular diffusion constant (m^2/hr.), θ_{DO} is the dissolved oxygen temperature correction factor, and *SS* is the water depth (m). Similar equations were also referenced by Gonenc and Wolfin (2005), Kadlec and Knight (1996), Bicknell et al. (2001). A value of 7.33×10^{-6} m^2/hr. for D_L was suggested by Kadlec and Knight (1996) for a water temperature of 20°C. Bicknell et al. (2001), Kadlec and Knight (1996), and USEPA (2000) assumed a typical wetland flow velocity of around1.25 m/hr. Flow velocities simulated by the hydrologic module of were used to determine final surface aeration values in the current project. No temperature correction factor was included in surface aeration calculations due to the high uncertainty associated with the extrapolation over a temperature range (Kadlec and Knight 1996).

4.4.2.4 Influent water quality concentrations and loads

The water quality portion of the model simulates the dynamics of ammonium (NH_4^+), nitrate (NO_3^-), total suspended solids (TSS), and dissolved oxygen (DO) within the wetland. Influent pollutant concentrations and loads vary depending on the type of water being treated. Stormwater pollutant concentrations, for example, were found in the literature to be dependent on runoff flowrates (Lee and Bang 2000; Sansalone and Cristina 2004). Pollutant concentrations are also often much higher during the initial portion of a storm event; this phenomenon is referred to as the first flush. However, due to the lack of knowledge surrounding the relationship between pollutant concentrations and flowrate, the current study assumed influent stormwater concentrations to be constant. Therefore, the definition of pollutographs corresponding to influent wetland hydrographs would be useful in the future development of the model as this work was out of the scope of the current study.

Conversely, wastewater pollutant concentrations are not related in the same way to flowrates. Data provided by a local wastewater treatment plant did not show significant annual trends in influent NH_4^+, BOD, and TSS daily grab sample concentrations nor any significant trends relating the plant flowrate and influent pollutant concentrations. Insufficient data have been found to affirm or negate any trends between water quality and flowrate. Additionally, flowrate and pollutant concentration relationships vary from site to site, and therefore, are difficult to predict for any other site or condition.

The wetland model developed herein simulates influent water from three sources, (1) stormwater (urban, residential, agricultural, etc.), (2) municipal wastewater, and (3) agricultural (mainly swine) wastewater. In the current study influent pollutant concentrations were assumed to be constant user inputs for all water input types. However, in future versions of the model, more complexity could be added to influent pollutant concentration characteristics as relevant data become available.

Chapter 5: Model Calibration

The model developed in the current study was calibrated and verified with data provided by Dr. Thomas Jordan from the Smithsonian Environmental Research Center (SERC) for a restored wetland (Barnstable 1) receiving agricultural runoff in Kent Island, MD. The following chapter describes in detail all model inputs, data manipulation methods, calibration procedures, and results. The object of this chapter was to demonstrate that the model could be calibrated with real data and produce reliable water quantity and quality results.

5.1 BARNSTABLE 1 WETLAND DATA

The wetland was referred to as Barnstable 1 (Jordan et al. 2003; Kalin et al. 2012). Water quality and quantity data were collected over the two-year period from May 8, 1995, through May 12, 1997. The restored wetland, which has an area of 1.3 ha (3.2 acres), treats water from a drainage area of 14 ha (35 ac) that is comprised of 18% forest and 82% cropland (covered by corn during 1995 and 1997 and by soybeans during 1996). The average watershed slope is 1%, and the soil is comprised of two layers. The top layer is silt loam with moderately low to low permeability and was 0.2 m (0.656 ft) deep. The bottom layer extended down from 0.2 to 1 m (0.656 - 3.28 ft) below the surface and was comprised of silty clay loam containing 18-30% clay. Drainage ditches and channels transport runoff from the drainage area to the wetland inflow point.

The restored wetland had previously been drained to serve as cropland. However, in 1986, the wetland was restored by excavating an area to a uniform depth of 1 m (3.28 ft) over the 1.3 ha (3.2-ac) area of the wetland. Berms were also constructed to maintain water levels at an average depth of 0.2 m (0.656 ft). Elevation contours of the wetland with a scale of 10-cm were measured by a Total Station CTS-2/2B (Topcon, Tokyo, Japan). The resulting elevation map is shown in Figure 5-1. Emergent vegetation was reported to cover an average of 80% of the total wetland surface area during the growing season but only 15% during the non-growing season. Table 5-1 summarizes all relevant wetland characteristics. Jordan et al. (2003) provided complete details.

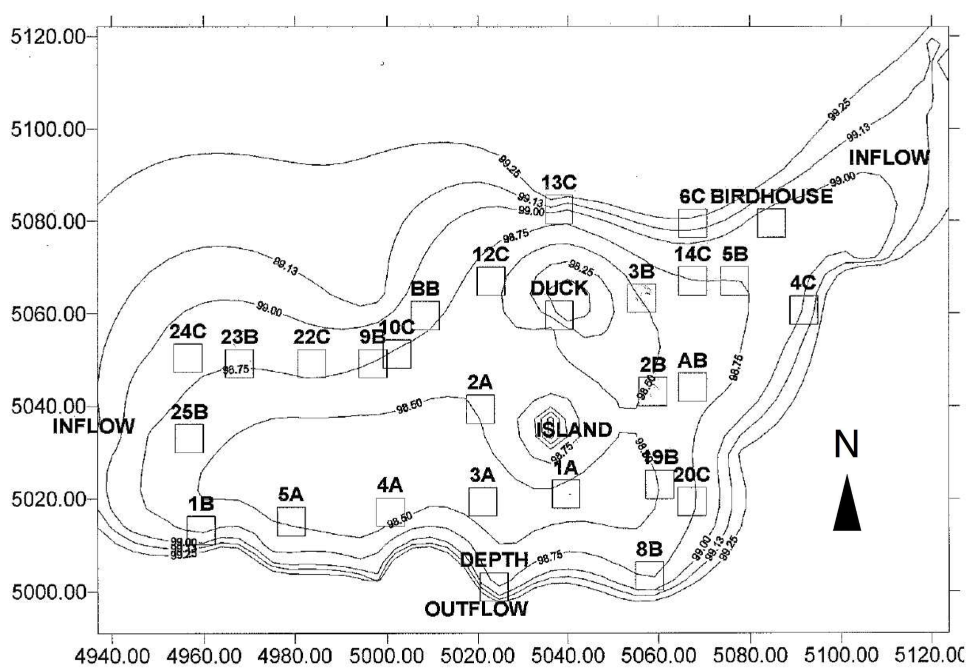

Figure 5-1 Barnstable 1 wetland topographic map with all elevations in meters. All boxes represent areas from which vegetation was samples. The labels A, B, and C respectively represent deep zones with submerged vegetation, intermediate depths, and very shallow (often dry) upper edges of the wetland. Most all runoff entered the wetland through the inflow labeled in the top right corner (Jordan 2013).

Table 5-1 Watershed and wetland specifications for the restored wetland situated in Kent Island, MD, as reported by or estimated from Jordan et al. (2003).

Parameter	**Value**
Drainage area (acres)	35
Land uses	18% forest 82% cropland (corn/soybean)
Drainage area hydrologic soil type	C
Soil type	Top 0.656 ft silt loam Bottom 0.656-3.3 ft silty clay loam
Estimated hydraulic conductivity (ft/d)	0.00283
Drainage area slope (%)	1
Wetland area (acres)	3.2
Wetland mean depth (ft)	0.656
Emergent vegetation cover	80% during growing season 15% during non-growing season

5.1.1 Hydrologic data

Intermittent inflow (combined term of runoff inflow and direct precipitation over wetland) and outflow rates were reported by Jordan et al. (2003). Outflow rates were calculated via recorded depths through a 120°V-Notch weir and C10 data logger (Campbell Scientific, Logan, Utah) located at the outlet. Observed intermittent outflow rates are plotted in Figure 5-2. The term combined inflow data was used in the current study to include both runoff from the contributing drainage area and precipitation that fell directly onto the wetland. Jordan et al. (2003) calculated these combined inflow rates by adding the corresponding outflow rate and change in volume for a given time interval in the wetland:

$$(IN + P)_i = OUT_i + \frac{\Delta S_i}{\Delta t_i} \tag{5-1}$$

where $(IN+P)_i$ represents the combined inflow rate (cfs) over a given time interval *i*, OUT_i is the intermittent outflow rate (cfs) at time t_i (s), and $\frac{\Delta S_i}{\Delta t_i}$ is the rate of change in storage (cfs) within the wetland over the time interval *i*. The combined inflow rates provided by Jordan (2013) are plotted in Figure 5-3.

Flowrates were only measured when a detectable change in wetland water depth was detected rather than at a regular time interval. During large flow events, flow could be recorded up to every 15 minutes. However, if depth changes were not detected by the depth sensors, flow was not recorded. Wetland ET was estimated from pan evaporation measured at the Smithsonian Environmental Research Center (SERC), which was 25 km from the wetland site. Infiltration was assumed to be negligible given that the wetland was underlain with a clay layer of 0.5 m (1.64 ft). Additionally, Jordan et al. (2003) reported that clay samples taken below the wetland bottom were dry. However, Jordan et al. (2003) did report that errors in calculated annual water balances at the site may have been due to the exclusion of infiltration and dam seepage in their calculations. Given this information, the current study assumed infiltration occurred at the site at a very slow rate due to the low permeability of the underlying clay layer.

Daily rainfall data were measured by a Universal Rain Gage Model 5-780 (Belfort Instrument, Baltimore, MD) located at the Wye Research and Education Center (WREC), which is 13 km west of the wetland. These rainfall data were used by Jordan et al. (2003) to estimate the weekly rainfall volumes over the wetland as well as the total annual runoff into the wetland (see Figure 5-4) by subtracting it from the corresponding combined inflow data. Jordan et al. (2003) provided more in-depth information on instrumentation.

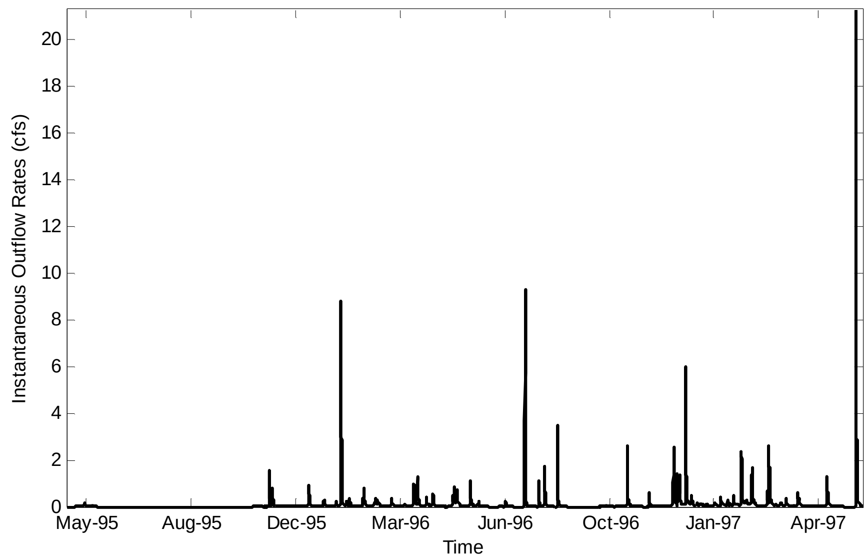

Figure 5-2 Intermittent outflow rates (cfs) from Barnstable 1 wetland over the study period as plotted from data provided by Jordan (2013).

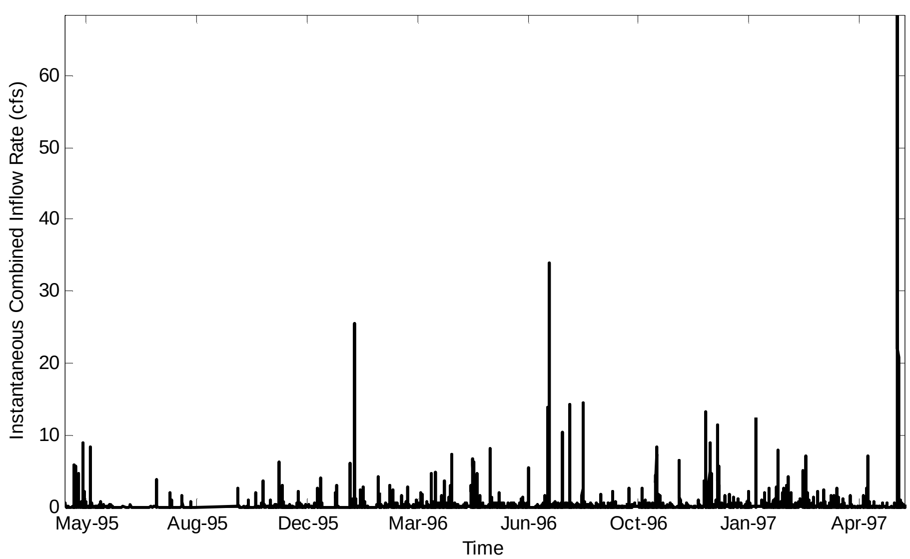

Figure 5-3 Intermittent combined (runoff inflow + direct rainfall) inflow (cfs) to wetland over the study period as plotted by data provided by Jordan (2013).

Weekly combined inflow (runoff inflow + direct rainfall), direct rainfall onto the wetland, and outflow volumes were also made available by Jordan (2013). Direct rainfall volumes were, again, estimated by Jordan et al. (2003) from daily rainfall measured at the Wye Research and Education Center (WREC) gage. All three weekly data series are shown in Figure 5-4. These weekly volumes were later used to estimate weekly influent and effluent pollutant loads.

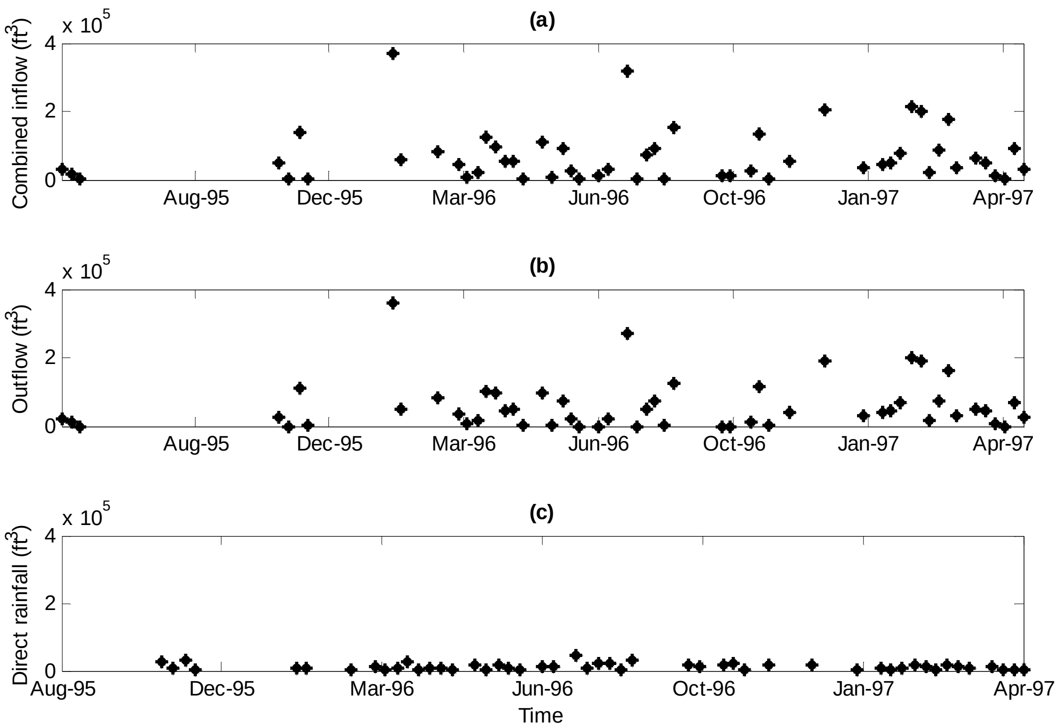

Figure 5-4 Weekly volumes with units of ft^3 for combined inflow (a), outflow (b), and direct rainfall (c) over the wetland.

In addition to inflow and outflow data, Jordan (2013) also provided intermittent water depths in the outlet weir (see Figure 5-5), as well as corresponding intermittent wetland storage volumes within the wetland (see Figure 5-6). Water depths in the weir were assumed to be zero when the wetland was at capacity (88,219 ft^3) and outflow did not occur. A negative depth implied that the water level was below the invert of the outlet weir, while a positive value implied that the water level was above the weir invert. Data provided by Jordan (2013) were also used to determine the weir invert elevation of 98.901 m (324 ft) as shown in Figure 5-7, suggesting that a depth reading of -1.32 ft was indicative of a dry wetland. Wetland storage volumes were computed by Jordan et al. (2003) with the corresponding water depths at the outlet weir and the wetland topography as measured by the Total Station CTS-2/2B.

It was found that the Jordan et al. (2003) reported water depth in the weir went below the bottom elevation of the outlet cell during the initial dry period from May through October of 1995. As shown in Figure 5-5, water depths in the weir should not go below -1.32 ft; however, the minimum recorded water depth at the weir was -1.59 ft. These negative depths reported by Jordan (2013) may have been a result of the depth gage sensitivity at low depths or due to a bias in the gage calibration.

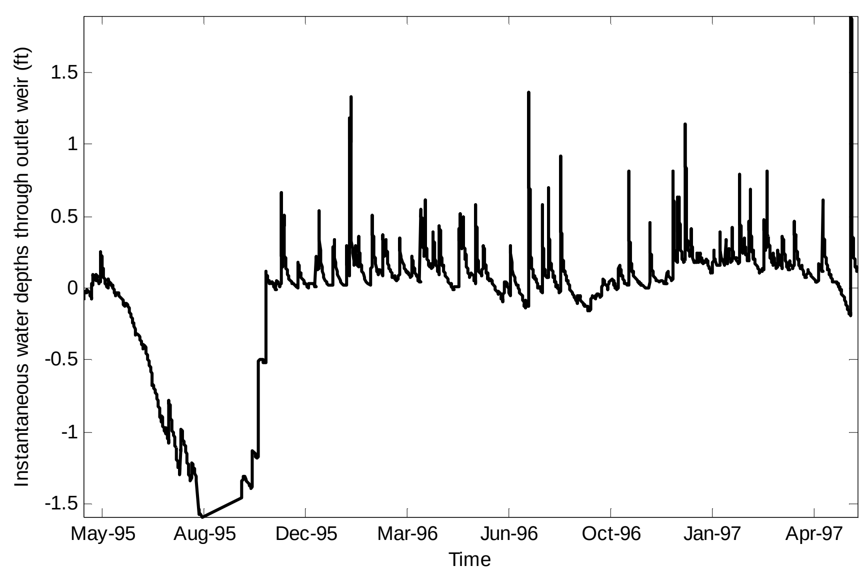

Figure 5-5 Observed water depths in the weir (ft) over the 2-yr study period from 1995 to 1997 as reported by Jordan et al. (2003).

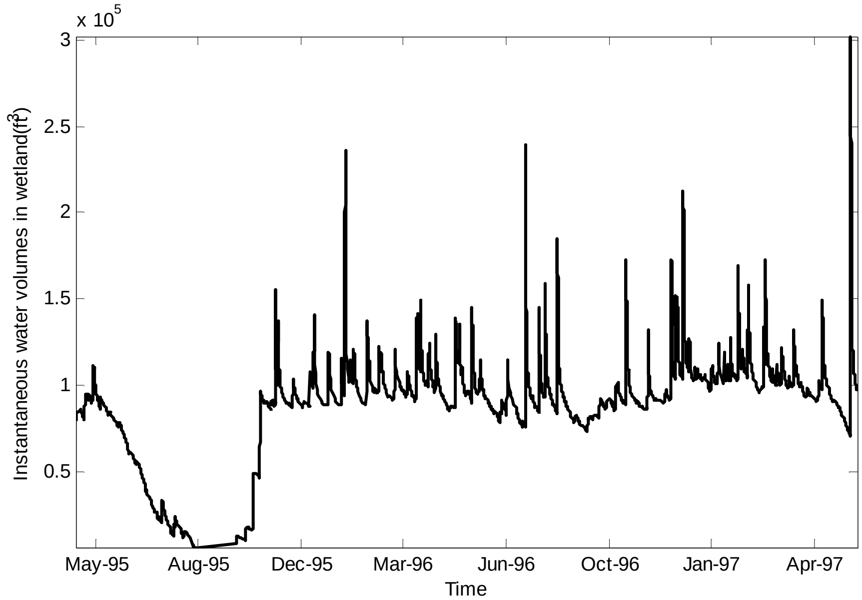

Figure 5-6 Intermittent water volume storages as estimated and reported by Jordan (2013) based on corresponding outlet weir water depths and wetland elevations.

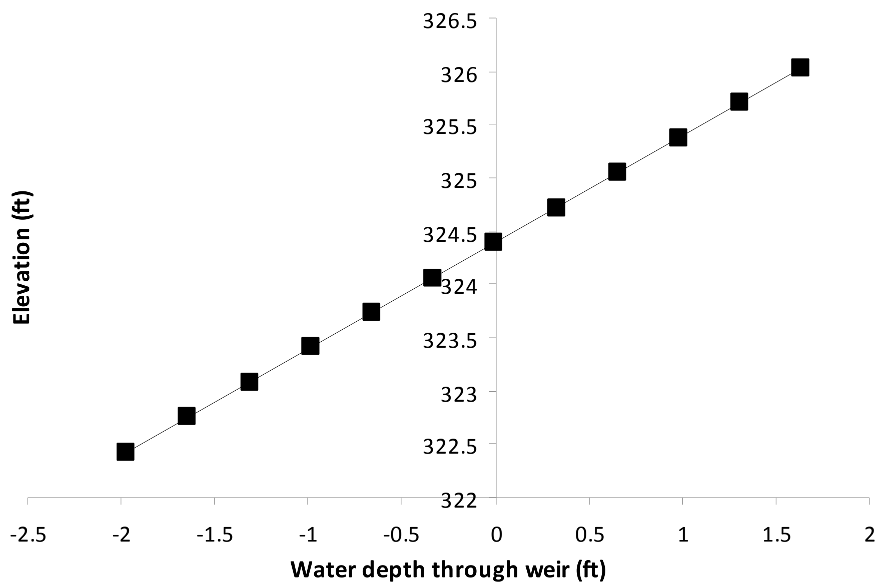

Figure 5-7 Observed relationship between water depth flowing the outlet weir and elevation at the Barnstable 1 site based on data provided by Jordan (2013).

5.1.2 Water quality data

Mean weekly concentrations for TKN, TPO_4^{-3}, NH_4^+, NO_3^- (defined as the sum of all nitrite and nitrate species present), TOC, and TSS were computed from composite weekly samples collected at the inflow and outflow points of the wetland. Composite samples were volume-weighted, collecting inflow or outflow volumes proportional to the corresponding flowrate. Larger flowrates, for example, resulted in larger corresponding collected volumes for a given time increment. Samples were only collected when a certain threshold volume of inflow or outflow passed the pump. Therefore, water samples could be collected up to every 15-min during large storm events or not for days or weeks during drought periods.

Jordan et al. (2003) also calculated the total nitrogen (TN) levels by adding TKN and NO_3^-, as well as the total organic nitrogen (TON) by subtracting TKN from the calculated total organic nitrogen (TON) values. Concentrations for all constituents in the rainfall were also estimated from another study performed by Jordan et al. (1995) and reported on a weekly basis by Jordan (2013). The current study was only concerned with TSS, NH_4^+, and NO_3^-. Figure 5-8 and Figure 5-9, respectively, plot the weekly composite influent and effluent concentrations for TSS, NH_4^+, and NO_3^-. Dissolved oxygen (DO) levels were not recorded by Jordan et al. (2003). Concentrations of pollutants in the rainfall were also not used in the current study, as the developed model structure did not include input rainfall water quality characterization. This exclusion of rainfall water quality was assumed to be reasonable due to the scarcity of rainfall water quality data in the literature as well as the small relative volume of direct rainfall volume to that of runoff inflow volume (see Figure 5-4). Additionally, TSS concentrations were equal to zero for all weeks of record. NH_4^+ and NO_3^- recorded weekly concentrations are plotted in Figure 5-10. For purposes of the current study, influent DO levels were assumed to equal saturated DO levels based on personal communication with Jordan (2013). Given that most wetland inflow entered the wetland via irrigation ditches, inflow velocities should promote aeration. Additionally, direct rainfall should have high dissolved oxygen concentrations given the high surface area of raindrops and their prolonged contact with the atmosphere as they fall. Jordan (2013) also predicted that background DO levels were near or at saturation given the vegetation present in the wetland.

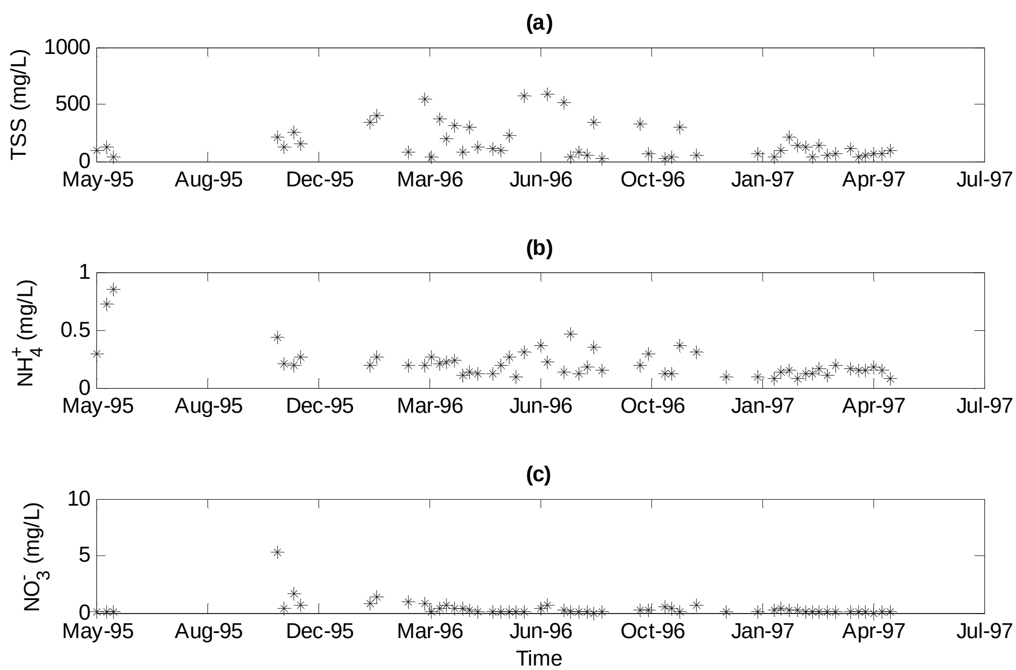

Figure 5-8 Weekly composite influent weekly concentrations (mg/L) of TSS (a), NH_4^+ (b), and NO_3^- (c) as plotted with data provided from Jordan (2013).

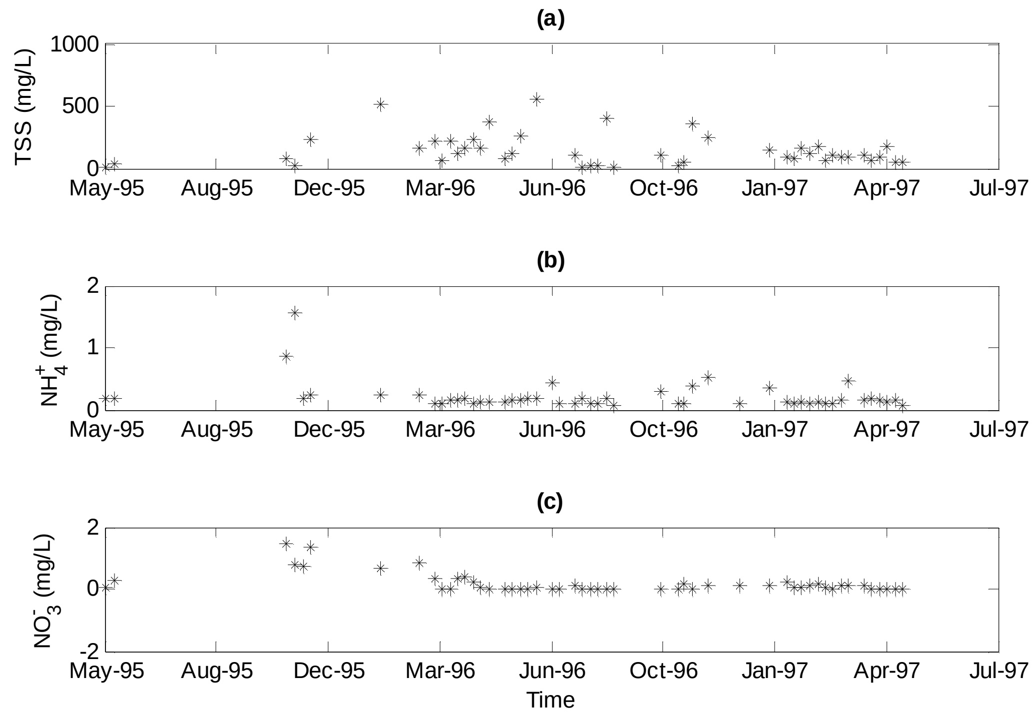

Figure 5-9 Weekly composite effluent weekly concentrations (mg/L) of TSS (a), NH_4^+ (b), and NO_3^- (c) as plotted with data provided from Jordan (2013).

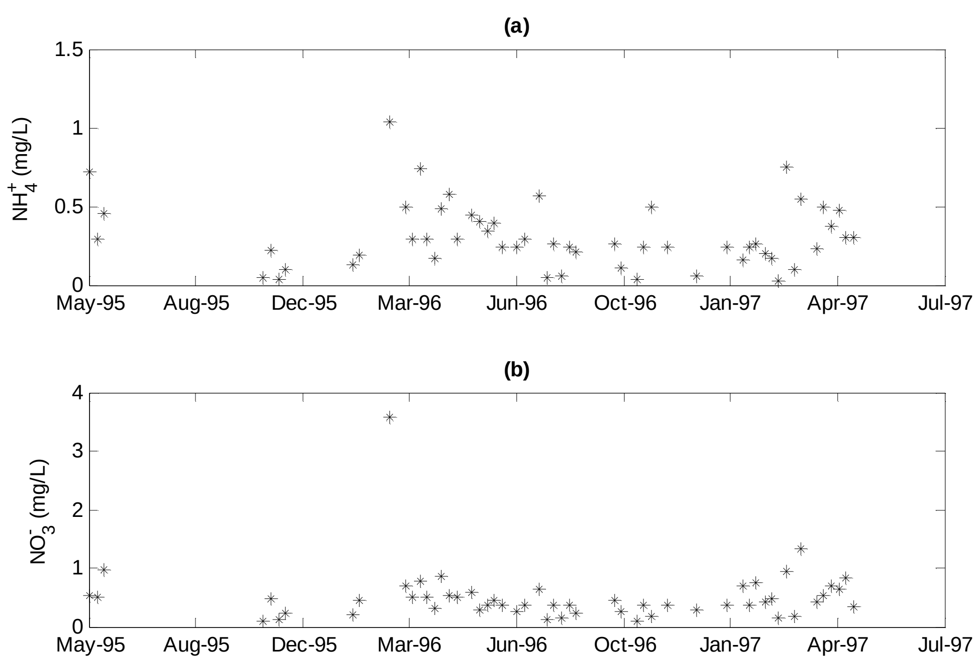

Figure 5-10 Weekly recorded rainfall concentrations (mg/L) NH_4^+ (a) and NO_3^- (b) as plotted with data provided from Jordan (2013).

5.1.3 Barnstable 1 layout and topography

The topographic map shown in Figure 5-1 defined Barnstable 1 wetland elevations, vegetation type distributions, and included wildlife structures (i.e., habitat island and duck shelter). Due to the algorithms used in the computer, generation of the elevations shown in Figure 5-1 resulted in a number of the wetland edges that appear wavy, which, in reality, represent straight topographic lines (Jordan 2013). The southern edge of the wetland, for example, represents a straight berm despite its wavy appearance in Figure 5-1. The boxed areas represent plots for vegetation sampling. Boxes labeled with A represent deep wetland areas where only submerged vegetation grew. Boxes labeled with B indicate areas of intermediate water depth, and boxes labeled with C were very shallow, often dry upper edges of the wetland (Jordan 2013). The areas labeled with B and C were both assumed to support emergent vegetation.

5.1.4 Characterizing error in the Barnstable 1 data

In order to fully understand the Barnstable 1 wetland data, it was necessary to estimate the error associated with all observed values (e.g., effluent rates, weekly TSS concentrations, etc.) that were used in model calibration. The current study defined error as the discrepancy existing between the presumed true values and corresponding measured or computed values. While the errors of direct measurements such as the stage depth at the outlet weir $D_{S(i)}$ were easy to define based on instrumentation error, other data were much more difficult to define as their accuracy and precision were dependent on a number of factors.

Error propagation was used to estimate the total error associated with each individual data point (i.e., 1-min outflow rates, weekly TSS concentration, etc.) consisting of more than one error source (Topping 1972):

$$E_P = \sqrt{E_1^2 + E_2^2 + E_3^2 + \cdots + E_n^2} \tag{5-2}$$

where E_P is the resulting total error (± %) in a given data value, *n* is the total number of error sources, and $E_1, E_2, E_3, \cdots E_n$ represent all the sources of error (± %) within the data value. The measured and computed data provided by Jordan (2013) used in the current study included the following directly measured data:

1. Intermittent stage-depth measurements ($D_{S(i)}$) at the outlet weir (ft).
2. Weekly composite TSS influent and effluent concentrations (mg/L).
3. Weekly composite NH_4^+ influent and effluent concentrations (mg/L).
4. Weekly composite NO_3^- influent and effluent concentrations (mg/L).

Jordan et al. (2003) also used the direct $D_{S(i)}$ measurements to compute corresponding inflow and outflow values:

1. Intermittent outflow rates (OUT_i) with units of (cfs).
2. Intermittent storage volumes (S_i) in wetland (ft^3).
3. Storage change rates ($\Delta S_i / \Delta t_i$) over each time interval (cfs).
4. Intermittent combined inflow rates ($IN_i + P_i$) to wetland (runoff inflow + direct precipitation) (cfs).
5. Weekly effluent volumes (ft^3).
6. Estimated weekly combined inflow volumes (ft^3).

7. Weekly direct rainfall volumes as estimated by daily rainfall at WREC rain gage (ft^3).

5.1.4.1 Intermittent depth measurements

The intermittent depth measurements at the outlet weir ($D_{S(i)}$) were the only direct hydrologic measurements taken at the Barnstable 1 wetland. These measurements were taken by a depth sensor, which was cited to deliver measurements with an uncertainty of ± 2% (Harmel et al. 2006). All other data provided by Jordan (2013) were computed with the use of these $D_{S(i)}$ measurements, and therefore all contain an inherent error of at least ± 2%.

5.1.4.2 Outflow intermittent rates and weekly volumes

Intermittent outflow rates were calculated from the corresponding $D_{S(i)}$ values through use of a stage-discharge relationship (i.e., a weir equation for the 120° V-notch outlet weir). Harmel et al. (2006) estimated errors introduced by friction in weir flow were estimated to be ± 5-10%. For the current study, an average uncertainty of ±7.5% was chosen to represent weir measurement error. The resulting propagated error in all OUT_i values, both intermittent rates and weekly volumes, was, therefore, estimated to equal ±7.76%.

5.1.4.3 Storage change rates

Wetland storage volumes S_i were computed by Jordan et al. (2003) through use of the elevation measurements (see resulting topographic map in Figure 5-1) taken by a Total Station CTS-2/2B and corresponding wetland depth $D_{S(i)}$ measurements. The current study estimated a ±10% error to be associated with these elevation measurements. An additional error of ± 2% was also inherent in storage measurements due to the corresponding $D_{S(i)}$ values from which they were computed. Given these two sources of error, the total error associated with each S_i value was computed to be ±10.3% using Equation 5-2. Corresponding $\Delta S_i / \Delta t_i$ values were calculated accordingly:

$$\frac{\Delta S_i}{\Delta t_i} = \frac{S_{i+1} - S_i}{t_{i+1} - t_i} \tag{5-3}$$

where *i* represents the time interval, *t* is the time in seconds within a given time interval. Error associated with the measurement of time *t* was assumed to be negligible. Therefore, the error in $\Delta S_i / \Delta t_i$ was equal to that of S_i, which was estimated to equal ±10.3%.

5.1.4.4 Combined intermittent rates and weekly volumes

Intermittent combined inflow rates ($IN_i + P_i$) were computed by Jordan et al. (2003) accordingly:

$$(IN + P)_i = OUT_i + \frac{\Delta S_i}{\Delta t_i} \tag{5-4}$$

In addition to the errors associated with OUT_i (±7.76%) and $\Delta S_i / \Delta t_i$ (±10.3%), $(IN + P)_i$ values also had an error term that was generated from negative values that resulted from the addition of OUT_i and $\Delta S_i / \Delta t_i$ for some time intervals. Physically, these negative rates implied that for a given time interval *i*, Equation 5-4 did not fully characterize the water balance of the wetland (i.e., ET, infiltration, or other losses were not accounted for). Because these negative values accounted for only 0.2% of the total combined inflow volume over the 2-year study period, Jordan et al. (2003)

assumed them to be negligible, setting them to zero. For error characterization purposes, this ±0.2% error was propagated along with the OUT_i (±7.76%) and $\Delta S_i / \Delta t_i$ (±10.3%) errors to compute a final error of ±12.9% for all $(IN + P)_i$ values. Weekly combined inflow volumes also were assumed to have the same inherent error of ±12.9%.

5.1.4.5 Weekly direct rainfall volumes

Weekly rainfall volumes were summed from daily depths recorded by a Universal Rain Gage Model 5-780 located at WREC. Inherent in these measurements were the errors due to instrumentation limits and the errors associated with differences in weekly rainfall patterns of the Barnstable 1 site and WREC, which is 13 km east of the wetland. The area of the wetland (3.2 acres), which was used by Jordan et al. (2003) to convert WREC-recorded rainfall depths to volumes over the wetland, was assumed to have negligible error. Belfort Instrument, the manufacturer of the WREC rain gage, defines an accuracy for the gage of ±0.5% of "full scale." Full scale refers to the holding capacity depth of the gage, which was 12 in. for the 5-780 model (<http://belfortinstrument.com/products/universal-rain-gauge/>). Therefore, rain gage measurements were recorded within an accuracy of ±0.06 in. Over the 2-year study period from May 1995 through May 1997, a mean daily rainfall depth of 0.377 in. (excluding days with no rainfall) was recorded by the WREC rain gage. The ±0.06 in. accuracy represents ±15.9% of this mean daily depth. This error of ±15.9% was assumed to equal the instrumentation error inherent in the weekly rainfall volumes over the wetland.

In order to estimate the error introduced in weekly rainfall volumes due to the distance between the rain gage at the Barnstable 1 wetland, the Quadrant method was used to estimate annual rainfall depths at the wetland based on precipitation data recorded at the three rain gages closest to the wetland. These gauges were located in Chestertown, MD (35.2 km north of the wetland), in Baltimore, MD (44.3 km northeast of the wetland), and at WREC (13km east of the wetland). Precipitation data for Baltimore, MD, and Chestertown, MD, were obtained at a monthly time-scale by NOAA. Rainfall from the three sites was accumulated according to the study years defined by Jordan et al. (2003) where year 1 spanned from May 3, 1995 through April 30, 1996 and year 2 was defined to range from May 1, 1996 to May 1, 1997. Resulting annual precipitation at each of the three sites and resulting estimated annual precipitation at the Barnstable 1 wetland are reported in Table 5-2. The error for years 1 and 2 between the annual precipitation depths reported at WREC and estimated at the wetland were, respectively, calculated accordingly:

$$E_{P(1)} = 100 \times \left(\frac{42.1 - 43.3}{43.3} \right) = -2.77\% \tag{5-5}$$

$$E_{P(2)} = 100 \times \left(\frac{49.1 - 51.0}{51.0} \right) = -3.73\% \tag{5-6}$$

where $E_{P(1)}$ and $E_{P(2)}$ are the resulting errors (%) in annual WREC precipitation depths for years 1 and 2 relative to estimated annual depth at the wetland site. These results suggest that, if the Quadrant method produced reasonable annual precipitation estimations at the wetland site, the WREC rain gage observed less rainfall than the Barnstable 1 wetland. $E_{P(1)}$ and $E_{P(2)}$ were averaged to produce an overall gage location error of -3.25%. Given that this error reflects annual variation rather than weekly variation between gage locations, the actual weekly error could be higher. However, given the available data, this error value of -3.25% was assumed to be sufficient for characterizing the error resulting from estimating weekly rainfall from an offsite rain gage. Error propagation was used to estimate a total rainfall error term of ±16.2%. Weekly inflow volumes were estimated in the current study by subtracting the weekly rainfall volumes from the corresponding weekly combined inflow volumes. Therefore, the resulting weekly inflow volume error was propagated from the errors associated with the weekly combined inflow (±12.9%) and rainfall (±16.2%) volumes and had a value of ±20.7%.

Table 5-2 Reported annual precipitation depths at three rain gages surrounding the Barnstable 1 wetland and resulting estimated annual precipitation depths at the site.

	Annual Precipitation		
	Year 1 (in.)	**Year 2 (in.)**	**Quadrant Method weights**
Baltimore	44.6	55.0	0.0704
Chestertown	51.5	62.4	0.112
WREC	41.6	49.1	0.818
Wetland Site (estimated)	43.3	51.0	---

5.1.4.6 Water quality measurements

Errors associated with weekly composite TSS, NH_4^+, and NO_3^- concentrations were estimated based on values given by Harmel et al. (2006) and the associated weekly volume error. Weekly influent concentrations for all water quality constituents had an error ±20.7% that was due to uncertainty in the associated estimated weekly inflow volumes, as computed in the previous paragraph. Similarly, all observed weekly effluent water concentrations had an inherent error of ±7.76% due to weekly outflow volume error.

Measurement errors for influent and effluent TSS, NH_4^+, and NO_3^- were assumed to be the same and were estimated from errors computed by Harmel et al. (2006) for water quality measurements. The study done by Harmel et al. (2006) assumed that water quality measurements had four sources of inherent error, (1) flow measurement error, which was accounted for in the current study by the weekly influent and effluent volume errors; (2) error in sample collection (i.e., errors associated with using volume-weighted composite sample as well as low-flow thresholds); (3) error associated with sample preservation/storage (Jordan et al. (2003) used acidification to preserve NH_4^+, and NO_3^-); and (4) error in laboratory analysis of samples (Standard Methods followed for all analyses). Sample collection also had two sources of error, one of which was from the volume-weighted method used to collect composite samples at the inlet and outlet of the site and the second of which due to the use of a low flow threshold below which samples were not taken. All errors associated with weekly influent and effluent water quality concentrations are summarized in Table 5-3. Using error propagation, influent TSS, NH_4^+, and NO_3^- errors were respectively found to equal ±23.6, 25.1, and 21.0%. Effluent TSS, NH_4^+, and NO_3^- errors were similarly estimated to equal ±13.5, 16.1, and 13.9%. All errors associated with weekly water quality data are shown in Table 5-3. Final errors associated with all data used in model calibration are also summarized in Table 5-4.

Table 5-3 Estimated error terms associated with weekly influent and effluent concentrations of TSS, NH_4^+, and NO_3^-.

Error Source	**TSS**	**NH_4^+**	**NO_3^-**
Weekly inflow volume error (%)	± 20.7	± 20.7	± 20.7
Weekly outflow volume error (%)	±7.76	±7.76	±7.76
Flow/volume-weighted sampling error (%)	±11	±11	±11
Low-flow threshold error (%)	±3	±3	±3
Sample preservation error (%)	---	-8%	-1%
Laboratory analysis error (%)	-0.85	+2.5	-1.5
Total influent error (%)	±23.6	±25.1	±21.0
Total effluent error (%)	±13.5	±16.1	±13.9

Table 5-4 Associated errors for all data and data-derived inputs provided by Jordan (2013) used to calibrate the model.

Data Array	**Total Error (%)**
Intermittent stage depth measurements ($D_{S(i)}$) at outlet weir (ft)	±2
Intermittent outflow rates (OUT_i) with units of (cfs)	±7.76
Intermittent storage volumes (S_i) in wetland (ft^3)	±10.3
Storage change rates ($\Delta S_i / \Delta t_i$) over each time interval (cfs)	±10.3
Intermittent combined inflow rates ($IN_i + P_i$) to wetland (cfs)	±12.9
Weekly effluent volumes (ft^3)	±7.76
Estimated weekly combined inflow volumes (ft^3)	±12.9
Weekly direct rainfall volumes as estimated by daily rainfall at WREC rain gage (ft^3).	±16.2
Computed weekly runoff inflow volumes (ft^3)	±20.7
Weekly composite TSS influent concentrations (mg/L)	±23.6
Weekly composite TSS effluent concentrations (mg/L)	±13.5
Weekly composite NH_4^+ influent concentrations (mg/L)	±25.1
Weekly composite NH_4^+ effluent concentrations (mg/L)	±16.1
Weekly composite NO_3^- influent concentrations (mg/L)	±21.0
Weekly composite NO_3^- effluent concentrations (mg/L)	±13.9

5.1.5 Conversion of intermittent to 1-min time intervals

The intermittent combined inflow (runoff inflow + direct rainfall) (cfs), recorded outflow (cfs), outlet weir water depth (ft), and wetland storage volume (ft^3) values were reformatted into 1-minute time intervals in the current study for use with the model, which runs on a 1-min time step. The initial intermittent combined inflow and outflow rates were reported by Jordan (2013) at sporadic time intervals (ranging from 15-min to 39 days). Linear interpolation was used to interpolate between consecutive outflow, outlet weir depth, and wetland storage volume intermittent data points and to estimate corresponding flowrates, depths, and storage volumes on a 1-minute basis. Additionally, resulting 1-min combined inflow and outflow rates (cfs) were converted to 1-min volumes in ft^3 by multiplying each 1-min flowrate by 60 s/min and by the time interval of 1 min.

5.1.6 Separation of inflow and rainfall

To calibrate the model using a 1-min time increment, it was necessary to separate the 1-min combined inflow volumes (ft^3) into two 1-min input vectors, (1) the runoff via the main wetland inlet and (2) direct rainfall onto the wetland. A method referred to as the ratio method was developed to achieve this goal. With this method, the ratio of the computed annual on-site runoff inflow depth to the depth of the annual combined inflow provided by Jordan (2013) was computed for both years of record where year 1 spanned from May 3, 1995, through April 30, 1996, and year 2 was defined to range from May 1, 1996, to May 1, 1997. These two ratios were then multiplied by each 1-min combined inflow volume (ft^3) within the corresponding year of record in order to estimate the corresponding 1-min runoff inflow volumes (ft^3).

Lastly, 1-min direct rainfall volumes (ft^3) over the wetland were computed by subtracting the resulting 1-min runoff inflow volumes from the corresponding 1-min combined inflow volumes.

The first step of the ratio method estimated annual direct rainfall volumes over the Barnstable 1 wetland. Corresponding annual rainfall depths at the Barnstable 1 site were previously estimated to equal 43.3 and 51.0 in. (see Table 5-2). The total associated direct rainfall volume over the wetland for each year of record was computed accordingly (the Barnstable 1 wetland has an area of 13,000 m^2):

$$P_1 = 43.3\,\text{in.} \times \frac{1\,\text{ft}}{12\,\text{in.}} \times 13{,}000\,\text{m}^2 \times \frac{(3.28\,\text{ft})^2}{\text{m}^2} = 504{,}659\,\text{ft}^3 \tag{5-7}$$

$$P_2 = 51.0\,\text{in.} \times \frac{1\,\text{ft}}{12\,\text{in.}} \times 13{,}000\,\text{m}^2 \times \frac{(3.28\,\text{ft})^2}{\text{m}^2} = 594{,}402\,\text{ft}^3 \tag{5-8}$$

where P_1 and P_2 were the respective annual direct rainfall volumes (ft^3) over the Barnstable 1 wetland for years 1 and 2. Once P_1 and P_2 were computed, they were subtracted from the corresponding annual combined inflow volumes ($(IN + P)_1$ and $(IN + P)_2$) reported by Jordan (2013) for the Barnstable 1 site in order to determine the annual runoff inflow volumes (IN_1 and IN_2). These annual combined inflow volumes were computed by summing all 1-min combined influent volumes (see Section 5.1.5) for each year of record. $(IN + P)_1$ was equal to 1.77 x 10^6 ft^3 and $(IN + P)_2$ was equal to 3.81 x 10^6 ft^3. Based on these values, the following IN_1 and IN_2 were computed:

$$IN_1 = 1.77 \times 10^6\,\text{ft}^3 - 504{,}659 = 1{,}265{,}341\,\text{ft}^3 \tag{5-9}$$

$$IN_2 = 3.81\times10^6\ \text{ft}^3 - 594{,}402 = 3{,}215{,}598\,\text{ft}^3 \tag{5-10}$$

These annual inflow volumes were then converted into depths over the contributing drainage area (140,000 m^2):

$$IN_1 = \frac{1{,}265{,}341\,\text{ft}^3}{140{,}000\,\text{m}^2}\times\frac{1\,\text{m}^2}{(3.28\,\text{ft})^2}\times\frac{12\,\text{in.}}{1\,\text{ft}} = 10.1\text{in.} \tag{5-11}$$

$$IN_2 = \frac{3{,}215{,}598\,\text{ft}^3}{140{,}000\,\text{m}^2}\times\frac{1\,\text{m}^2}{(3.28\,\text{ft})^2}\times\frac{12\,\text{in.}}{1\,\text{ft}} = 25.6\,\text{in.} \tag{5-12}$$

It was noted that while the rainfall depths for years 1 and 2 were comparable (43.3 and 51.0 in.), IN_2 (25.6 in.) was more than double IN_1 (10.1 in.). This discrepancy in inflow volumes was thought to be due to a change in irrigation practices on the drainage area. Jordan et al. (2003) cited that soybeans were grown on the drainage area during years 1995 and 1997, and corn was grown during 1996. While changes in crops may have contributed to a change in drainage area runoff volumes for years 1 and 2, additional changes such as alterations to drainage ditches could have also contributed to this discrepancy.

Given these runoff inflow depths, separate runoff coefficient values were computed for each year of record accordingly:

$$C_1 = \frac{10.08\,\text{in.}}{43.3\,\text{in.}} = 0.233 \tag{5-13}$$

$$C_2 = \frac{25.6\,\text{in.}}{51.0\,\text{in.}} = 0.502 \tag{5-14}$$

The resulting C_1 and C_2 values were computed in order to show that both years of record exhibited rational runoff coefficients for an agricultural drainage area with drainage ditches. Again, due to the discrepancies between IN_1 and IN_2, the resulting

C_2 (0.502) was more than double C_1 (0.233), suggesting that contributing drainage area properties changed significantly after the first year of record. Despite these differences, both C_1 and C_2 were rational, suggesting that that estimated IN_1 and IN_2 depths also were reasonable.

In order to separate 1-min combined inflow volumes into (1) 1-min direct rainfall volumes and (2) 1-min runoff inflow volumes, two annual separation ratios were computed based on the annual IN_1 and IN_2 volumes and corresponding annual $(IN + P)_1$ and $(IN + P)_2$ volumes. These separation ratios were computed accordingly:

$$R_1 = \frac{IN_1}{(IN + P)_1} = \frac{1{,}265{,}341 \text{ ft}^3}{1.77 \times 10^6 \text{ ft}^3} = 0.715 \tag{5-15}$$

$$R_2 = \frac{IN_2}{(IN + P)_2} = \frac{3{,}215{,}598 \text{ ft}^3}{3.81 \times 10^6 \text{ ft}^3} = 0.844 \tag{5-16}$$

where the resulting separation ratios R_1 and R_2 represent the estimated proportion of the annual combined inflow (runoff inflow + direct rainfall) entering the wetland that was due to runoff inflow for years 1 and 2. They are not runoff coefficients (see Equations 5-13 and 5-14). The resulting R_1 and R_2 ratios were further assumed to represent the proportion of each 1-min combined inflow volume that was contributed by runoff inflow. Given this assumption all 1-min combined inflow volumes in year 1 were multiplied by 0.715 and those for year 2 by 0.844 in order estimate corresponding 1-min runoff inflow volumes for the record period:

$$IN_{i(1)} = R_1 (IN + P)_{i(1)} \tag{5-17}$$

$$IN_{i(2)} = R_2 (IN + P)_{i(2)} \tag{5-18}$$

where *i* represents the 1-min time increment and the subscripted 1 and 2 indicate years 1 and 2. Corresponding 1-min rainfall volumes P_i (ft^3) were then computed by subtracting IN_i (ft^3) from corresponding $(IN + P)_i$ volumes (ft^3):

$$P_{i(1)} = (IN + P)_{i(1)} - IN_{i(1)} \tag{5-19}$$

$$P_{i(2)} = (IN + P)_{i(2)} - IN_{i(2)} \tag{5-20}$$

With 1-min rainfall (P_i) and 1-min runoff inflow (IN_i) volumes defined for each year of record, all necessary hydrologic inputs were ready for model simulations. The resulting separated $P_{i(1)}$ and $P_{i(2)}$ rainfall 1-min inputs summed respectively to 43.4 and 51.0 in. for years 1 and 2. Similarly, $IN_{i(1)}$ and $IN_{i(2)}$ 1-min runoff inputs summed to 10.1 and 25.7 in. for years 1 and 2.

5.1.6.1 Separated rainfall distribution

While the ratio method allowed for the relatively simple and rational separation of the 1-min combined inflow volumes, it also introduced some error into the resulting 1-min rainfall and runoff inflow volumes. Because the ratios R_1 and R_2 were multiplicative, if combined inflow occurred for a given 1-min interval, both rainfall and runoff inflow were computed to occur as well. In reality, rainfall does not always occur concurrently with runoff, and generally ends before runoff depending on the drainage area properties. The ratio method, however, distributed rainfall and runoff over the same combined inflow pattern. Therefore, even when combined inflow occurring at the end of a storm event was very small and most likely attributable only to runoff inflow, the corresponding ratio (R_1 or R_2) divided the combined inflow into both rainfall and inflow. As a result of this method limitation, a large number (91.8% of all hourly rainfall depths) of very small rainfall volumes (amounting to hourly depths over the wetland of ≤ 0.01 in.) were generated, which resulted in a high annual average of 293 of "rainy" days. However, 196 of "rainy" these days were found to have daily depths of 0.01 in. or less. Therefore, an estimated 97 days/year experienced rainfall with depths greater than 0.01 in., which is reasonable for the Baltimore, MD area, which has been cited to have 90 to 114 wet days/yr (climatezone.com; McCuen 2013). While these small rainfall depths did not have a significant impact on the inflow volumes to the wetland, they did hinder ET in the model as ET was bypassed in the model if rainfall occurred with a given hour of simulation. This effect is addressed and discussed in more detail during the hydrologic calibration of the Barnstable 1 in Section 5.3.1.

5.1.7 Influent pollutant concentration 1-min inputs

Jordan (2013) provided weekly composite concentrations of TSS, NH_4^+, and NO_3^- over the two-year study period (see Figure 5-8). Because the model developed in the current study operates on a 1-min time interval, 1-min influent water quality concentrations were also required as inputs in addition to the 1-min runoff influent and direct rainfall volumes (ft^3) developed in Section 5.1.6. Input 1-min pollutant concentrations were assumed to be equal to the corresponding weekly composite concentrations recorded by Jordan et al. (2003). Missing weeks of data were also assumed to have influent pollutant concentrations equal to those of next recorded week. All 1-min concentrations had units of mg/L. In reality, influent concentrations can vary greatly from minute to minute (McDiffett et al. 1989; Lee and Bang 2000). However, the method used herein was found to provide the best estimate of 1-min influent concentrations given the lack of sufficient data and knowledge in the literature surrounding the behavior of pollutant concentrations in runoff at such small time scales.

5.1.8 Barnstable 1 Outflow

A 120° V-notch weir was used to measure outflow from the Barnstable 1 wetland (Jordan 2013). The following weir equation computed outflowing rates from the wetland in order to model outflow from a 120° V-notch weir:

$$Q_w(t) = 4.33 \cdot d_w(t)^{2.5} \tag{5-21}$$

where $Q_w(t)$ is the flowrate (cfs) over the weir at time *t* (min) and $d_w(t)$ is the depth of water over the weir at time *t*. Intermittent depths $d_w(t)$ were reported by Jordan (2013).

5.2 SUMMARY OF USER INPUTS FOR BARNSTABLE 1 WETLAND

The following section describes the user inputs specific to the model for the Barnstable 1 wetland. Due to uncertainty associated with the values, a number of inputs required calibration. Estimated ranges were defined for each of these uncertain inputs based on literature values and on the properties of the Barnstable 1 wetland and its contributing drainage area. All user inputs and their initial values are shown in Table 5-5. A number of model inputs related to wetland inflow and rainfall (i.e., number of wet days per year, drainage area runoff coefficient, etc.) were not required for the Barnstable 1 model because 1-min inflow and rainfall volumes derived from Jordan (2013) data were directly input into the model.

Table 5-5 All model parameters for the Barnstable 1 model, their assigned initial values, and corresponding estimated ranges.

User input	Initial value	Estimated range
Number of years of simulation	3	---
Cell length (ft)	104	---
Number of cells in wetland design	13	---
FID vector	See Table 5-6	---
Vegetation specification for each cell (no vegetation = 0, emergent = 1, submerged = 1)	See Table 5-6	---
Initial water depth in each cell	See Table 5-6	---
Bottom elevation in each cell	See Table 5-6	---
Berm height at exit of each cell	no berms	---
Weir invert height H_I (ft)	1.32	---
Hydraulic conductivity K_V (ft/d)	0	0 -0.00283
Shelter factor f_s	0.75	0.5-1
Wetland albedo a	0.159	0.066-0.252
Leaf area index LAI	6.5	4-16.8
Maximum leaf conductance C^*_{leaf} (mm/s)	9.7	0-19.87
Emergent vegetation height z_v (m)	0.635	0.5-0.8
Wind speed measurement height z_m (m)	2	---
Maximum photosynthesis rate $PMAX$ (mg-O_2/m^2-hr)	910	777-1063
TSS particle diameter D (m)	$2x10^{-6}$	$1x10^{-7}$-$1x10^{-4}$
Initial water temperature $T_{w(o)}$ (°C)	15.5	1.11-32.2
Nitrification reaction rate K_{NIT} (hr^{-1})	0.0100	0.000417-0.0196
Denitrification reaction rate K_{DNT} (hr^{-1})	0.0208	0.00375-0.0379
TSS wetland background concentration TSS_o (mg/L)	3	2-5
NH_4^+ wetland background concentration $NH4_o$ (mg/L)	0	0.2-1.5
NO_3^- wetland background concentration $NO3_o$ (mg/L)	0	---
DO initial concentration in wetland DO_o (mg/L)	7.5	5-15
Influent DO concentration DO_{in} (mg/L)	15	5-15

5.2.1 Model layout of Barnstable 1 wetland

The Barnstable 1 wetland has a triangular shape with an area of 1.3 ha (3.2 ac). A clear flow path through the wetland was not discernible when it was full (Jordan et al. 2003). While the wetland had two inlets (Sharifi et al. 2013), only the main inlet at the top of the wetland area (see Figure 5-11) was simulated by the model, as the model only has the capacity for one inlet. Additionally, the main inlet at the top of the wetland was cited to deliver considerably more water to the wetland than the secondary inlet and was used for all inflow sampling (Jordan 2013). Therefore, the current study assumed negligible inflow from the secondary inlet as well as any runoff into the wetland that did not enter through the main inlet. In order to capture the shape of the wetland, an initial cell size of 104 ft x 104 ft was chosen; which divided the wetland into 13 cells, each with an area of 10,753 ft^2. Cell depths and flow directions were estimated from a computer program-generated elevation map provided by Jordan (2013) and shown in Figure 5-1. The initial 13-cell model configuration for the Barnstable 1 wetland is shown in Figure 5-11. This design has a principal flowpath consisting of five cells.

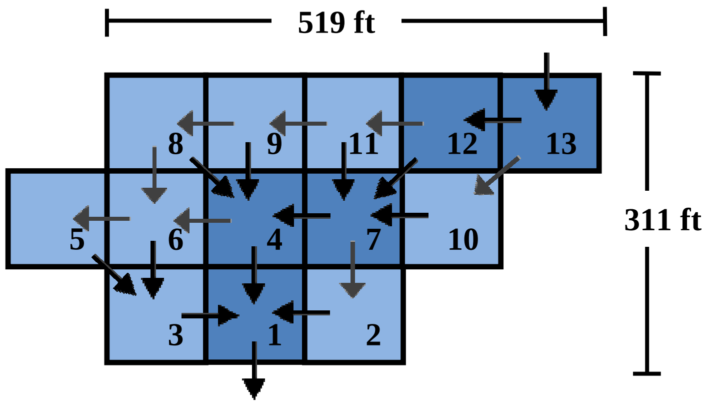

Figure 5-11 Model representation of the Barnstable 1 wetland. The numbers represent the FID values for each cell number and the arrows designate the initial postulated flowpath of the wetland. The darkly-shaded cells highlight the main flowpath through the wetland. Black arrows represent the primary flowpath and grey arrows represent the secondary flowpath.

Table 5-6 Barnstable 1 wetland model design specifications for the primary and secondary flowpaths (FID and FID2), initial water depths, elevations, and vegetation descriptors for each wetland cell.

Cell	FID	FID2	Initial Water Depth (ft)	Elevation (ft)	Vegetation Descriptor
1	0	1	1.29	0	2
2	1	2	1.29	0	2
3	1	3	1.29	0	2
4	1	6	0.785	0.503	1
5	3	5	0.254	1.03	1
6	3	5	0.254	1.03	1
7	4	2	1.19	0.103	1
8	4	6	0	2.26	1
9	4	8	0.468	0.820	1
10	7	10	0.000	1.38	1
11	7	9	0.878	0.410	1
12	7	11	0.238	1.05	1
13	12	10	0	1.87	1

The specifications for the Barnstable 1 model design are summarized in Table 5-6. In order to simplify the inputs, the datum for the lowest cell bottom elevation was set to equal zero at as opposed to using the original elevations used shown in

Figure 5-1. This linear transformation resulted in a weir invert height (H_I) of 1.32 ft, which is the difference between the elevation of the weir, 98.90 m (324 ft) and the bottom elevation at the outlet of 98.50 m (323 ft.).

Initial water depths were estimated based on the initial recorded water depth in the outlet of -0.0276 ft on May 3, 1995, which was the first day of simulation. The Flow Identification Direction numbers for the primary (FID) and secondary (FID2) flowpaths (see Section 4.4.1) specify the cell number(s) into which a given cell flows. Vegetation descriptors of 1 represent submerged vegetation while those of 2 represent emergent vegetation. As mentioned above, all elevations, flowpaths, and water depths were estimated from the topographic map provided by Jordan (2013) (see Figure 5-1).

5.2.2 Weir invert height H_I

The weir invert height H_I (ft) was defined as the distance from the bottom of the wetland to the bottom of the outlet weir. As discussed in Section 0, H_I was estimated to equal 1.32 ft based on data provided be Jordan (2013). Error in H_I was assumed to be negligible and, therefore, was not calibrated.

5.2.3 Hydraulic conductivity K_V

The vertical hydraulic conductivity K_V of the wetland controls the rate of infiltration lost from the surface storage within the wetland. A number of sources of error are associated with the estimation of K_V including soil heterogeneity, macropores, and soil grain orientation and shape. Jordan et al. (2003) initially assumed infiltration to be negligible due to the underlying clay layer of 0.5 m (1.64 ft) below the wetland. However, they did report that errors in calculated annual water balances at the site may have been due to the exclusion of infiltration and dam seepage in their calculations. Given this information, the current study assumed an initial infiltration rate of 0 but allowed for the possibility that infiltration occurred at the site at a very slow rate due to the low permeability of the underlying clay layer. This clay layer was assumed to have a K_V range of 10^{-9} to 10^{-6} cm/s (2.83×10^{-8} to 0.00283 ft/d) (Fetter 2001). Therefore, while the initial K_V input value was 0, a corresponding range of 0 to 0.00283 ft/d was specified for calibration.

5.2.4 PET input parameters

PET input parameters included albedo (a), shelter factor (f_s), maximum leaf conductance (C^*_{leaf}), and height of wind measurements (z_m). All initial PET input values were assigned based on literature ranges found and discussed in Section 0. While z_m was assumed to be constant given that NOAA (1998) reported that all wind speed measurements were taken from gages situated at a height of 2 m, all other PET parameters were calibrated for the Barnstable 1 model. Emergent vegetation height above water (z_v) was set equal to 0.635 m, which was the mean value for the wetland as estimated by Jordan (2013).

5.2.5 Maximum photosynthesis rate

As estimated in Section 3.4.2.4.1, $PMAX$ was initially set to equal 910 mg/m^2-hr based on a mean literature value for annual wetland oxygen production of

1710 g-O_2/m^2-yr. This annual photosynthesis varied greatly within the literature (Mitsch and Gosselink 1993; USEPA 2000; Tian et al. 2010). The corresponding $PMAX$ values calculated for the annual rates from each source were 777, 888, and 1063 mg/m^2-hr. Therefore $PMAX$ was calibrated in the Barnstable 1 model within estimated range of 777 to 1063 mg/m^2-hr.

5.2.6 TSS particle diameter *D*

While reports have studied the TSS particle size distributions of agricultural runoff, distributions varied greatly between study sites with different soil types and geographic location (Liebens 2001; Pathak et al. 2004). Additionally, particle size studies were not found for the mid-Atlantic region of the US. Pathak et al. (2004), however, observed that agricultural runoff particle distributions in India closely followed topsoil distributions during large storm events as well at peak flows for all storms. Liebens (2001) also found that TSS particle diameter sizes in swales that received agricultural runoff reflected the high content of sand in the contributing drainage areas, which were located in Escambia County, Florida. Given these results, the current study assumed that the particle size distribution of the contributing drainage area was a reasonably accurate representation of the corresponding runoff particle diameter distribution for agricultural runoff at the Barnstable 1 site.

Under this assumption, a simple particle diameter distribution was estimated for agricultural runoff entering the wetland used to calibrate the model developed in the current study. The Barnstable 1 wetland received runoff from a drainage area comprised of a silt loam soil (Jordan et al. 2003). According to the USDA textural soil classification study guide, there are two definitions of silt loam: (1) greater than or equal to 50% silt, 12-27% clay, and the remaining percent sand; and (2) 50-80% silt, < 12% clay, and the remaining percent sand (USDA 1987). Using the USDA soil texture triangle (see Figure 5-12), a silt loam with 65% silt, 15% clay, and 20% sand was assumed for the soil within the Barnstable 1 drainage area. The USDA (1987) defined clay to have a diameter of < 0.002 mm, silt to have a diameter between 0.002 and 0.05 mm, and sand to have a diameter range of 0.05 mm (very fine sand) to 2 mm (very coarse sand). Based on these proportions, a simple particle diameter distribution was estimated and is shown in Table 5-7. From these estimates a median particle diameter of 2 μm was estimated for the initial TSS particle diameter *D* of the agricultural runoff entering the Barnstable 1 wetland.

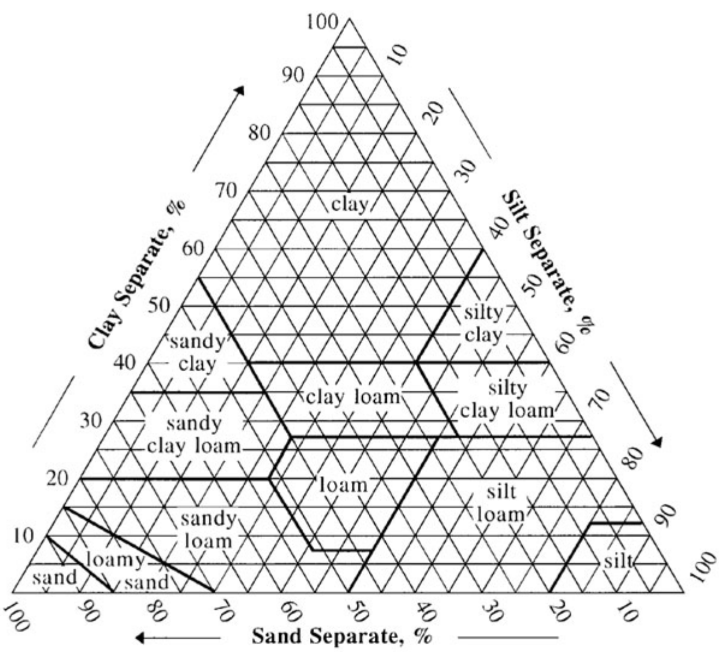

Figure 5-12 The USDA soil texture triangle (soils.usda.gov).

Table 5-7 Estimated particle diameter distribution for runoff entering Barnstable 1 wetland used for model calibration.

Particle Diameter (μm)	% Weight finer
50	80
2	20

5.2.7 Initial water temperature $T_{w(o)}$ (°C)

The first day of record simulated for the Barnstable 1 model was May 3, 1995 (day of year 123). Based on the 59 years of climactic data obtained from NOAA (see Section 4.2.4), the average daily mean, minimum, and maximum air temperatures for this date were respectively 15.5, 1.11, and 32.2°C. Mean daily water temperatures simulated within the model followed mean daily air temperatures closely (see Section 4.3.1). Therefore, the initial $T_{w(o)}$ input value was set equal to 15.5°C to match the mean daily temperature for DOY 123. An estimated range of 1.11 to 32.2°C was also used as a guide in the calibration of $T_{w(o)}$ in the Barnstable 1 model.

5.2.8 First-order nitrification K_{NIT} and denitrification K_{DNT} rate constants

The current study used first-order reactions to simulate the chemical processes of nitrification and denitrification. Rate constants were used to control the rate at which each of these processes occurred within each cell of the model wetland. Furthermore, these rate constants were applied on a one-minute interval and input into the model with units of hr^{-1}. Most studies reported rate constant values with units of d^{-1}, yr^{-1}, or m/yr (Kadlec and Knight 1996; Carleton et al. 2001; Bastviken 2006; Chavan and Dennet 2008). Rate constants with units of 1.0/time represent volumetric constants while those with units of length over time were area-based constants. A summary of rate constants reported in the literature in their original units for both nitrification (K_{NIT}) and denitrification (K_{DN}) is provided in Table 5-8. Volumetric rate constants were used in the current study as they were used within each wetland cell, each of which was assumed to act as an individual completely mixed flow reactors (CMFR) rather than plug-flow reactors. Analogous rate constants for such a small time interval could not be found in the literature.

Table 5-8 Literature K_{NIT} and K_{DN} values in their original units.

Source	K_{NIT}	K_{DN}
Kadlec and Knight (1996)	18 m/yr	35 m/yr
Carleton et al. (2001)	30.1 yr^{-1} (-6.9 – 86.7 yr^{-1})	53.4 yr^{-1} (-0.7 – 203 yr^{-1})
Bastviken (2006)	35.3 m/yr (0.5 – 70 m/yr)	185 m/yr (10 – 360 yr^{-1})
Chavan and Dennet (2008)	0.24 d^{-1} (0.01 – 0.47 d^{-1})	0.50 d^{-1} (0.09 – 0.91 d^{-1})

Rate constants could not be reliably converted to the desired units of hr^{-1} due to the uncertainties of rate variation throughout the day. Factors affecting reaction rates within a given day include oxygen levels, water temperature, pH, and the amount of bioaviable organic carbon (Bastviken 2006). Additionally, rate constants reported in the literature represent the overall rate of chemical plug-flow processes for entire wetland systems, while the current study aims to simulate the rate of chemical processes within each designated completely mixed flow reactor (CMFR) wetland cell in order to better understand internal wetland processes. The reported rate constants represent a wide variety of wetland treatment systems, preventing them from being reliably used to characterize individual wetland systems.

Given these difficulties in obtaining rate constants from the literature, all rate constants were determined by calibration. Literature values (see Table 5-8) consistently reported K_{DN} values to be approximately double the corresponding K_{NIT} values regardless of the units used to express them. These relative magnitudes were used as a guide to initial K_{DN} and K_{NIT} estimates in calibration. The initial

K_{DN} and K_{NIT} input values were estimated to equal 0.0208 and 0.01 hr^{-1} based on the mean daily values reported by Chavan and Dennet (2008) in Table 5-8 as they were the values based on the smallest time interval in the literature. Corresponding estimated ranges for K_{DN} and K_{NIT} were also estimated on the ranges observed by Chavan and Dennet (2008), which were respectively 0.00375 to 0.0379 hr^{-1} for K_{DN} and 0.000417 to 0.0196 hr^{-1} for K_{NIT}.

5.2.9 Wetland background concentrations (TSS_o, $NH4_o$, $NO3_o$, DO_o)

An initial estimated wetland background concentration of 3 for TSS_o was taken from USEPA (2000). This value represents the typical background TSS concentration found in free water surface constructed wetlands due to internal wetland processes such as resuspension and plant degradation. According to USEPA (2000), an estimated calibration range for TSS_o of 2 to 5 mg/L was also chosen.

Typical $NH4_o$ values from 0.2 to 1.5 were reported by EPA (2000) for free surface water treatment wetlands. Given that observed weekly effluent NH_4^+ concentrations had a mean of 0.232 mg/L and a minimum of 0.0701 mg/L, it was thought that the Barnstable 1 model wetland $NH4_o$ values were lower than those reported by USEPA 2000. In order to prevent positive bias in the simulated effluent NH_4^+ concentrations an initial $NH4_o$ value of zero was input into the Barnstable 1 model.

Wetland background nitrate levels $NO3_o$ are generally zero (Kadlec and Knight 1996; USEPA 2000). Therefore, the current study assumed an input $NO3_o$

value of zero. The input parameter $NO3_o$ was not calibrated as the literature consistently reported values of zero for wetland background nitrate concentrations.

DO_o only represented the initial dissolved oxygen concentration in the wetland as DO was generated in the wetland via surface aeration in all cells and submerged vegetation photosynthesis in three cells (see Table 5-5). Jordan (2013) estimated that dissolved oxygen levels within the Barnstable 1 wetland were near or at saturation during the day given these DO sources in the wetland. An initial DO_o of 7.5 mg/L was input to the model and was calibrated within the model with an estimated range of 5 to 15 mg/L.

5.2.10 Influent DO concentration DO_{in}

Influent DO concentrations were not recorded by Jordan et al. (2003). Therefore, 1-min DO_{in} values had to be estimated for the Barnstable 1 model. As discussed in Section 5.1, influent DO levels were assumed to equal saturated DO levels based on personal communication with Jordan (2013). Therefore the initial input DO_{in} value was set to 15 mg/L. The USEPA (1998) also reported that urban stormwater runoff throughout the US was found to have DO concentrations of greater than or equal to 5 mg/L and cited that urban runoff generally did not cause downstream DO sags. Therefore, a calibration range of 5 to 15 mg/L was estimated for the Barnstable 1 model.

5.3 PROCEDURE FOR CALIBRATING THE BARNSTABLE 1 MODEL

Manual subjective optimization was used to calibrate the user inputs for the wetland model with the data provided by Jordan (2013). Wetland hydrology was calibrated first and was followed by wetland water quality and all relevant water quality parameters. The bias ($\bar{e}$), the relative bias ($\bar{e} / \bar{y}$), the root mean square error (RMSE), and the relative standard error ($\bar{S}_e / \bar{S}_y$) were used to assess calibration results based on the Barnstable 1 formatted 1-min outflow (cfs), water depth through outlet weir (ft), corresponding wetland water volume storage (ft^3), as well as the weekly water quality (mg/L) and outflow volume (ft^3) data.

Annual depths for all relevant water balance fluxes for years 1 and 2 of the study period, which are summarized in Table 5-9, were also used as guides to assess model performance. Additionally, Table 5-9 shows the maximum and mean 1-min effluent rates from the Jordan (2013) outflow data. Mean effluent rates were computed excluding zero-flows. These flowrates were used as additional measures of model evaluation.

Table 5-9 Annual equivalent depths for all wetland fluxes and storages for both years of the record period. Annual peak and mean effluent flowrates were also included. Notation with * indicates that a given value was computed in the current study. All other depths were summed annually based on Jordan (2013) data.

	Year 1	**Year 2**
Runoff inflow* IN (in.)	10.1	25.7
Surface outflow OUT (in.)	9.59	23.5
Initial storage S_o (in.)	7.24	7.52
Final Storage S_f (in.)	7.52	8.80
Change in storage ΔS (in.)	0.286	1.28
Estimated Rainfall* P (in.)	43.4	51.0
Peak outflow discharge (cfs)	8.80	9.27
Mean outflow discharge (cfs)	0.0750	0.123

Because water quality transformations are dependent on the hydrology (i.e., flowpath, water depths, retention time, flowrates, etc.) of a given wetland, all of the hydrologic components (i.e., 1-min outflow rates, 1-min wetland storage volumes, 1-min water depth at the outlet weir, and weekly effluent volumes) of the wetland were calibrated first, followed by all water quality model components. All hydrologic calibration trials and resulting goodness-of-fit statistics as well as annual equivalent depths for all relevant water fluxes are listed in Table 5-11 and Table 5-12.

5.3.1 Calibration of Barnstable 1 hydrology

5.3.1.1 Trial 1

The initial trial produced positively biased hydrology outputs. Simulated 1-min outflow rates had a $\bar{e}/\bar{Y}$ of 16.0% and are shown in Figure 5-19 and Figure 5-20. Weekly effluent volumes also reflected this bias with a $\bar{e}/\bar{Y}$ of 15.4% (see Figure 5-15 and Figure 5-16). Similarly, wetland storage volumes (Figure 5-17 and Figure 5-18) and weir water depths (Figure 5-19 and Figure 5-20) were biased by respective values of 13.9 and 7.7%. Despite these biases, all resulting $\bar{S}_e/\bar{S}_y$ show strong agreement between the model and the observed data, ranging from 0.192 for 1-min outflow rates to 0.445 for weekly effluent volumes. The biased results of the model suggested that the model was over-predicting storage within the wetland as well as outflow from the wetland. In order to reduce these biases, it was necessary to reduce storage in the wetland by increasing the amount of water within the system lost due to ET and infiltration (*I*), which would in turn reduce the wetland outflow flux. An overall water balance was also performed in trial 1 in order to ensure that the model was conserving water correctly. The resulting volumes associated with all water balance fluxes are shown in Table 5-10.

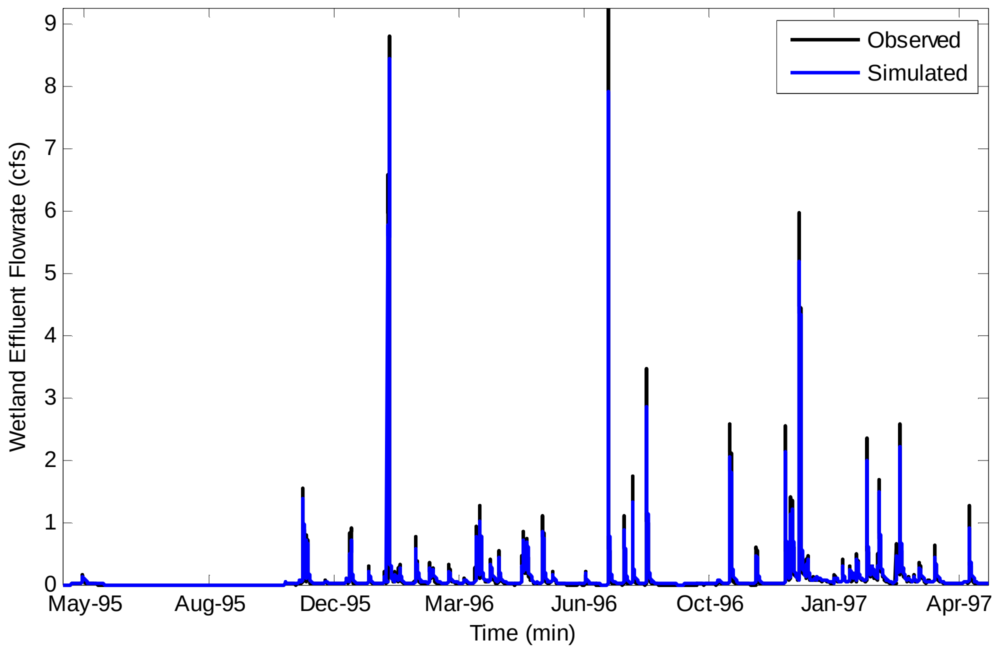

Figure 5-13 Compares the model-simulated (blue) and observed (black) 1-min effluent flowrates (cfs) for trial 1 of calibration.

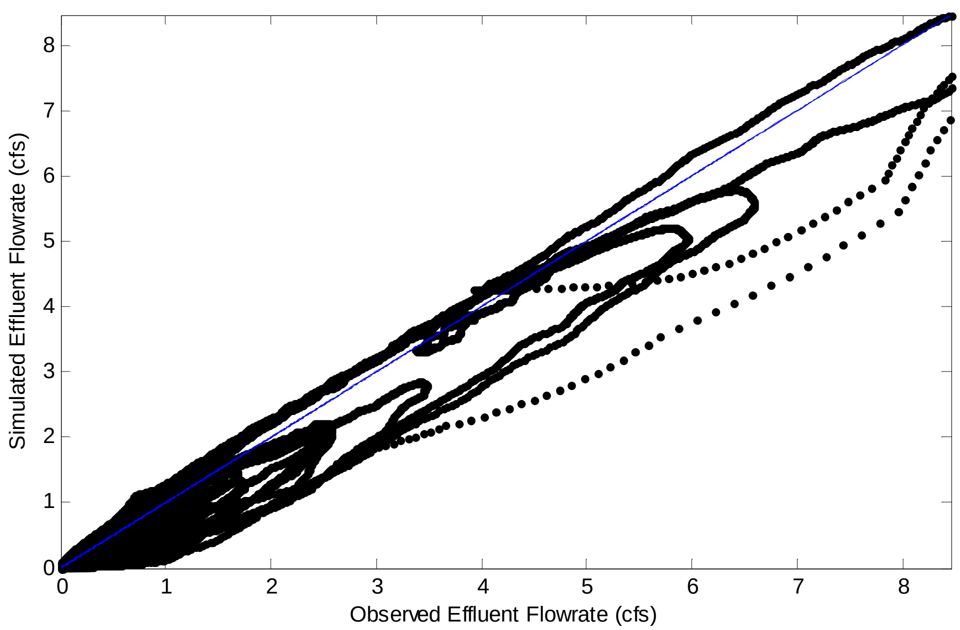

Figure 5-14 Comparison plot of simulated versus observed 1-min effluent flowrates (cfs) for trial 1 of calibration.

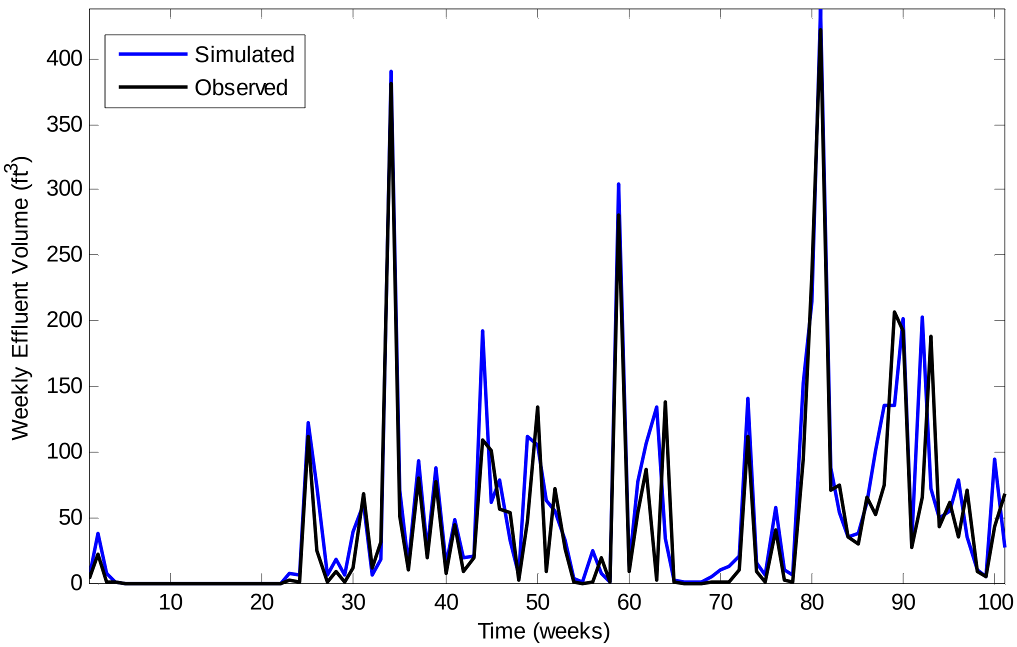

Figure 5-15 Compares the model-simulated (blue) and observed (black) weekly effluent (ft^3) for trial 1 of calibration.

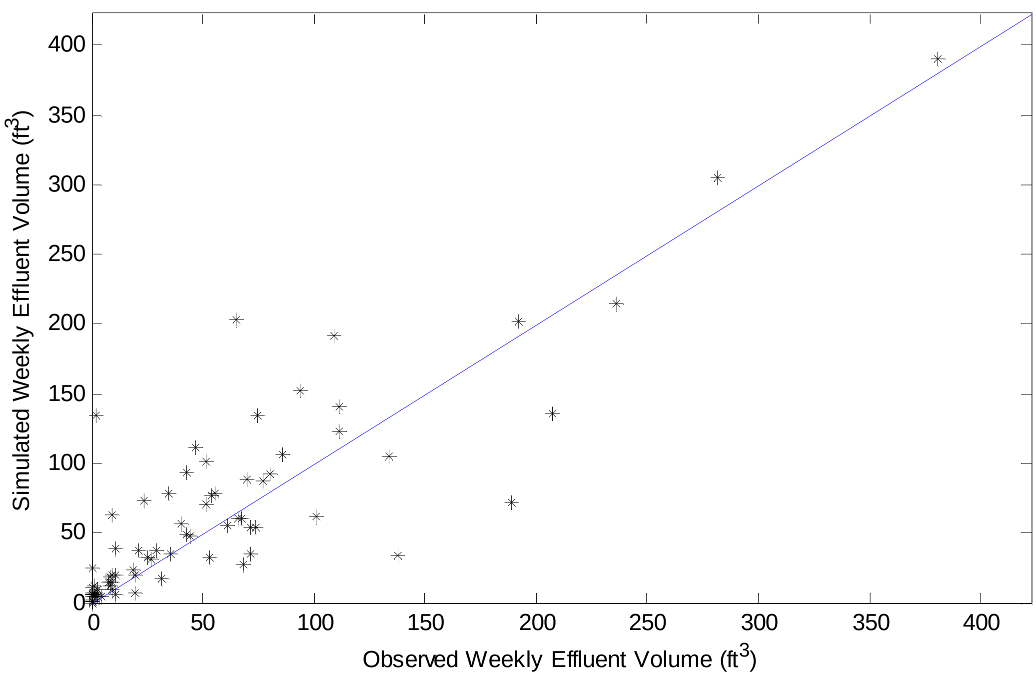

Figure 5-16 Comparison plot of simulated versus observed weekly effluent volumes (ft^3) for trial 1 of calibration.

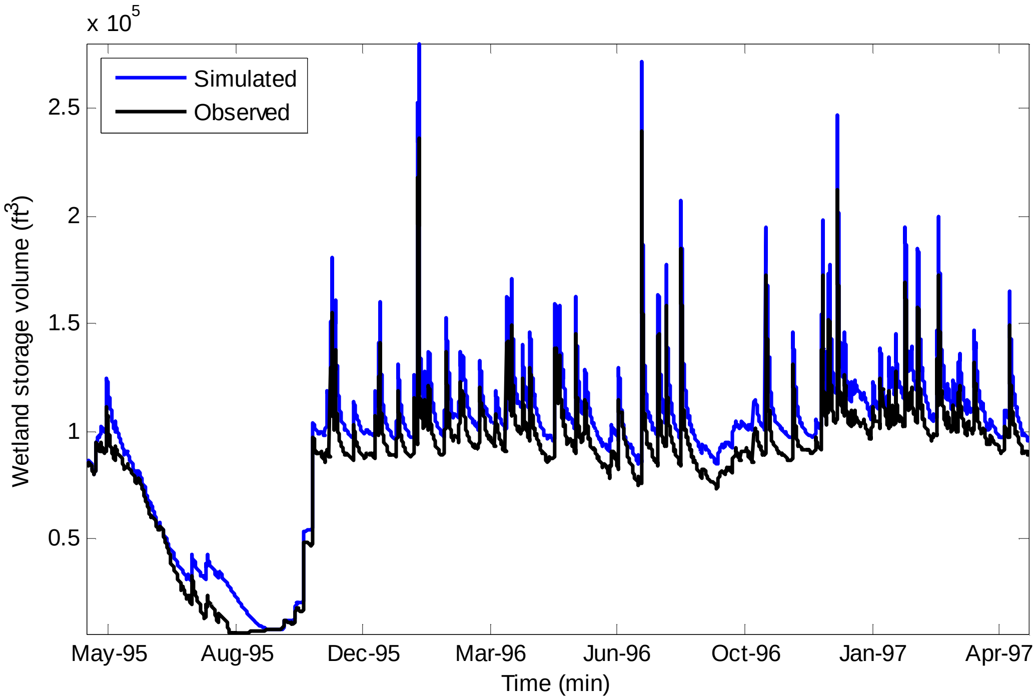

Figure 5-17 Compares the model-simulated (blue) and observed (black) wetland storage volume (ft^3) on a 1-min time interval for trial 1 of calibration.

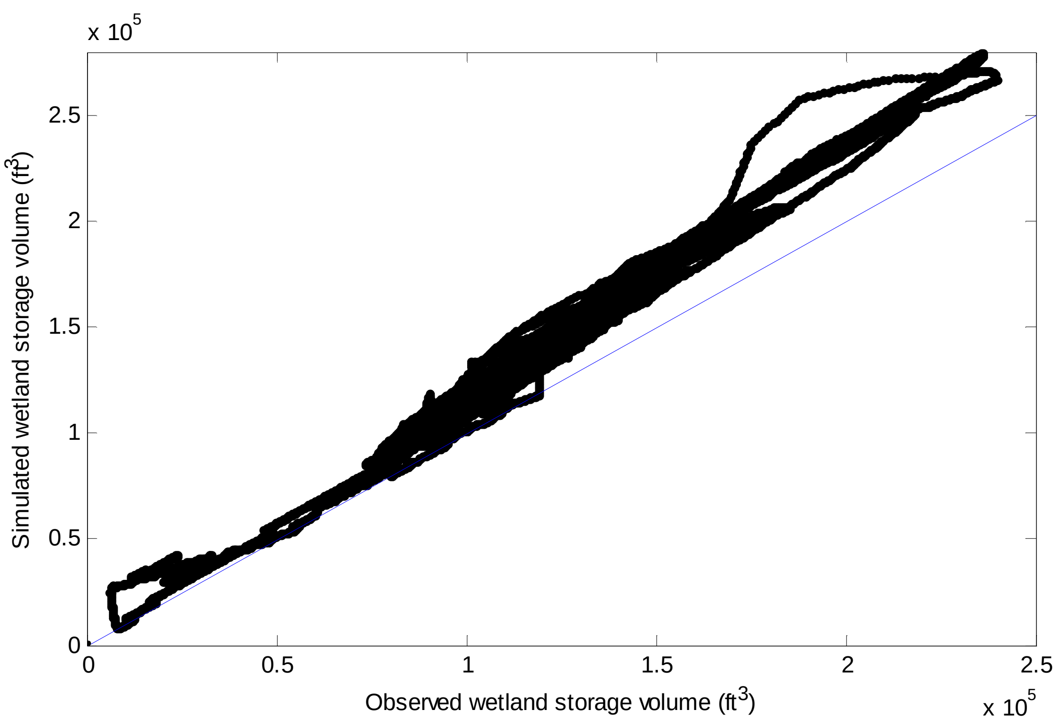

Figure 5-18 Comparison plot of simulated versus observed 1-min wetland storage volumes (ft^3) for trial 1 of calibration..

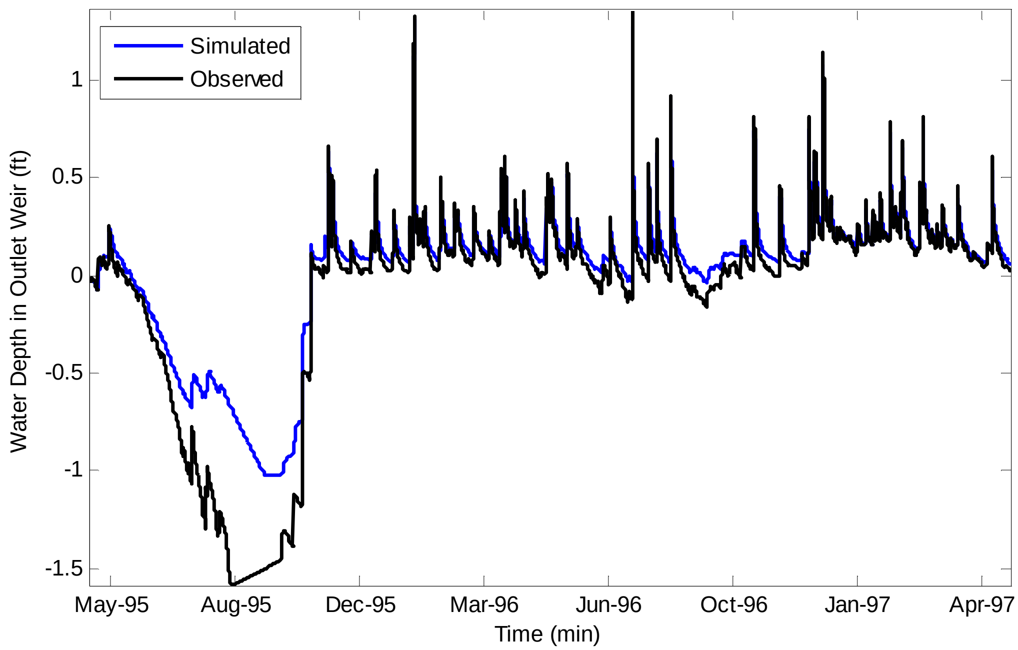

Figure 5-19 Compares the model-simulated (blue) and observed (black) water depths (ft) in the outlet weir on a 1-min time interval for trial 1 of calibration.

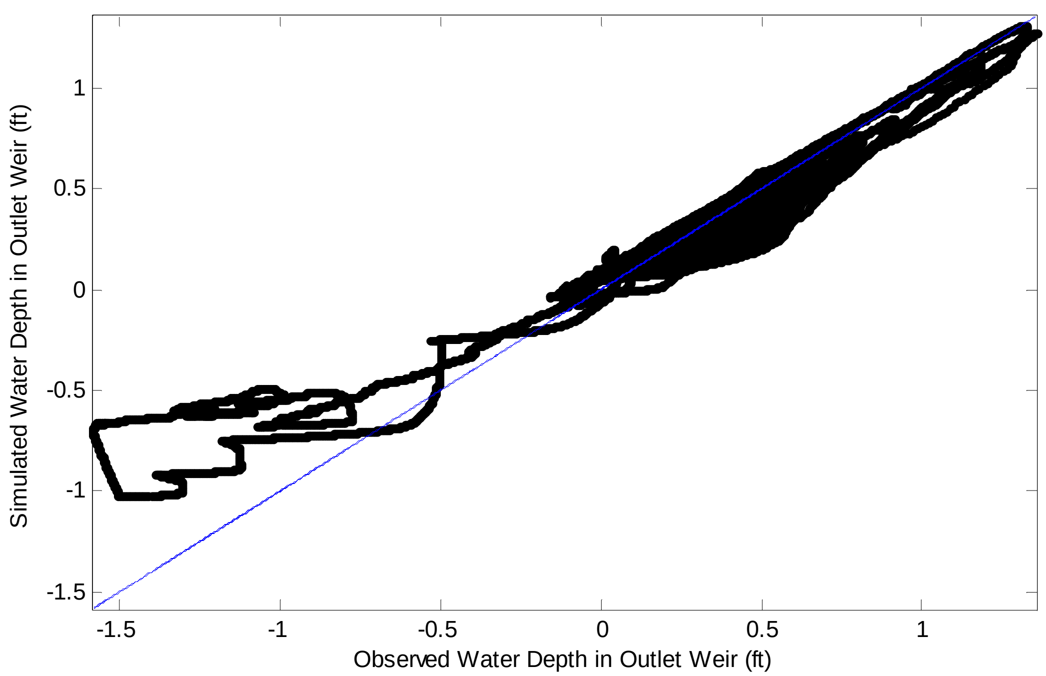

Figure 5-20 Comparison plot of simulated versus observed 1-min water depths at the outlet weir (ft) for trial 1 of calibration.

Table 5-10 Resulting water balance and corresponding water fluxes for trial .

	Year 1 (ft^3)	**Year 2 (ft^3)**
Runoff inflow (*IN*)	1,268,692	3,219,170
Estimated Rainfall (*P*)	505,481	594,752
Surface outflow (*OUT*)	1,602,323	3,652,048
Change in storage (ΔS)	13,590	18,591
Wetland ET (*ET*)	158,259	143,283
Wetland Infiltration (*I*)	0	0
Water Balance	2.53×10^{-8}	-0.04

5.3.1.2 Trial 2

A large portion (91.8%) of the hourly rainfall depths were very small (≤ 0.01 in.), which could be credited to the ratio method used to separate rainfall inputs from other inflow inputs in the original combined inflow input reported by Jordan (2013) (see Section 5.1.6.1). For perspective, 0.01 in. is the smallest depth recorded by a typical tipping bucket rainfall gage, which can be triggered by heavy dew or fog. In order to increase simulated ET, the model was changed to suppress ET only if a rainfall depth greater than 0.01 in. fell in a given hour. This change improved goodness-of-fit statistics for all four outputs compared with Jordan (2013) observed data. The 1-min outlet water depths had a relative bias of 0.046, while the other outputs had higher relative biases ranging from 0.095 and 0.101. All resulting $\bar{S}_e / \bar{S}_y$ showed strong agreement between the model and observed data, ranging from 0.202 for 1-min outflow rates and outlet water depths to 0.432 for weekly effluent volumes.

5.3.1.3 Trial 3

While statistics improved from trials 1 to 2, the model was still positively biased, suggesting that too much water was still being allocated to storage and outflow within the model. Therefore, more water needed to be routed to the fluxes of

ET and infiltration. Simulated annual ET for year 1 (19.9 in.) was also low compared to estimated annual wetland ET range of 25.7 to 38 in. that was computed in Section 4.2.6. Additionally, the simulated ET depth for year 2 (28.6 in.) was at the lower end of this estimated annual wetland ET range, suggesting that the ET rate could be increased while maintaining rational ET depths for both years of record. Based on these annual ET depths, the leaf area index (LAI) was increased from 6.5 to 10 in trial 3 in order to increase ET rates.

Results from trial 3 showed further reductions in bias, producing relative bias values less than 0.08 for all of the time series outputs with the largest value being 0.076 for 1-min outflow rates (see Table 5-12). The $\overline{S}_e / \overline{S}_y$ values for all 1-min outflow rates and wetland storage volumes were slightly larger than those produced in trial 2. These increased $\overline{S}_e / \overline{S}_y$ values were likely due to the local biases present in the simulated 1-min values. The model appears to be more sensitive to changes to the water balance than the corresponding observed data. This difference in model prediction of wetland storage volumes, water depths, and outflow rates may be due to the discretization of the wetland into cells. The cell structure and defined flowpath of the model may not fully characterize the complexity of the actual flowpath and elevations of the Barnstable 1 wetland.

Finally, the annual ET depths that resulted from trial 3, which increased with respective depths for years 1 and 2 of 22.5 and 35.7 in., were analyzed. While simulated ET for year 2 was still within the previously estimated range of 25.7 to 38 in. (see Section 4.2.6), the first year ET was still relatively low, which was thought to reflect the 53-day dry period that occurred in the wetland during the summer of 1995.

The respective daily ET rates for years 1 and 2 were computed as 0.0726 in./d over 310 wet days in year 1 and 0.0978 in./d over 365 days in year 2. Given the total number of wet days in the wetland, the ET rates for both years are comparable. The resulting year 1 ET rate represents an annual ET depth of 26.5 in. assuming the wetland held water for 365 days. The discrepancy in ET rates between years 1 and 2 could be due to a number of factors including vegetation growth/maturation in the wetland, differing spring/summer air temperatures, and limitations of ET in year 1 due to shallow water depths. Based on this analysis, while the model simulated more than 10 inches less of ET in year 1 than in year 2, both annual depths appear to be reasonable when compared with the estimate literature range of 25.7 to 38 in.

5.3.1.4 Trial 4

In order to further reduce model overprediction of wetland storage and outflow, infiltration was increased in the model by increasing the input hydraulic conductivity K_V from 0 to 2.38 x 10^{-8} ft/d, which was the specified lower bound for clay hydraulic conductivity as defined by Fetter (2001). This change in K_V did not produce any significant changes in model output as only 0.0001 and 0.00012 in. of water was infiltrated from the wetland over years 1 and 2.

5.3.1.5 Trial 5

Given that notable change in infiltration was not observed in trial 6, K_V was further increased from 2.38 x 10^{-8} to 2.38 x 10^{-3} ft/d in trial 5 to increase infiltration in the wetland and reduce wetland storage and outflow volumes and rates. The resulting annual infiltration depths for years 1 and 2 were 8.69 and 11.8 in. compared well with the input K_V of 2.38 x 10^{-3} ft/d, which equates to 12.4 ft/yr. Both years produced a little less infiltration due to dry periods that left either a portion of or the entire wetland dry. This simulation also produced $\bar{e}/\bar{Y}$ values with magnitudes below 0.05 for all of the time series outputs (i.e., 1-min outflow rates, 1-min water depths at outlet weir, 1-min wetland volume storages, and weekly effluent volumes). Additionally, resulting $\bar{S}_e/\bar{S}_y$ ranged from 0.221 for 1-min water depths at the outlet weir to 0.422 for weekly effluent volumes. Based on these results, the model hydrology was assumed to be sufficiently calibrated. The final calibrated results are shown graphically both against time with corresponding observed data as well as in comparative plots with observed data on the x-axis and simulated results of the y-axis for 1-min outflow rates. These plots are shown in Figure 5-21 and Figure 5-22, respectively, for weekly effluent volumes in Figure 5-23 and Figure 5-24, for 1-min water depths at the outlet weir in Figure 5-25 and Figure 5-26, and in 1-min wetland storage volumes in Figure 5-27 and Figure 5-28. Additionally, all hydrologic calibration trials are documented in Table 5-15.

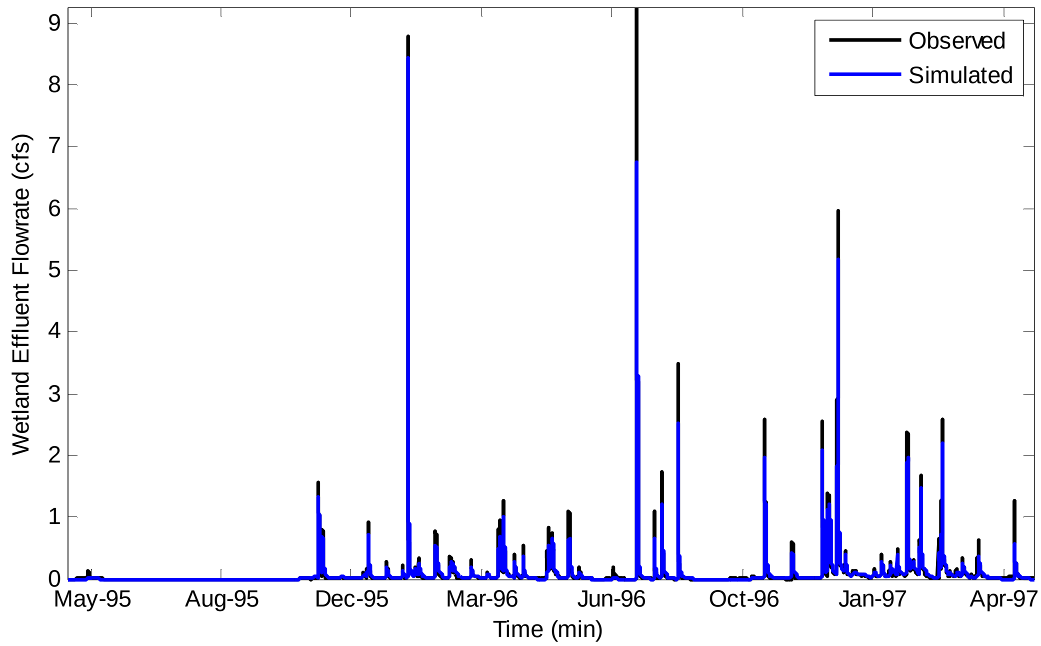

Figure 5-21 Compares the model-simulated (blue) and observed (black) effluent flowrates (cfs) on a 1-min time interval for trial 5 of calibration.

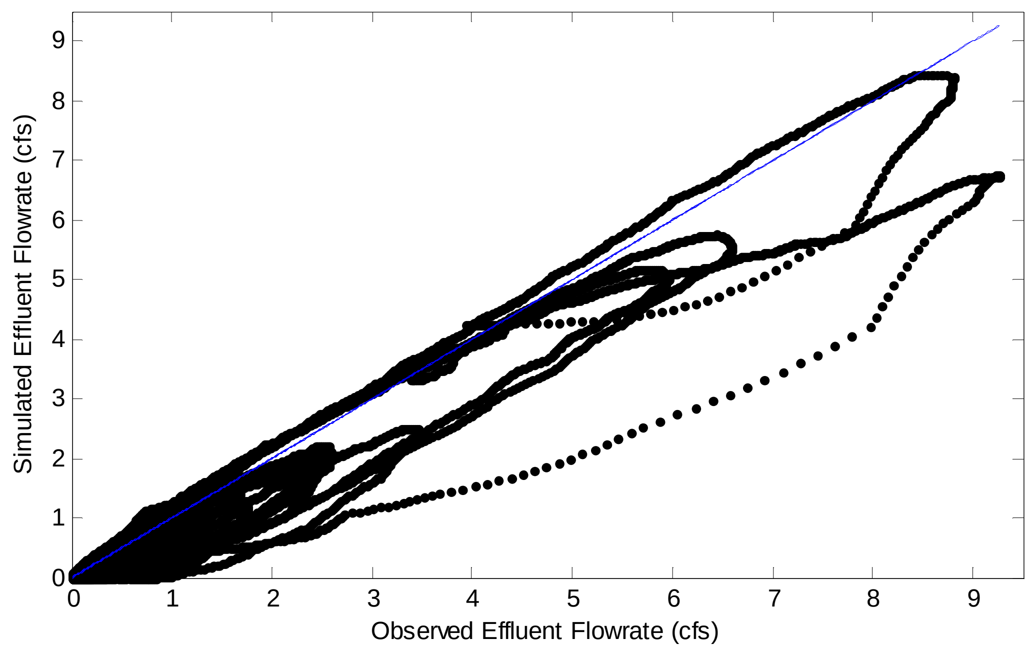

Figure 5-22 Comparison plot of simulated versus observed 1-min effluent flowrates (cfs) for trial 5 of calibration.

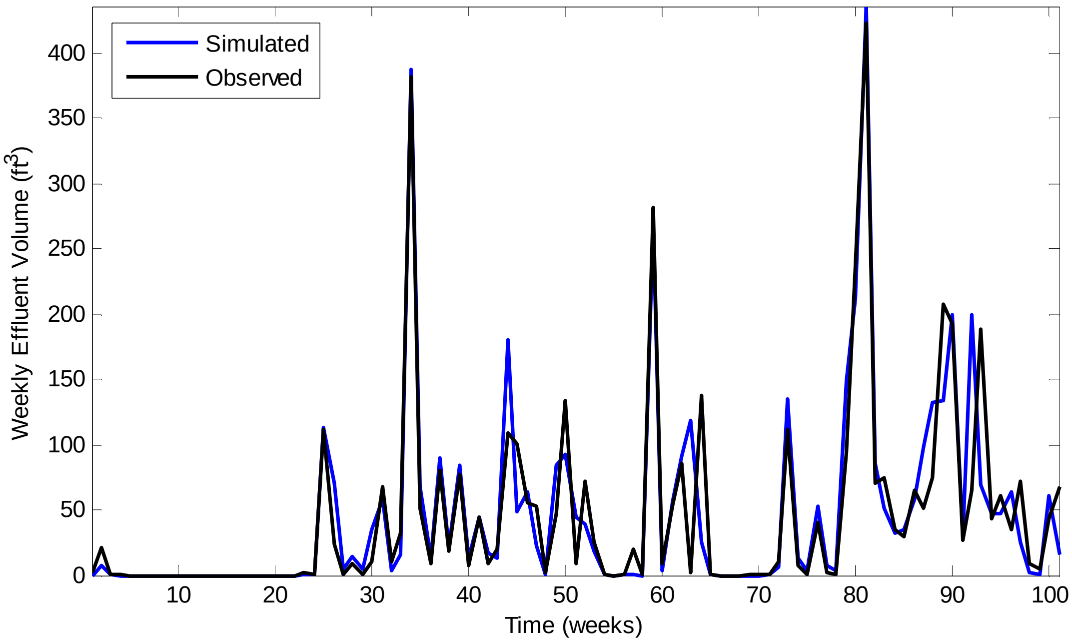

Figure 5-23 Compares the model-simulated (blue) and observed (black) weekly effluent volumes (ft^3) for trial 5 of calibration.

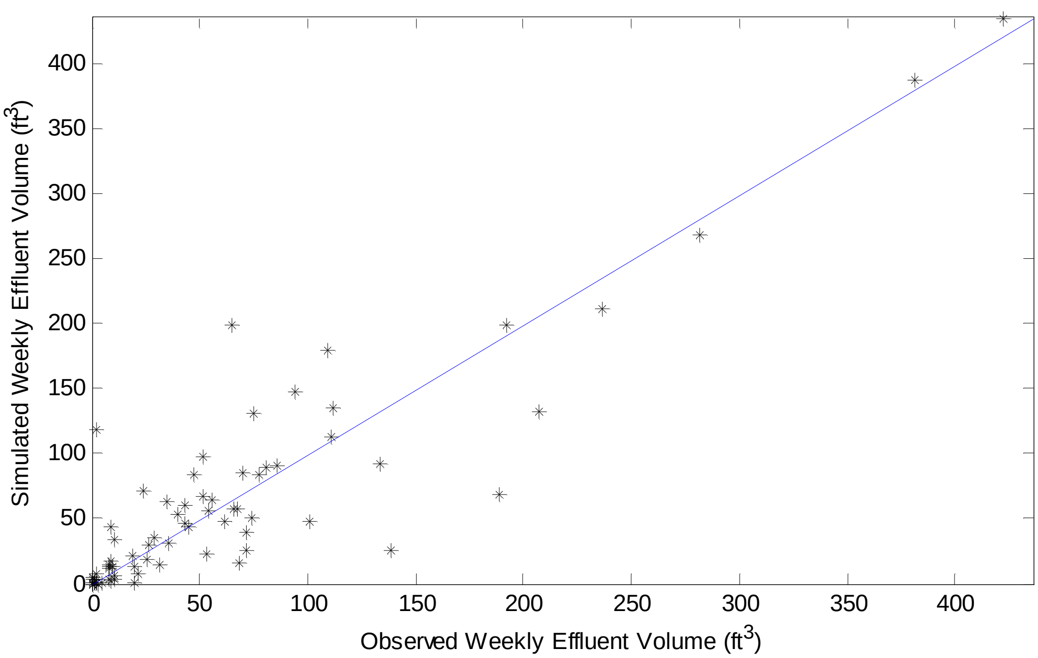

Figure 5-24 Comparison plot of simulated versus observed weekly effluent volumes (ft^3) for trial 5 of calibration.

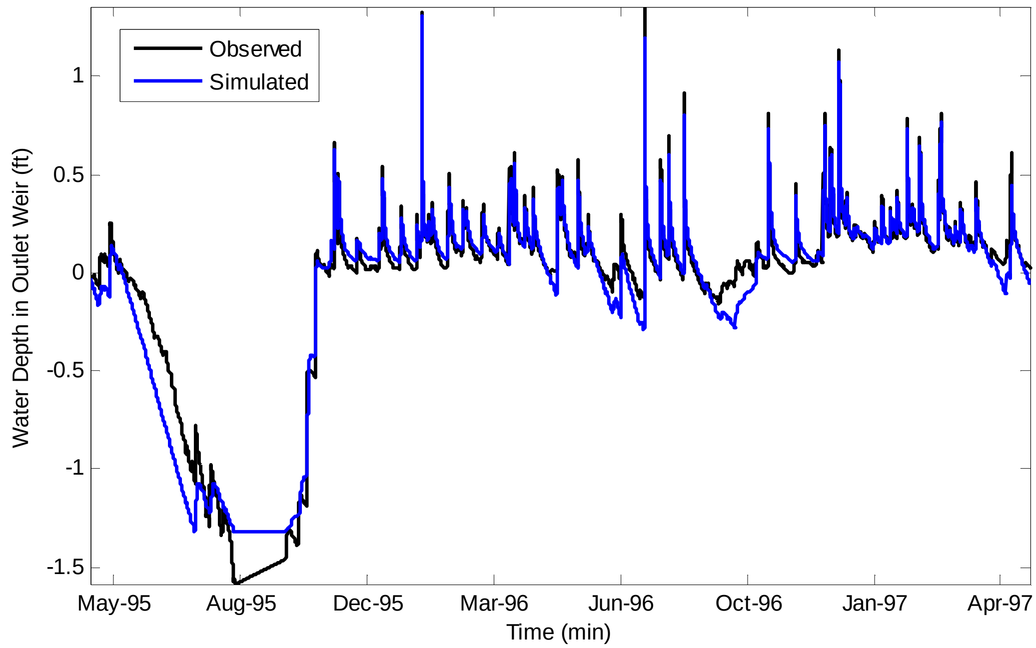

Figure 5-25 Compares the model-simulated (blue) and observed (black) wetland water depths at the outlet weir (ft) on a 1-min time interval for trial 5 of calibration.

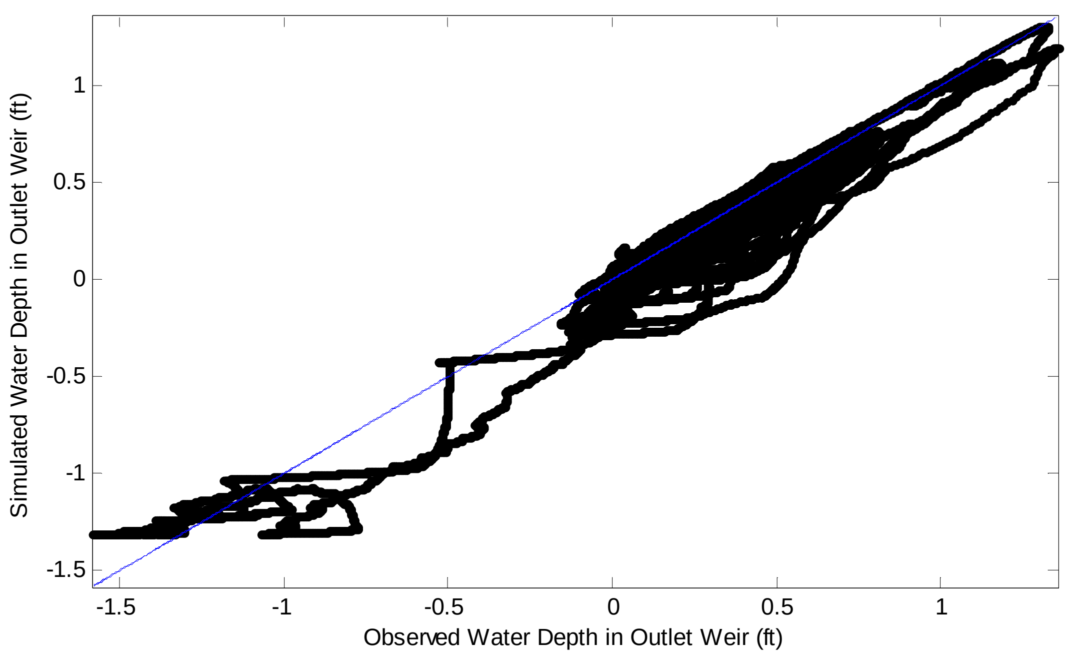

Figure 5-26 Comparison plot of simulated versus observed 1-min water depths at the outlet weir (ft) for trial 5 of calibration.

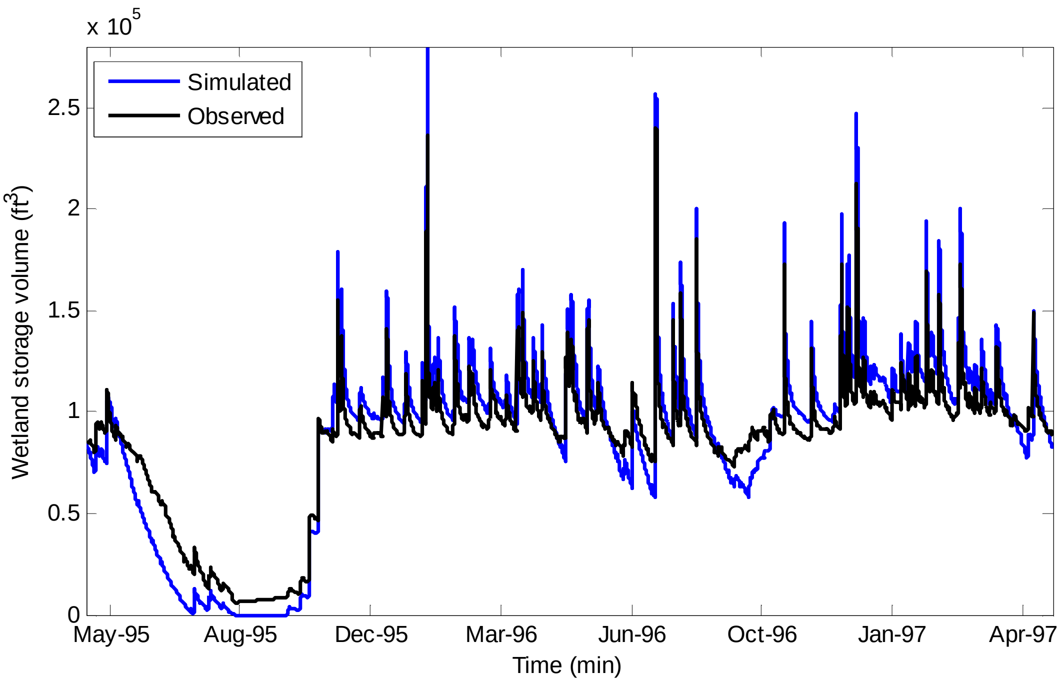

Figure 5-27 Compares the model-simulated (blue) and observed (black) 1-min wetland water storage volumes (ft^3) for trial 5 of calibration.

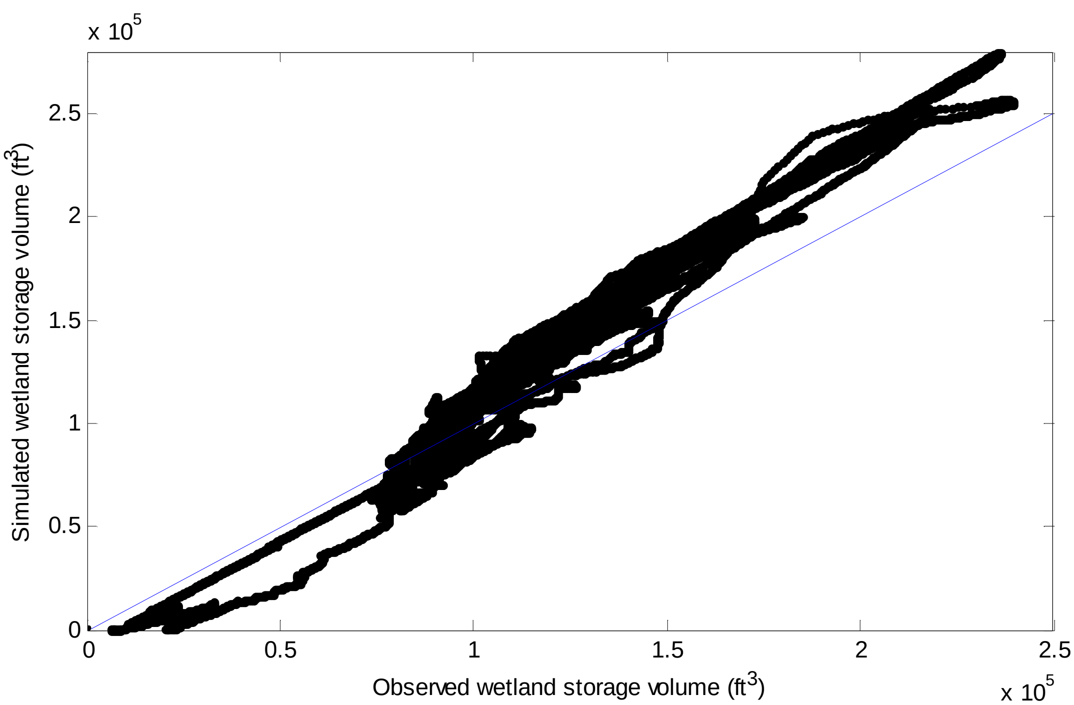

Figure 5-28 Comparison plot of simulated versus observed 1-min wetland water storage volumes (ft^3) for trial 5 of calibration.

Table 5-11 Resulting annual equivalent depths for wetland outflow, infiltration, ET, and change in storage. Annual peak and mean discharges were also included. Mean flowrates excluded zero-flows.

Trial #	Total annual outflow (in.)		Total annual infiltration (in.)		Total annual ET (in.)		Annual change in storage (in.)		Peak outflow discharge (cfs)		Mean outflow discharge (cfs)	
	Year 1	Year 2	Year 1	Year 2	Year 1	Year 2	Year 1	Year 2	Year 1	Year 2	Year 1	Year 2
1	11.7	26.6	0	0	13.8	12.2	1.11	1.59	8.46	7.92	0.0881	0.119
2	11.2	25.2	0	0	19.9	28.6	0.54	1.97	8.46	7.57	0.0880	0.127
3	11.0	24.6	0	0	22.5	35.7	0.18	2.26	8.46	7.13	0.0897	0.133
4	11.0	24.6	0.0001	0.00012	22.5	35.7	0.18	2.26	8.46	7.13	0.0897	0.133
5	10.4	23.6	8.69	11.8	20.7	35.3	-0.07	2.46	8.45	6.75	0.0864	0.137

Table 5-12 Hydrologic calibration trials and corresponding statistics for (1) weekly outflow volumes (m^3), (2) 1-min water depth in the outlet weir (ft), (2) 1-min wetland storage volume (ft^3), and (3) 1-min effluent rates (cfs).

Trial #	Weekly effluent volume (ft3)				Water Depth at Weir (ft)				Wetland Storage Volume (ft^3)				Outflow (cfs)			
	$\bar{e}$	$RMSE$	$\bar{e}/\bar{Y}$	$\bar{S}_e/\bar{S}_y$	$\bar{e}$	$RMSE$	$\bar{e}/\bar{Y}$	$\bar{S}_e/\bar{S}_y$	$\bar{e}$	$RMSE$	$\bar{e}/\bar{Y}$	$\bar{S}_e/\bar{S}_y$	$\bar{e}$	$RMSE$	$\bar{e}/\bar{Y}$	$\bar{S}_e/\bar{S}_y$
1	6.90	33.3	0.154	0.445	0.114	0.217	0.077	0.422	11762	13088	0.139	0.403	0.011	0.058	0.160	0.192
2	4.28	32.3	0.095	0.432	0.069	0.155	0.046	0.300	8170	11466	0.096	0.353	0.007	0.061	0.101	0.202
3	3.15	32.0	0.070	0.429	0.025	0.108	0.017	0.210	5246	12203	0.062	0.376	0.005	0.065	0.076	0.215
4	3.15	32.0	0.070	0.429	0.025	0.108	0.017	0.210	5246	12203	0.062	0.376	0.005	0.065	0.076	0.215
5	1.06	31.5	0.024	0.422	-0.012	0.114	-0.008	0.221	2517	13400	0.030	0.413	0.002	0.070	0.029	0.230

Table 5-13 Summary of trials 1-5 including the changes made, the reasons for each change, and the corresponding results.

Trial #	Parameter change	Reason	Trial result
1	No changes	Initial trial with all Jordan (2013) inputs	Simulated outputs match trends of observed values but are biased. ET annual depth low and wetland storage and outflow volumes and rates high.
2	Allow hourly ET simulation for all hourly rainfall depths less than or equal to 0.00083 ft.	To increase ET depths and indirectly decrease OUT depths.	Annual ET depths increased. Model still overpredicted storage and outflow volumes and rates. Goodness-of-fit statistics improved for all outputs.
3	Increase LAI from 6.5 to 10	To increase ET depths and indirectly decrease OUT depths.	ET depths increased and OUT depths decreased. Model still overpredicted storage and outflow volumes and rates. Goodness-of-fit statistics improved for all outputs.
4	Increase K_V from 0 to 2.38×10^{-8} ft/d	To increase wetland infiltration and to reduce model overprediction of effluent volumes and rates, and wetland storage.	No significant changes.
5	Increase K_V from 2.38×10^{-8} to 2.38×10^{-3} ft/d	To increase wetland infiltration to reduce model overprediction of effluent volumes and rates, and wetland storage.	Goodness-of-fit statistics improved, resulting in $\bar{e}/\bar{Y}$ and $\bar{S}_e/\bar{S}_y$ values for all outputs indicative of a good fit.

5.3.1.6 Hydrologic calibration results and discussion

While the model predicted observed hydrologic values fairly well (all $\overline{e}/\overline{Y}$ magnitudes below 0.05 and $\overline{S}_e/\overline{S}_y$ values below 0.500), a number of discrepancies were apparent between the simulated and observed hydrologic time series. These problems likely stemmed from the discretization of the Barnstable 1 wetland into a 13-cell system (see Figure 5-11) and the lack of knowledge of the wetland internal flowpath. The current section discusses these incongruities in detail.

Firstly, it was noted that the simulated outflow hydrographs exhibited lower peaks and higher falling limbs than corresponding observed outflow hydrographs. As documented in Table 5-11, the final model underpredicted 1-min peak effluent rates for years 1 and 2 (8.45 and 6.75 cfs) as compared to corresponding observed 1-min peaks of 8.80 and 9.27 cfs. Conversely, simulated mean flowrates for each year (0.0864 and 0.137 cfs) were overpredicted by the model compared with the observed values of 0.0750 and 0.0123. Additionally, Figure 5-21 shows that the model systematically underpredicted peaks. As mentioned earlier, the model also produced outflow hydrographs with larger flowrates on the falling limb. An example of this trend was plotted in Figure 5-29, which was zoomed in to better show the model hydrograph behavior. This systematic error in the model suggests that simulated peak water levels at the outlet weir were less than corresponding observed levels, which can be seen in Figure 5-25. These reduced water depth peaks produced smaller outflow rate peaks and outflow hydrographs with more gradual falling limbs. This problem likely stemmed from the inability of the model to fully characterize the internal wetland elevations and flowpath(s) via the 13-cell design shown in Figure

5-11. Additional sources of error within the model could be due to the methods used to simulate roughness coefficients within each wetland cell based on vegetation type/inclusion.

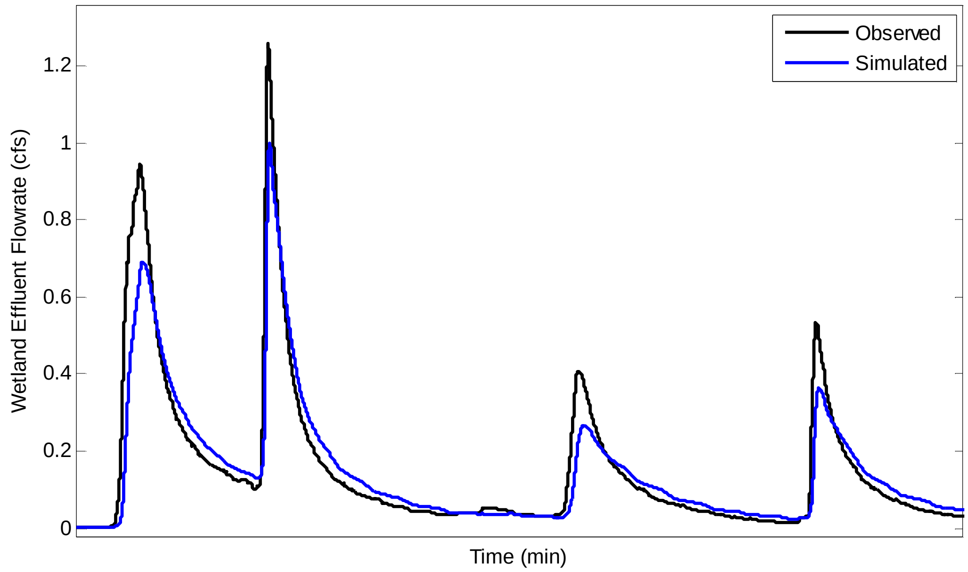

Figure 5-29 Zoomed-in view of simulated (blue) and observed (black) 1-min outflow rates to show the difference in predicted and observed hydrographs. Note that simulated peaks are smaller than corresponding peaks and that simulated hydrographs produce larger flowrates in the falling limb than observed hydrographs.

In addition to the errors seen in the model outflow rates and water depths at the outlet weir, errors were also noted in the simulated 1-min wetland storage volumes. As shown in Figure 5-27, there were a number of local biases in the simulated wetland storage volumes. The model appeared to overpredict high storages and underpredict low storages. This error trend in the predicted 1-min storage volumes was, again, indicative of the inability of the model to fully characterize the flowpath and bottom elevations (see Figure 5-1) of the Barnstable 1 wetland due to the large uncertainty associated with these measurements. In order to address this problem, the effects of flowpath and cell-size were further investigated in Sections 5.3.3and 5.3.4.

5.3.2 Calibration of Barnstable 1 water quality

All hydrologic inputs calibrated in Section 5.3.1 were held constant during water quality calibration in order to evaluate the performance of the model in predicting observed weekly effluent water quality concentrations (i.e., TSS, NH_4^+, and NO_3^-). The mean, the minimum, the maximum, and the standard deviation for effluent TSS, NH_4^+, and NO_3^- concentrations reported by Jordan (2013) are summarized in Table 5-14. It was noted that observed weekly influent NH_4^+ concentrations, with a median value of 0.152 mg/L, were often less than or equal to effluent NH_4^+ concentrations, which had a median of 0.155 mg/L. This trend suggests that the Barnstable 1 actually generated NH_4^+ at times. The model did not contain a mechanism for actively generating NH_4^+, but rather specified a minimum background concentration limit for each pollutant, which did not allow TSS, NH_4^+, and NO_3^- concentrations in the wetland to be reduced below corresponding user-specified values. Therefore, the model could not fully capture this NH_4^+ generation behavior in the wetland. A similar, but weaker trend was also seen in TSS weekly concentrations, which had respective influent and effluent median concentrations of 112 and 106 mg/L. As a result of this wetland behavior, the inclusion of NH_4^+ and TSS generation mechanisms would be suggested in any future versions of the model. Each water quality calibration trial was discussed in detail in the following subsections and resulting goodness-of-fit criteria and statistics from each trial are summarized in Table 5-15 and Table 5-16.

Table 5-14 Influent and effluent water quality statistics for weekly TSS, NH_4^+, and NO_3^- concentrations computed in the current study from data provided by Jordan (2013).

	Observed weekly water quality statistics									
	Mean		Median		Minimum		Maximum		Standard deviation	
	IN	OUT	IN	OUT	IN	OUT	IN	OUT	IN	OUT
TSS (mg/L)	175	153	112	106	23.9	6.35	583	846	151	161
NH4 (mg/L)	0.170	0.222	0.152	0.155	0.079	0.070	0.438	1.57	0.071	0.22
NO3 (mg/L)	0.303	0.218	0.176	0.099	0.003	0.000	1.19	1.33	0.326	0.303

5.3.2.1 Trial 1

Initial water quality related input parameters used in trial 1 were estimated via literature values as well as Barnstable 1 characteristics and are summarized in Table 5-5. The results from trial 1 showed that the model underpredicted weekly effluent TSS and NH_4^+ concentrations with large respective $\bar{e}/\bar{Y}$ values of -0.903 and -0.794, repsectively. Conversely, the model predicted weekly NO_3^- concentrations with very little bias as indicated by its $\bar{e}/\bar{Y}$ of -0.01. None of the water constituents, however, followed the trends of the observed weekly data well as evidenced by the resulting $\bar{S}_e/\bar{S}_y$ values of 1.32 for TSS, 1.29 for NH_4^+, and 0.877 for NO_3^-. The underprediction of simulated TSS concentrations was likely due to an incorrect initial TSS particle diameter estimate, while the errors associated with NH_4^+ and NO_3^- concentrations could be due to a number of factors including incorrect nitrification K_{NIT} and denitrification K_{DNT} rate constants, dissolved oxygen levels, and wetland NH_4^+ and NO_3^- background levels. All of these factors were addressed in the water quality calibration trials in the current section. Results from trial 1 are shown graphically both against time with corresponding observed data and in comparative plots with observed data on the x-axis and simulated results of the y-axis for weekly effluent TSS concentrations in Figure 5-30 and Figure 5-31, NH_4^+ concentrations in Figure 5-32 and Figure 5-33, and NO_3^+ concentrations in Figure 5-34 and Figure 5-35.

These poor goodness-of-fit statistics should be expected given that weekly composite influent concentrations were predicted using corresponding 1-min concentrations entering the wetland. Without water quality data at a finer time scale, it was difficult to accurately model variation in water concentrations in the wetland at the 1-min time scale. Therefore, large errors were associated with the resulting summed weekly effluent water quality concentrations.

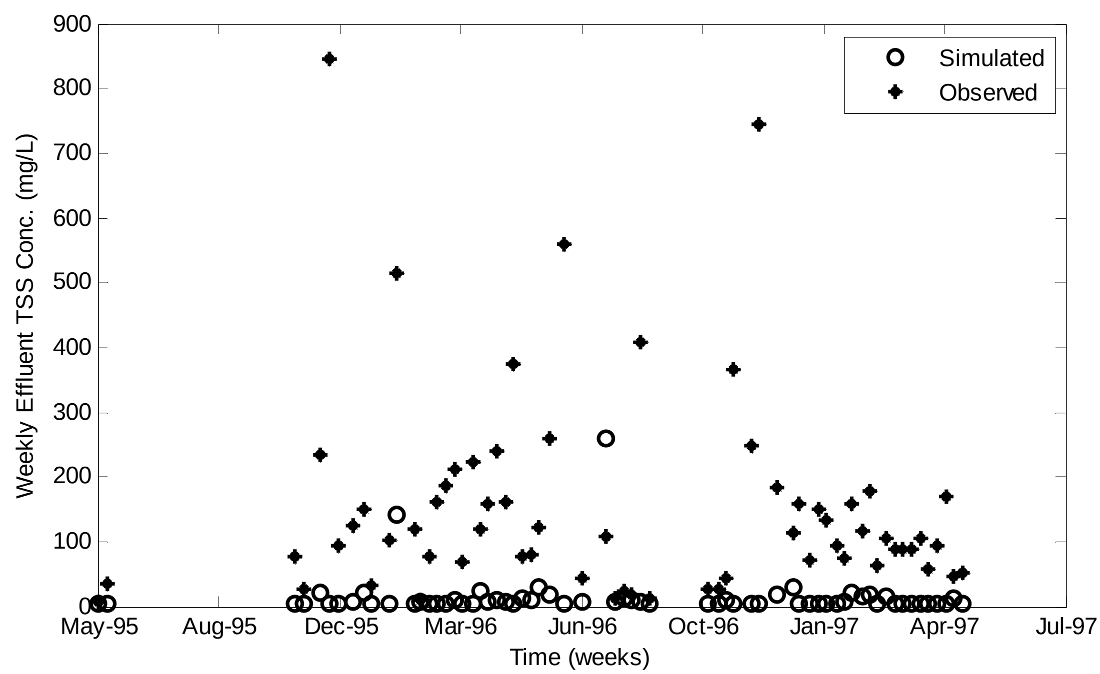

Figure 5-30 Comparison plot of simulated (open circles) and observed (stars) weekly effluent TSS concentrations (mg/L) for trial 1.

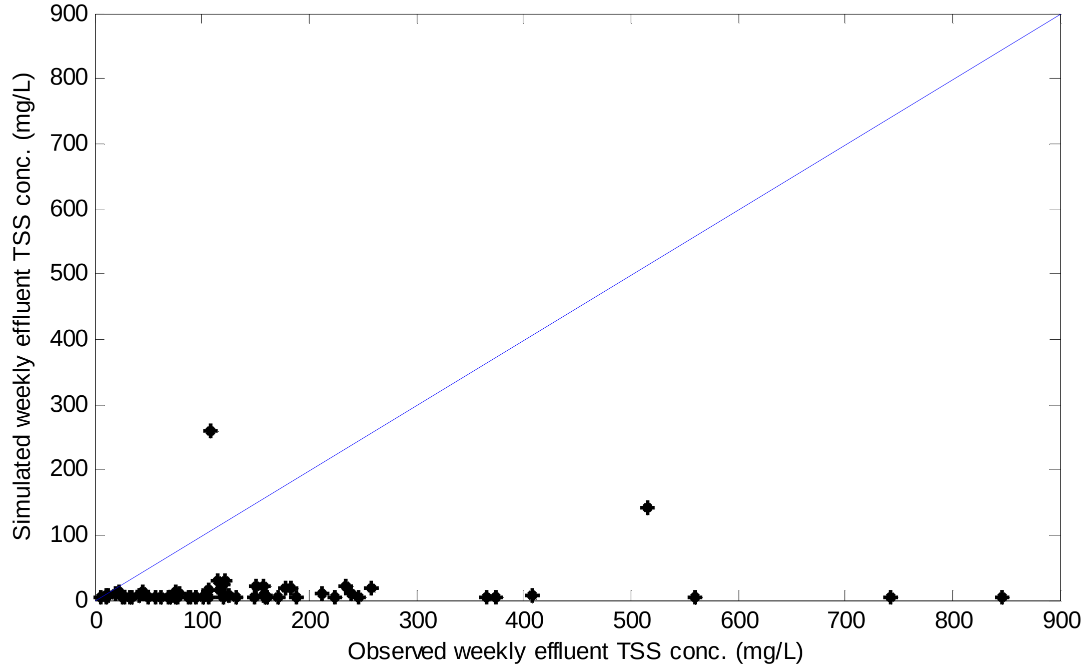

Figure 5-31 Plot of simulated versus observed weekly effluent TSS concentrations with units of mg/L for trial 1. A 1:1 line is drawn for reference.

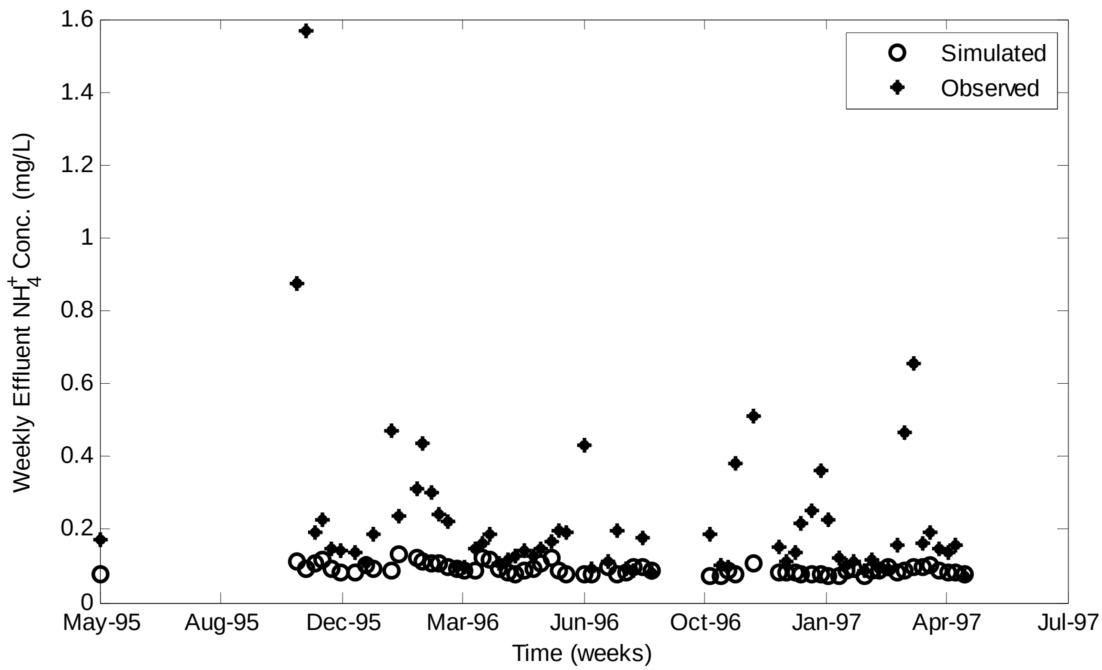

Figure 5-32 Comparison plot of simulated (open circles) and observed (stars) weekly effluent NH_4^+ concentrations (mg/L) for trial 1.

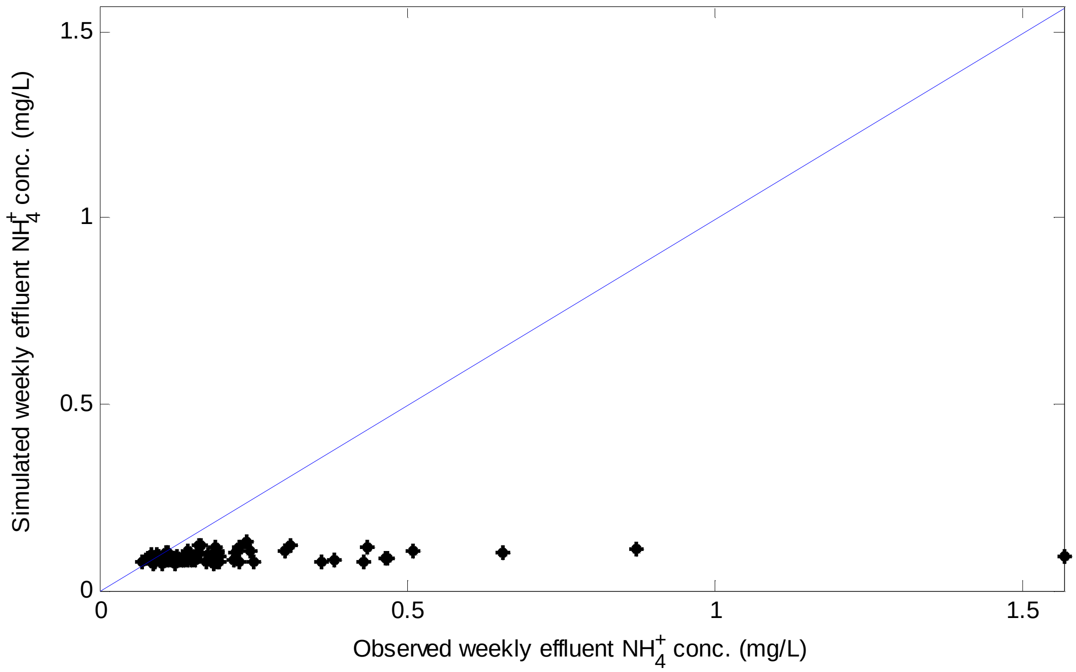

Figure 5-33 Plot of simulated versus observed weekly effluent NH_4^+ concentrations with units of mg/L for trial 1. A 1:1 line is drawn for reference.

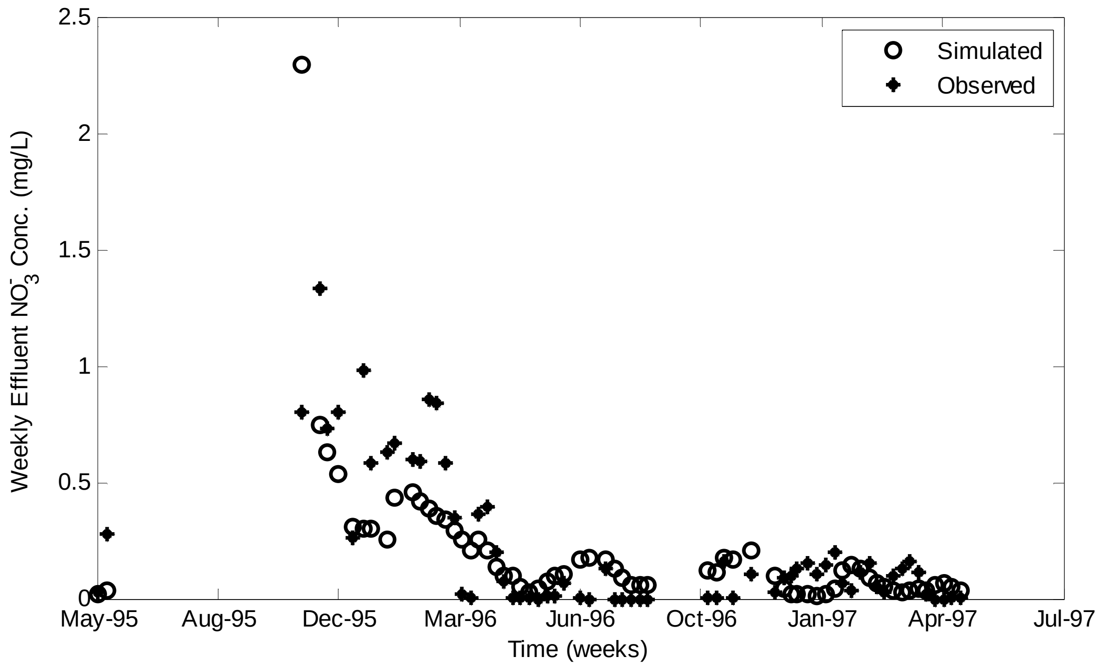

Figure 5-34 Comparison plot of simulated (open circles) and observed (stars) weekly effluent NO_3^- concentrations (mg/L) for trial 1.

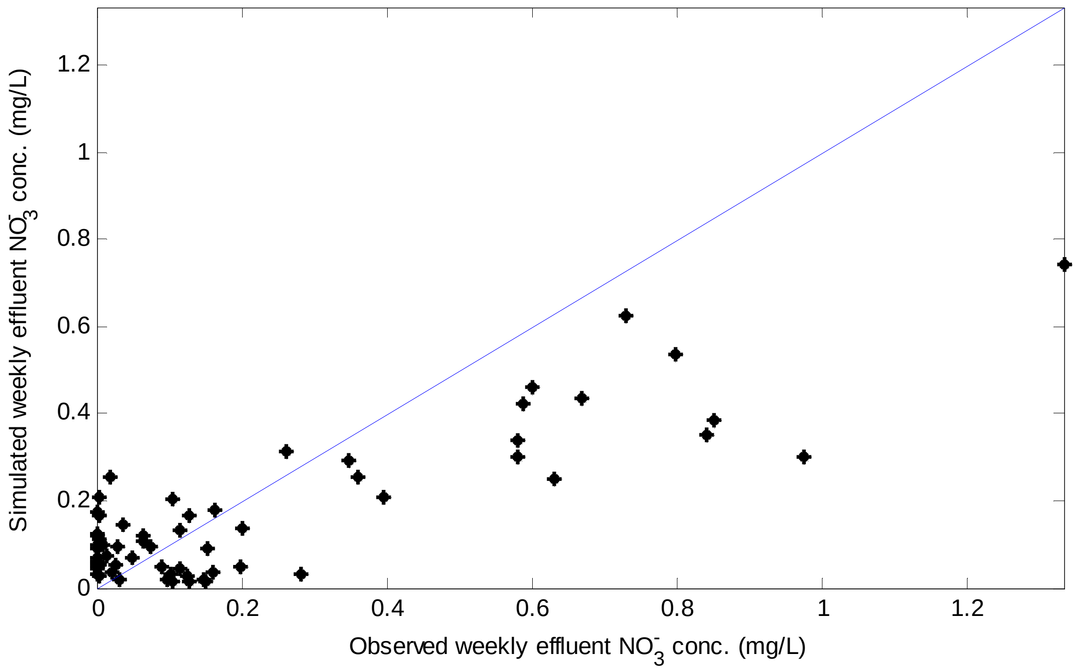

Figure 5-35 Plot of simulated versus observed weekly effluent NO_3^- concentrations with units of mg/L for trial 1. A 1:1 line is drawn for reference.

5.3.2.2 Trial 2

In order to improve the accuracy of simulated weekly concentrations, the TSS particle diameter was decreased from $2x10^{-6}$ m to $2x10^{-7}$ m. Decreasing the TSS particle size should decrease the TSS settling velocity and, therefore, decrease the wetland trap efficiency, which in turn, would increase the amount of TSS leaving the wetland through the outlet. Resulting simulated weekly TSS concentrations improved in fit with $\bar{e}/\bar{Y}$ and $\bar{S}_e/\bar{S}_y$ values of -0.196 and 1.16, and a mean effluent concentration of 123 mg/L, which was closer to the observed mean of 153 mg/L. While the simulated TSS concentrations were less biased, they still did not follow the trend of the corresponding observed weekly concentrations well, which, was likely due to the complexity of TSS behavior in the actual Barnstable 1 wetland. As mentioned earlier, observed Barnstable 1 weekly influent and effluent composite concentrations were fairly similar (see Table 5-14). Therefore, in addition to settling, TSS concentrations in the actual Barnstable 1 wetland were also likely controlled by processes such as resuspension, and TSS generation via vegetation decay and algae growth/decay. Because the current model only simulated TSS settling and an enforced background TSS level of 3 mg/L, the resulting simulated TSS concentrations could not follow the same trend as the observed data.

The poor $\overline{S}_e / \overline{S}_y$ of 1.16 and $\overline{e} / \overline{Y}$ of -0.196 were thought to be due to a number of factors including the discrepancies that existed between the simulated and actual flowpaths in the Barnstable 1 wetland, TSS resuspension, and TSS generation via vegetation decay and algae growth/decay. A shorter model-defined flowpath would have allowed for less settling and consequently higher effluent TSS concentrations. The defined model flowpath (see Figure 5-11) could have also oversimplified the true path or paths of water through the Barnstable 1 wetland. Additionally, flowpaths may change depending on water levels in the wetland, adding to the discrepancy between the simple model-defined flowpath and the actual, dynamic flowpath. As mentioned earlier, resuspension and TSS generation within the Barnstable 1 wetland could have also contributed to the high effluent TSS concentrations observed. Finally, the use of a sole mean TSS particle diameter may oversimplify the actual influent TSS particle diameter distribution. In reality TSS particles with differing diameters would settle out at different points within the wetland due to their varying settling velocities. The current model assumed all TSS particles to have the same diameter and, in turn, the same settling velocity. As a result of all of these factors, simulated weekly effluent TSS concentrations did not predict corresponding observed values well.

5.3.2.3 Trial 3

The simulated weekly effluent NH_4^+ concentrations were dependent on a number of inputs including K_{NIT}, $NH4_o$, $PMAX$, DO_o, DO_{in}, and $T_{w(o)}$. Nitrification, the transformation of NH_4^+ to NO_3^-, was simulated to occur in the water column only when DO concentrations were greater or equal to 2 mg/L. The model generates oxygen via surface aeration in every wetland cell and via photosynthesis only in wetland cells with submerged vegetation. However, neither DO_{in} nor DO_o should have a large impact on DO levels in the Barnstable 1 wetland design, as it is very shallow, allowing for rapid surface aeration, and because photosynthesis occurs in the three deepest wetland cells (see Table 5-6). This large production of oxygen within the wetland implies that the wetland surface water should generally be saturated, which was found to be true as the simulated mean daily dissolved oxygen concentration in the wetland was found to equal 10.9 mg/L. Given these internal wetland oxygen conditions, the input parameter K_{NIT} was first targeted in order to improve model prediction of weekly effluent NH_4^+ concentrations. Therefore, in trial 5, K_{NIT} was decreased from 0.01 to 0.001 hr^{-1} in an effort to increase the simulated mean weekly effluent NH_4^+ concentration of 0.046 mg/L up to the observed effluent mean of 0.222 mg/L.

This decrease in K_{NIT} improved model prediction of NH_4^+ with resulting NH_4^+ $\bar{e}/\bar{Y}$ and $\bar{S}_e/\bar{S}_y$ values of -0.468 and 1.05. The simulated NH_4^+ weekly mean effluent concentration also increased from 0.046 to 0.118 mg/L, which was still slightly more than half of the observed mean of 0.222 mg/L. Simulated NO_3^- also decreased as a result of the decreased K_{NIT} value, with a mean weekly effluent concentration of 0.180 mg/L versus 0.216 mg/L in trial 4. This decrease in NO_3^- was due to the slower resulting rate of production of NO_3^- via nitrification.

5.3.2.4 Trial 4

K_{NIT} was further decreased in trial 4 from 0.001 to 0.0001 hr^{-1} in order to correct the model underprediction of effluent NH_4^+. The resulting mean simulated weekly effluent NH_4^+ concentration was 0.148 mg/L, which was still lower than the corresponding observed mean of 0.222 mg/L. This order of magnitude decrease in K_{NIT} did not sufficiently increase the simulated effluent NH_4^+ concentrations so as to match observed values.

5.3.2.5 Trial 5

In trial 5, K_{NIT} was decreased again from 0.0001 to 0.00001 hr^{-1} to reduce nitrification in the wetland as to allow for higher effluent NH_4^+ concentrations. This change did not significantly affect NH_4^+ goodness-of-fit statistics, and only increased the mean simulated effluent NH_4^+ concentration from 0.148 to 0.152 mg/L, which was also equal to the median weekly influent NH_4^+ concentration (see Table 5-14). Based on these results, the input parameter K_{NIT} appeared to become less sensitive as it was decreased.

This apparent insensitivity of K_{NIT} within the model did not reflect the complex behavior of NH_4^+ in the actual Barnstable 1 wetland. In reality, the behavior of NH_4^+ in wetlands is very complex and is dependent on a number of factors such as wetland chemistry, microbial populations, and seasonality. Wetlands can also generate NH_4^+ via ammonification and resuspension (Kadlec and Knight 1996; USEPA 2000). As discussed earlier in this section, the model contains no mechanism by which to actively generate NH_4^+ within the wetland, but rather enforces a minimum background NH_4^+ concentration $NH4_o$ within the wetland. Therefore, within the model, when $NH4_o$ is set equal to zero, effluent NH_4^+ concentrations can only be less than or equal to corresponding influent concentrations. As a result, as input K_{NIT} values approach zero, effluent NH_4^+ concentrations approach corresponding influent NH_4^+ concentrations. However, in the case of the Barnstable 1 wetland, observed effluent concentrations were often greater than corresponding influent concentrations. Given the structure of the current model, simulated NH_4^+ concentrations could fully characterize the behavior of observed concentrations.

Another source of this discrepancy could also be due to the influent weekly data used, which were assumed constant over each week of record. These constant influent values did not capture the variation in the 1-min NH_4^+ concentrations that was most likely present in the actual wetland inflow. This lack of variation in influent NH_4^+ concentrations would then be transferred to corresponding effluent NH_4^+ concentrations, resulting in more constant simulated concentrations. As a result of all discrepancies exiting between the simulated and actual Barnstable 1 wetland system, weekly simulated effluent NH_4^+ concentrations did not follow the same trend as observed values as shown in Figure 5-36.

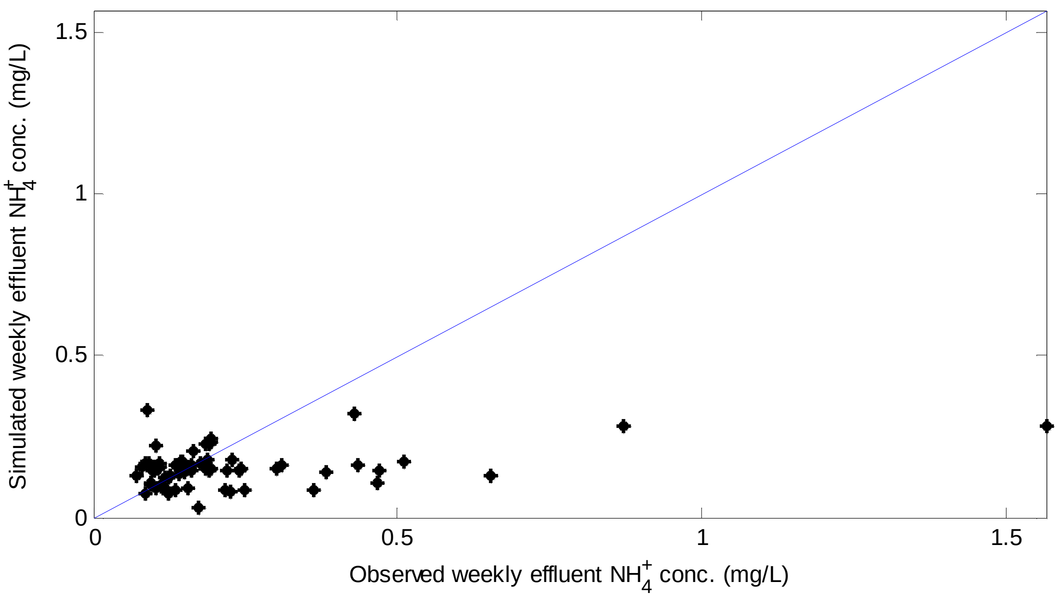

Figure 5-36 Plot of simulated versus observed weekly effluent NH_4^+ concentrations with units of mg/L from trial 5. A 1:1 line is drawn for reference.

5.3.2.6 Trial 6

In an effort to increase simulated effluent NH4+ concentrations, influent DO (DO_{in}) concentrations were decreased down to 0 mg/L with the idea of reducing internal wetland DO levels below 2 mg/L, which would, in turn, disable nitrification in the model. However, this change did not reduce wetland DO levels below 2 mg/L due to the large DO production within the Barnstable 1 wetland, which produced sufficient DO in the wetland to counter the reduction in DO_{in}. Therefore, this reduction in influent DO levels did not significantly change simulated NH_4^+ effluent concentrations.

5.3.2.7 Trial 7

In trial 7, the background NH_4^+ concentration $NH4_o$ was increased from 0 to 0.070 mg/L in an effort to increase effluent NH_4^+ concentrations. Additionally, the influent DO concentration was increased back to its original value of 7.5 mg/L. While USEPA (2000) reported a typical $NH4_o$ of 0.20 to 1.5 mg/L for constructed wetlands, a value of 0.07 mg/L was chosen for trial 9 as it was the minimum observed mean weekly effluent NH_4^+ concentration. This increase in $NH4_o$, however, did not affect predicted effluent NH_4^+ concentrations significantly (see Table 5-15 and Table 5-16), suggesting again that NH_4^+ behavior in the Barnstable 1 wetland was more complex than the simulated NH_4^+ behavior.

5.3.2.8 Trial 8

In order to reduce the negative model bias in effluent NH_4^+ concentrations, the $NH4_o$ was increased from 0.07 to 0.20 mg/L. This change resulted in a $\bar{e}/\bar{Y}$ value of 0.008 and slightly decreased $\bar{S}_e/\bar{S}_y$ value of 0.972. Based on the discrepancies between the model and the actual Barnstable 1 wetland system, the resulting simulated NH_4^+ values were assumed to be sufficiently calibrated.

5.3.2.9 Trial 9

Weekly NO_3^- concentrations were targeted next for calibration. Denitrification in the model was simulated in the anoxic sediments of all wetland cells with emergent vegetation (USEPA 2000) as well as in the water column of cells with water DO levels less than 2 mg/L. Simulated weekly NO_3^- concentrations were, therefore, a direct function of K_{DNT}, $NO3_o$, and an indirect function of all factors influencing corresponding simulated NH_4^+ concentrations. The denitrification rate constant K_{DNT} as well as the wetland flowpath dictated this reaction. In trial 10, the negative model bias in weekly effluent NO_3^- concentrations was addressed by decreasing K_{DNT} from 0.0208 to 0.002 hr^{-1}. Results from trial 10 produced a mean weekly effluent NO_3^- concentration of 0.303 mg/L, which was about 0.8 mg/L greater than the observed mean of 0.218 mg/L (see Table 5-14). This large increase in predicted effluent NO_3^- concentrations showed that K_{DNT} was much more sensitive than K_{NIT}. Because observed effluent NO_3^- concentrations were less than influent concentrations, the exclusion of a NO_3^- generation mechanism within the model did not play a large role in model prediction of NO_3^- as it did with TSS and NH_4^+. Therefore, within the Barnstable 1 wetland, the dominating processes controlling NO_3^- effluent concentrations appeared to be nitrification and denitrification.

5.3.2.10 Trial 10

Given that simulated weekly effluent NO_3^- concentrations were still greater than corresponding observed values, K_{DNT} was increased from 0.002 to 0.01 hr^{-1} in trial 10 in order to reduce simulated effluent NO_3^- concentrations. This change improved goodness-of-fit statistics, producing a $\bar{e}/\bar{Y}$ of -0.049 and a $\bar{S}_e/\bar{S}_y$ of 0.900. The resulting simulated mean weekly effluent NO_3^- concentration was 0.207 mg/L, which was very close the observed value of 0.215 mg/L.

5.3.2.11 Trial 11

In order to further decrease simulated weekly effluent NO_3^- concentrations to better match corresponding observed values, K_{DNT} was increased from 0.005 to 0.0085 hr^{-1} in trial 11. This change resulted in NO_3^- $\bar{e}/\bar{Y}$ and $\bar{S}_e/\bar{S}_y$ values of 0.001 and 0.923, and a mean simulated weekly effluent NO_3^- concentration of 0.218 mg/L. Discrepancies between simulated and observed NO_3^- concentrations were hypothesized to be due to the limitations of the provided weekly data and differences in the modeled and actual wetland flowpaths. The model did not experience the same problems in predicting NO_3^- concentrations as it did with NH_4^+ concentrations. This difference in prediction success was due to the fact that effluent NO_3^- concentrations were generally lower than corresponding influent concentrations. Therefore, the issues of resuspension and wetland generation did not play as large a role in effluent NO_3^- concentrations as in effluent NH_4^+ concentrations. Based on these results, trial 13 was the final trial in the water quality calibration of the model. The final TSS, NH_4^+, and NO_3^- plots resulting from trial 13 are shown in Figure 5-37 and Figure 5-38 for TSS, in Figure 5-39 and Figure 5-40 for NH_4^+, and in Figure 5-41 and Figure 5-42 for NO_3^-.

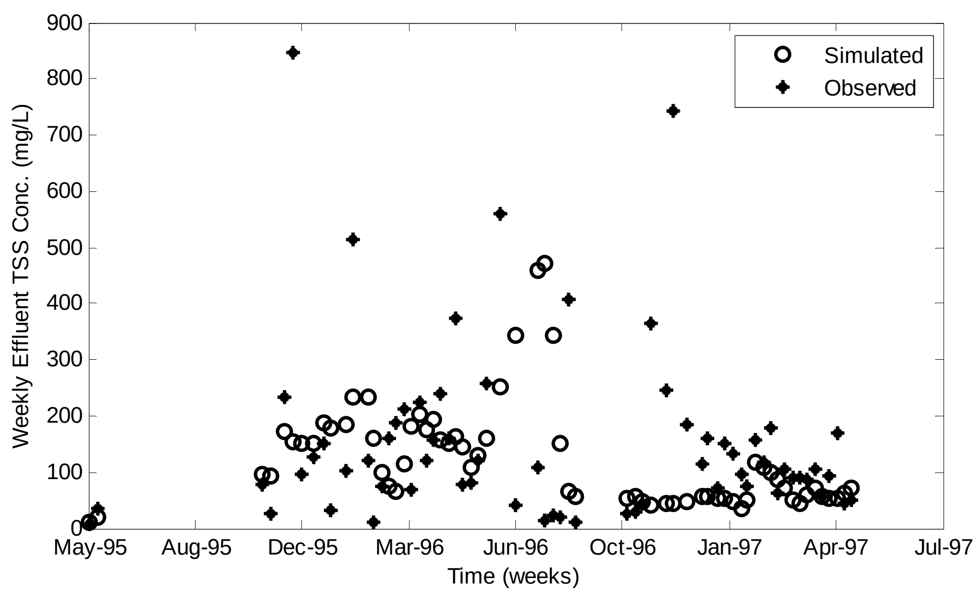

Figure 5-37 Comparison plot of simulated (open circles) and observed (stars) weekly effluent TSS concentrations (mg/L) for trial 11.

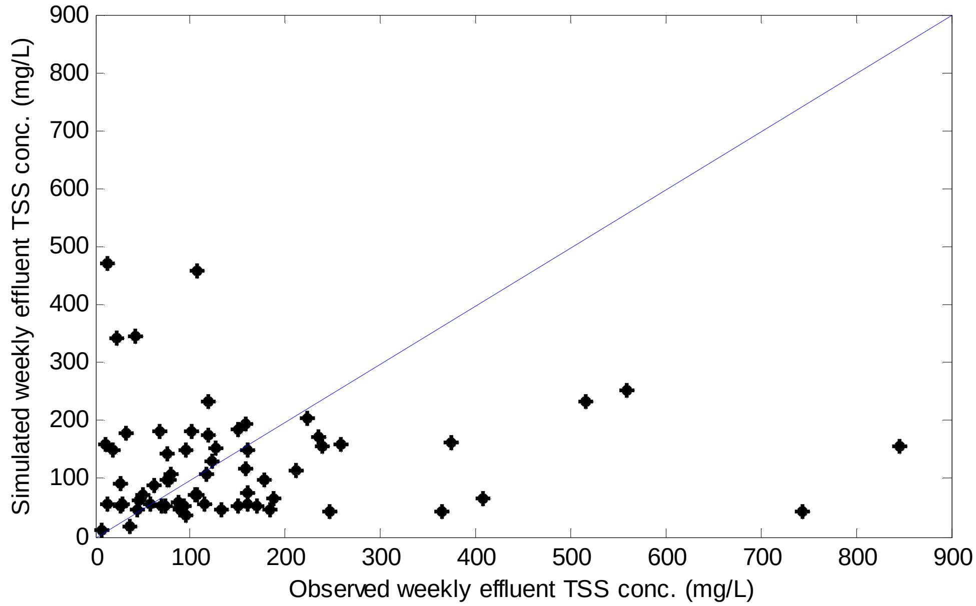

Figure 5-38 Plot of simulated versus observed weekly effluent TSS concentrations with units of mg/L from trial 11. A 1:1 line is drawn for reference.

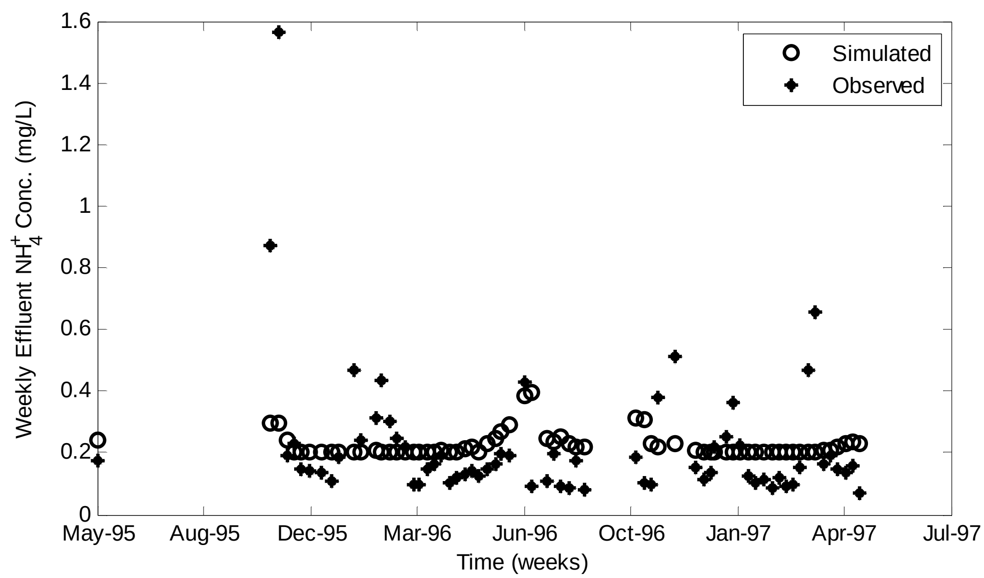

Figure 5-39 Comparison plot of simulated (open circles) and observed (stars) weekly effluent NH_4^+ concentrations (mg/L) for trial 11.

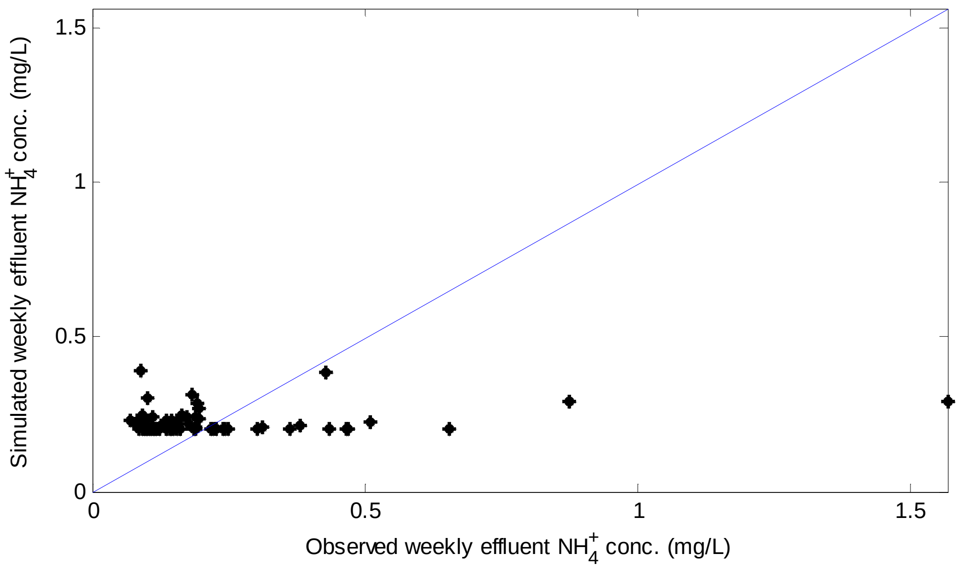

Figure 5-40 Plot of simulated versus observed weekly effluent NH_4^+ concentrations with units of mg/L from trial 11. A 1:1 line is drawn for reference.

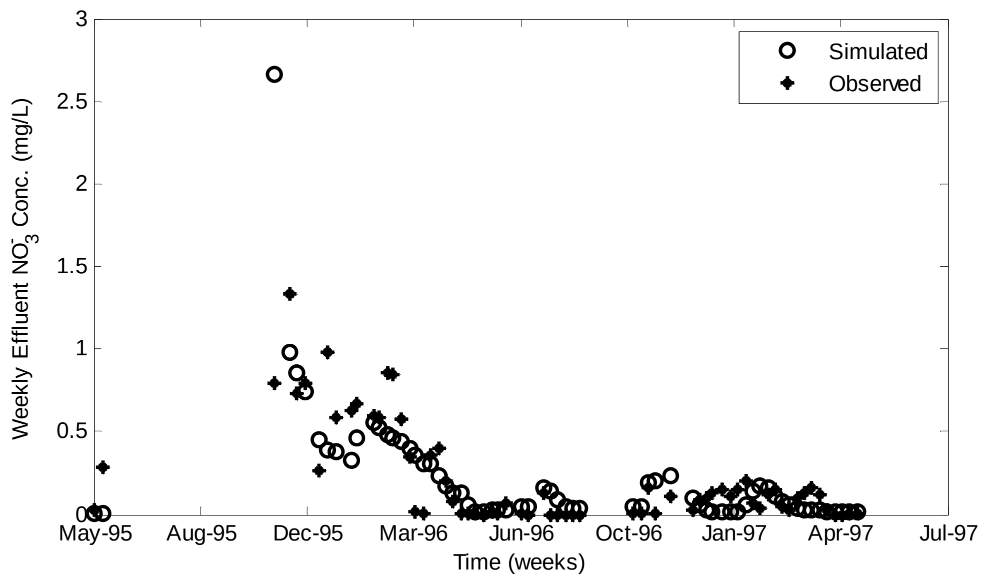

Figure 5-41 Comparison plot of simulated (open circles) and observed (stars) weekly effluent NO_3^- concentrations (mg/L) for trial 11.

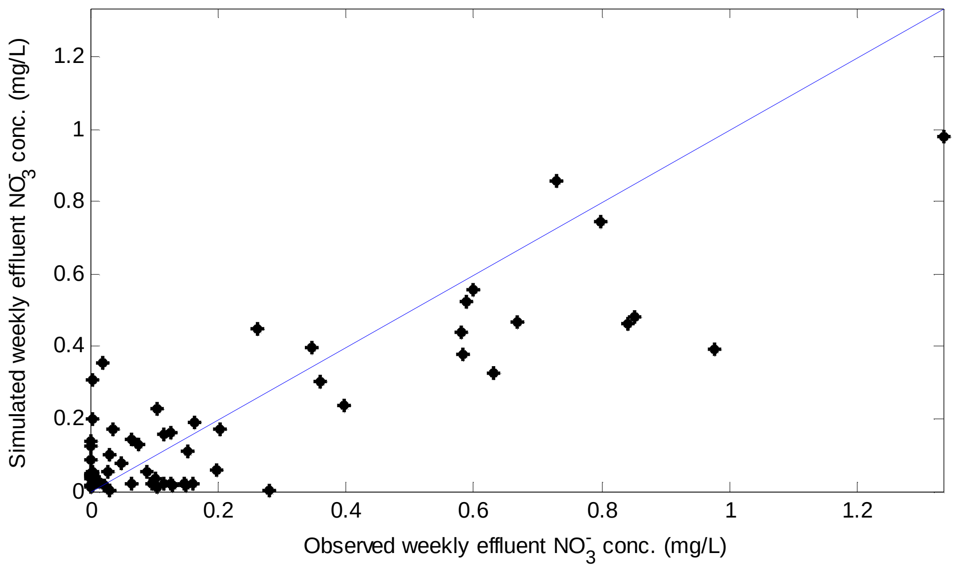

Figure 5-42 Plot of simulated versus observed weekly effluent NO_3^- concentrations with units of mg/L from trial 11. A 1:1 line is drawn for reference.

5.3.2.12 Water quality calibration results and discussion

While the model was able to match observed weekly effluent NO_3^- reasonably well, it did not accurately predict weekly effluent TSS and NH_4^+ concentrations well. The poor model fits for effluent TSS and NH_4^+ concentrations were hypothesized to be due to the complex behavior of TSS and NH_4^+ within the Barnstable 1 wetland, discrepancies between the simulated and actual wetland flowpaths, and the large time scale of the available water quality data. While the model assumed that the dominating water quality processes within any given wetland were TSS settling, nitrification, and denitrfication, the Barnstable 1 data showed that resuspension, plant decay, algae growth and death, and ammonification may also play important roles in wetland performance. These processes would, therefore, be incorporated in future versions of the current model. Goodness-of-fit statistics for simulated weekly effluent TSS, NH_4^+, and NO_3^- for all trials are shown in Table 5-15. Additionally, the mean, maximum, minimum, and standard deviations for both simulated and observed weekly effluent concentrations of all constituents are summarized in Table 5-16.

Table 5-15 Trials and corresponding statistics for calibration of model water parameters through the use of weekly composite concentrations of (1) TSS, (2) NH_4^+, and (3) NO_3^-.

Trial	TSS (mg/L)				NH4 (mg/L)				NO3 (mg/L)			
	$\bar{e}$	$RMSE$	$\bar{e}/\bar{Y}$	$\bar{S}_e/\bar{S}_y$	$\bar{e}$	$RMSE$	$\bar{e}/\bar{Y}$	$\bar{S}_e/\bar{S}_y$	$\bar{e}$	$RMSE$	$\bar{e}/\bar{Y}$	$\bar{S}_e/\bar{S}_y$
1	-134	212	-0.903	1.32	-0.176	0.281	-0.794	1.29	-0.002	0.264	-0.010	0.877
2	-29.9	185	-0.196	1.16	-0.176	0.281	-0.794	1.29	-0.002	0.264	-0.010	0.877
3	-29.9	185	-0.196	1.16	-0.104	0.229	-0.468	1.05	-0.038	0.257	-0.172	0.855
4	-29.9	185	-0.196	1.16	-0.074	0.217	-0.333	0.993	-0.057	0.252	-0.260	0.838
5	-29.9	185	-0.196	1.16	-0.070	0.216	-0.314	0.990	-0.060	0.252	-0.274	0.837
6	-29.9	185	-0.196	1.16	-0.070	0.216	-0.314	0.990	-0.060	0.252	-0.274	0.837
7	-29.9	185	-0.196	1.16	-0.067	0.215	-0.301	0.984	-0.060	0.252	-0.274	0.837
8	-29.9	185	-0.196	1.16	0.002	0.212	0.008	0.972	-0.060	0.252	-0.275	0.837
9	-29.9	185	-0.196	1.16	0.002	0.212	0.008	0.972	0.085	0.346	0.389	1.150
10	-29.9	185	-0.196	1.16	0.002	0.212	0.008	0.972	-0.011	0.271	-0.049	0.900
11	-29.9	185	-0.196	1.16	0.002	0.212	0.008	0.972	0.000	0.278	0.001	0.923

Table 5-16 The mean, maximum, minimum, and standard deviation for weekly composite simulated TSS, NH_4^+, and NO_3^- effluent concentrations from the Barnstable 1 wetland.

Trial	TSS (mg/L)				NH4 (mg/L)				NO3 (mg/L)			
	Mean	Min	Max	std	Mean	Min	Max	std	Mean	Min	Max	std
1	14.7	3.00	259	35.5	0.046	0.000	0.114	0.023	0.216	0.012	2.34	0.309
2	123	10.9	470	94.4	0.046	0.000	0.114	0.023	0.216	0.012	2.34	0.309
3	123	10.9	470	94.4	0.118	0.024	0.235	0.036	0.180	0.003	2.18	0.294
4	123	10.9	470	94.4	0.148	0.028	0.303	0.053	0.161	0.001	2.13	0.291
5	123	10.9	470	94.4	0.152	0.028	0.329	0.057	0.158	0.001	2.12	0.291
6	123	10.9	470	94.4	0.152	0.028	0.329	0.057	0.158	0.001	2.12	0.291
7	123	10.9	470	94.4	0.155	0.075	0.331	0.055	0.158	0.001	2.12	0.291
8	123	10.9	470	94.4	0.223	0.200	0.393	0.041	0.158	0.001	2.12	0.291
9	123	10.9	470	94.4	0.223	0.200	0.393	0.041	0.303	0.002	3.20	0.480
10	123	10.9	470	94.4	0.223	0.200	0.393	0.041	0.207	0.001	2.57	0.360
11	123	10.9	470	94.4	0.223	0.200	0.393	0.041	0.218	0.001	2.66	0.375

5.3.3 Effects of flowpath

Using five additional 13-cell designs, flow was simulated in order to assess the effect of flowpath on model performance. While the initial 13-cell design used for calibration was labeled as 13F1, the five additional designs were named 13F2, 13F3, 13F4, 13F5, and 13F6. The wetland design 13F2 consisted of a different secondary flowpath than 13F1. The wetland design 13F3 did not include a secondary flowpath but maintained the same primary flowpath as that of the design 13F1 shown in Figure 5-11. Designs 13F4, 13F5, and 13F6 were all variations of the design 13F3, all of which did not include secondary flowpaths and had differing main flowpaths through the wetland. Therefore, while designs 13F2 and 13F3 were compared directly to the original 13-cell design 13F1, the final three designs 13F4, 13F5, and 13F6 were compared to 13F3. The results for each 13-cell design are discussed in the following subsections. All hydrologic annual values and goodness-of-fit values were compiled in Table 5-23 and Table 5-24, and all water quality goodness-of-fit values were summarized in Table 5-25 and Table 5-26.

5.3.3.1 Wetland design 13F2

The secondary flowpath used in the wetland design 13F1 was altered in the 13F2 design (see Figure 5-11 and Table 5-6) in order to evaluate the sensitivity of model performance to changes in the secondary flowpath. This change in the secondary flowpath appeared to significantly affect the distribution of water within the simulated wetland as evidenced by the resulting 13F2 1-min wetland storage volume $\bar{e}/\bar{Y}$ of -0.069 as compared to that of the 13F1 design, which was 0.030. The annual change in storage for year 2 for design 13F2 was also 2.46 ft. versus the corresponding depth of 0.97 ft in design 13F1. As a result of this altered wetland storage, the peak outflow for year 2 was larger in design 13F2 (11.1 cfs) than in design 13F1 (6.75 cfs). These effects were most likely more notable in year 2 due to the greater amount of rainfall occurring in year 2 as well as the drought period in year 1. Drier periods in the wetland may have reduced water levels in the wetland so as to completely dry out some cells, which would further restrict the corresponding simulated wetland flowpath.

TSS effluent concentrations were also slightly higher in design 13F2 with a mean value 153 mg/L versus 147 in the 13F1 design, which indicate that design 13F2 incorporated a shorter flowpath through wetland, allowing for a shorter residence time and less TSS settling. The resulting mean weekly effluent NO_3^- concentration decreased from 0.218 mg/L in design 13F1 to 0.205 mg/L in 13F2. This decrease in effluent NO_3^- concentrations suggests that while the design 13F2 had a shorter residence time, more time relative to the design 13F1 was spent in cells with emergent vegetation, which simulated denitrification. Therefore, the decreased residence time in 13F2 appeared occur mostly in the cells with a VEG = 1 (see Figure 5-43). These water quality results suggest that the use of hydraulic residence time as the sole measure of wetland performance may not be appropriate if all wetland areas do not perform the same chemical and/or physical processes.

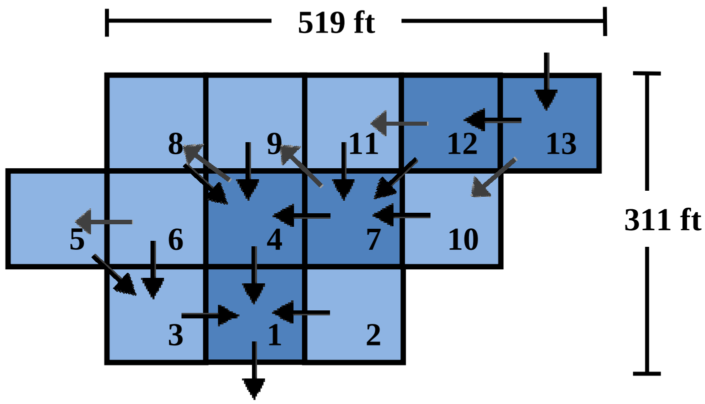

Figure 5-43 Model representation of the Barnstable 1 wetland with an altered secondary flowpath (design 13F2). Darkly-shaded cells highlight main flowpath through wetland.

Table 5-17 Barnstable 1 specifications for design 13F2 for flowpath (FID and FID2 #s), initial water depths, elevations, and vegetation descriptors for each wetland cell.

Cell	FID	FID2	Initial Water Depth (ft)	Elevation (ft)	Vegetation Descriptor
1	0	1	1.29	0	2
2	1	2	1.29	0	2
3	1	3	1.29	0	2
4	1	8	0.785	0.503	1
5	3	5	0.254	1.03	1
6	3	5	0.254	1.03	1
7	4	9	1.19	0.103	1
8	4	8	0	2.26	1
9	4	9	0.468	0.820	1
10	7	10	0.000	1.38	1
11	7	11	0.878	0.410	1
12	7	11	0.238	1.05	1
13	12	10	0	1.87	1

5.3.3.2 Wetland Design 13F3

The secondary flowpath in the original wetland design 13F1 was removed to make the design 13F3 (see Figure 5-44). This design was tested to show the importance of the inclusion of a secondary flowpath, which allowed all wetland cells to flow into up to two receiving cells. All hydrological time series output $\bar{e}/\bar{Y}$ values for 13F3 were at least double those of design 13F1. The 1-min wetland storage volumes were, again, the most sensitive outputs with a $\bar{e}/\bar{Y}$ of -0.146 in design 13F3. Effluent peak rates also increased for both years of record with respective values of 10.0 and 17.8 cfs. Given these trends, the exclusion of the secondary flowpath resulted in significantly different simulated wetland storages and depths, and, as a result, different outflow rates.

Simulated effluent TSS and NO_3^- concentrations for design 13F3, with mean values of 153 and 0.224 mg/L, were slightly larger than those for 13F1, which produced mean effluent values of 147 and 0.215 mg/L. This trend suggested that the design 13F3 had a shorter residence time within denitrifying cells (i.e., cells with VEG = 2) than the 13F1 design. Therefore, the 13F3 design performed slightly worse with respect to water quality than the 13F1 design.

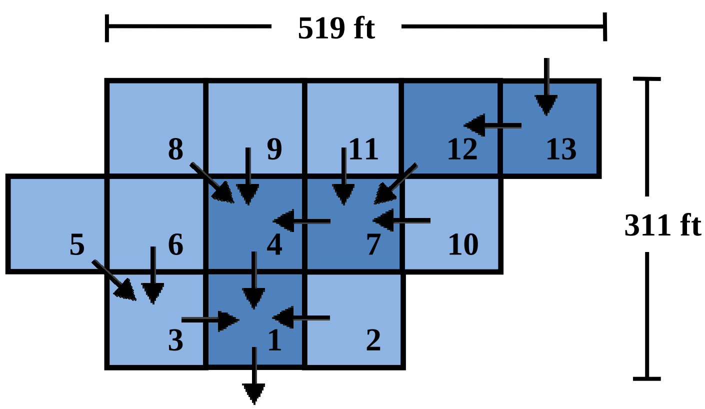

Figure 5-44 Model representation of the Barnstable 1 wetland with no secondary flowpath cells (design 13F3). Darkly-shaded cells highlight main flowpath through wetland. #s), initial water depths, elevations, and vegetation descriptors for each wetland cell.

Table 5-19 Barnstable 1 specifications for design 13F3 for the primary and secondary flowpaths (FID and FID2), initial water depths, elevations, and vegetation descriptors for each wetland cell.

Cell	FID	FID2	Initial Water Depth (ft)	Elevation (ft)	Vegetation Descriptor
1	0	1	1.29	0	2
2	1	2	1.29	0	2
3	1	3	1.29	0	2
4	1	4	0.785	0.503	1
5	3	5	0.254	1.03	1
6	3	6	0.254	1.03	1
7	4	7	1.19	0.103	1
8	4	8	0	2.26	1
9	4	9	0.468	0.820	1
10	7	10	0.000	1.38	1
11	7	11	0.878	0.410	1
12	7	12	0.238	1.05	1
13	12	13	0	1.87	1

5.3.3.3 Wetland Design 13F4

Wetland design 13F4 incorporated an altered main flowpath to that seen in 13F3 and did not incorporate a secondary flowpath. As show in Figure 5-45 and Table 5-20, the wetland design 13F4 had a main flowpath length of 5 cells. While both wetlands 13F3 and 13F4 had main flowpaths with the same length, the water was routed through different cells. The purpose of the 13F4 design was, therefore, to show the sensitivity of the Barnstable 1 wetland model to changes in the route of the main flowpath. Neither hydrologic not water quality outputs for the design 13F4 were significantly different from those of 13F3, suggesting that the model was not sensitive to the change in the main flowpath made in 13F4.

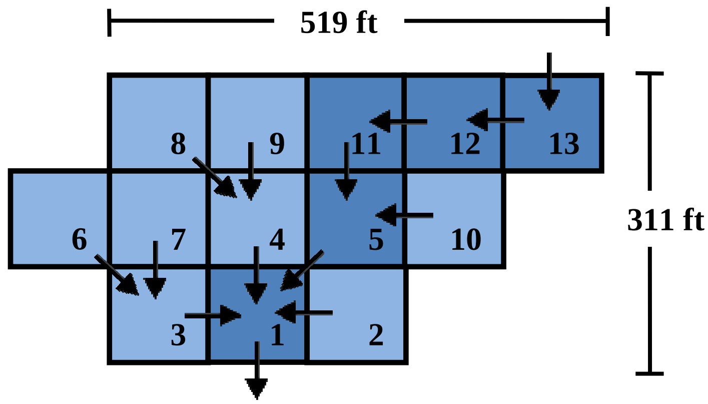

Figure 5-45 Model representation of the Barnstable 1 wetland with a main flowpath of 5 cells (design 13F4) that differs from the initial design (13F1) used for calibration as the FID numbers are not the same (darkly-shade cells highlight main flowpath).

Table 5-20 Barnstable 1 wetland model design specifications for design 13F4 for the primary and secondary flowpaths (FID and FID2), initial water depths, elevations, and vegetation descriptors for each wetland cell.

Cell	FID	FID2	Initial Water Depth (ft)	Elevation (ft)	Vegetation Descriptor
1	0	1	1.29	0	2
2	1	2	1.29	0	2
3	1	3	1.29	0	2
4	1	4	0.785	0.503	1
5	1	5	1.19	0.103	1
6	3	6	0.254	1.03	1
7	3	7	0.254	1.03	1
8	4	8	0	2.26	1
9	4	9	0.468	0.820	1
10	5	10	0	1.378	1
11	5	11	0.878	0.410	1
12	11	12	0.238	1.05	1
13	12	13	0	1.87	1

5.3.3.4 Wetland Design 13F5

The wetland design 13F5 altered the 13F3 design to incorporate a 6-cell main flowpath. This design was documented in Figure 5-46 and Table 5-21 and was used to assess the effect a longer flowpath had on the model performance relative to the 13F3 design, which had a 5-cell flowpath. The only notable hydrologic difference between the designs 13F3 and 13F5 was the improvement of the 1-min wetland storage volume prediction. The design 13F5 produced $\bar{e}/\bar{Y}$ and $\bar{S}_e/\bar{S}_y$ values of -0.097 and 0.546 for 1-min wetland storage volumes while the 13F3 design produced values of -0.146 and 0.682. The goodness of fit for all other hydrologic output time series also improved slightly. However, because wetland storage was the most sensitive wetland characteristic, it experienced the most change.

Water quality performance for the 13F5 design was also slightly better than that of the design 13F3, with respective mean weekly effluent TSS and NO_3^- concentrations of 148 and 0.194 mg/L. This reduction in effluent TSS and NO_3^- concentrations was due to the addition of cell 11 (see Figure 5-46) to the main flowpath, which resulted in an increased residence time in denitrifying cells. Therefore, increasing the flowpath within the model did affect both the hydrologic and the water quality performance of the wetland.

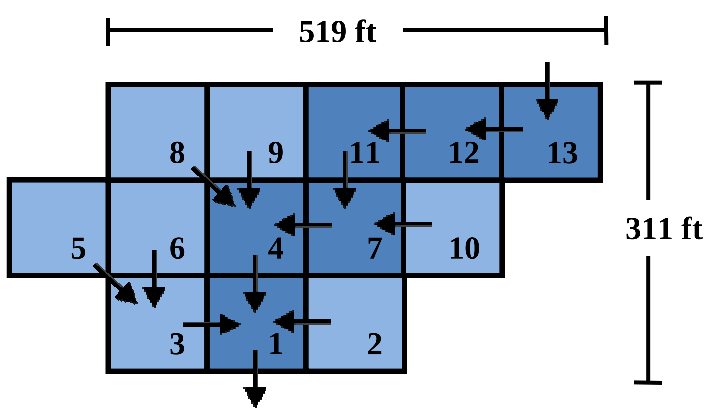

Figure 5-46 Model representation of the Barnstable 1 design wetland 13F5 with a main flowpath of 6 cells (darkly-shade cells).

Table 5-21 Barnstable 1 wetland model design specifications for design 13F5 for for the primary and secondary flowpaths (FID and FID2), initial water depths, elevations, and vegetation descriptors for each wetland cell.

Cell	FID #	FID2 #	Initial Water Depth (ft)	Elevation (ft)	Vegetation Descriptor
1	0	1	1.29	0	2
2	1	2	1.29	0	2
3	1	3	1.29	0	2
4	1	4	0.785	0.503	1
5	3	5	0.254	1.03	1
6	3	6	0.254	1.03	1
7	4	7	1.19	0.103	1
8	4	8	0.000	2.26	1
9	4	9	0.468	0.82	1
10	7	10	0.000	1.38	1
11	7	11	0.878	0.410	1
12	11	12	0.238	1.05	1
13	12	13	0.000	1.87	1

5.3.3.5 Wetland Design 13F6

Wetland design 13F6 altered the 13F3 design by incorporating a 4-cell main flowpath. All specifications for the 13F6 design are depicted in Figure 5-47 and listed in Table 5-22. The purpose of the design 13F6 was to assess the effect of a shortened flowpath on model performance relative to the 5-cell flowpath design 13F3. As a result of this shortened main flowpath, a greater portion of water was allocated to outflow in the 13F6 design. This trend was seen in the increased $\overline{e}/\overline{Y}$ values for 1-min effluent rates, 1-min water depths at the outlet weir, and weekly effluent volumes. Additionally, the design 13F6 predicted lower storage volumes, with a $\overline{e}/\overline{Y}$ of -0.179 versus that of design 13F3 of -0.146. The 4-cell flowpath allowed water to flow from the wetland more efficiently and more quickly, resulting in greater outflow and less storage within the wetland. The mean effluent weekly TSS and NO_3^- for design 13F6 were 156 and 0.250 mg/L, which were higher than those predicted by the 13F3 design of 153 and 0.224 mg/L. Based on these increases, the shorter flowpath produced a shorter wetland retention time, resulting in less accurate and poorer pollutant removal than that simulated by the 13F3 design.

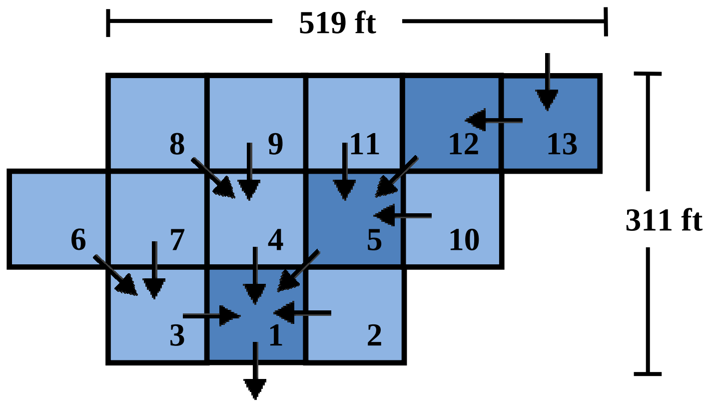

Figure 5-47 Model representation of the Barnstable 1 wetland 13F6 with a main flowpath of 4 cells (darkly-shade cells).

Table 5-22 Barnstable 1 wetland model design specifications for design 13F6 for for the primary and secondary flowpaths (FID and FID2), initial water depths, elevations, and vegetation descriptors for each wetland cell.

Cell	FID #	FID2 #	Initial Water Depth (ft)	Elevation (ft)	Vegetation Descriptor
1	0	1	1.29	0.00	2
2	1	2	1.29	0.00	2
3	1	3	1.29	0.00	2
4	1	4	0.785	0.503	1
5	1	5	1.19	0.103	1
6	3	6	0.254	1.03	1
7	3	7	0.254	1.03	1
8	4	8	0.00	2.26	1
9	4	9	0.468	0.820	1
10	5	10	0.000	1.38	1
11	5	11	0.878	0.410	1
12	5	12	0.238	1.05	1
13	12	13	0.000	1.87	1

5.3.3.6 Flowpath sensitivity results and discussion

The secondary flowpath appeared to have a significant impact on wetland storage and outflow allocation, which was shown through designs 13F2 and 13F3. However, significant hydrologic (see Table 5-23 and Table 5-24) and water quality (see Table 5-25 and Table 5-26) differences between the wetland designs 13F4 and 13F3, while both had a main flowpath length of 5 cells were not evident. Designs 13F5 (6-cell main flowpath) and 13F6 (4-cell main flowpath) did, however, produce different hydrologic and water quality outputs. These results suggest that the model was more sensitive to flowpath length rather than the specific defined main flowpath. Given a longer flowpath, water moved more slowly through the wetland, resulting in more storage, lower outflow rates, and a longer retention time. Conversely, the shorter 4-cell flowpath design 13F4 allowed for water to move more quickly through the wetland and resulted in higher outflow rates and shorter retention times. Additionally, as seen in design 13F2, a longer retention time did not always translate to more NO_3^- reduction and depended on the vegetative characteristics of the cells through which water was routed. These results suggest that while retention time is an important factor in both water quality and hydrologic performance, the retention time in specific wetland zones should be emphasized in addition to the overall wetland retention time.

Table 5-23 Resulting annual equivalent depths for wetland outflow, infiltration, ET, change in storage., annual peak and mean discharges for all six 13-cell wetland designs. Mean flowrates excluded zero-flows.

Trial	Total annual outflow (in.)		Total annual infiltration (in.)		Total annual ET (in.)		Annual change in storage (in.)		Peak outflow discharge (cfs)		Mean outflow discharge (cfs)	
	Year 1	Year 2	Year 1	Year 2	Year 1	Year 2	Year 1	Year 2	Year 1	Year 2	Year 1	Year 2
13F1	10.4	23.6	8.69	11.8	20.7	35.3	-0.07	2.46	8.45	6.75	0.0864	0.137
13F2	10.4	24.0	8.54	11.2	19.6	32.1	-0.01	0.97	8.86	11.1	0.0861	0.129
13F3	10.4	24.4	8.43	10.7	18.6	29.1	-0.02	-0.06	10.0	17.8	0.0838	0.123
13F4	10.4	24.4	8.43	10.8	18.7	29.6	-0.06	0.07	9.99	18.1	0.0839	0.123
13F5	10.4	24.3	8.49	10.9	19.0	29.8	0.01	0.38	9.45	15.2	0.0838	0.127
13F6	10.4	24.5	8.40	10.7	18.3	28.9	-0.09	-0.35	10.9	21.2	0.0813	0.120

Table 5-24 Hydrologic goodness-of-fit results for all six 13-cell wetland designs.

Trial	Weekly effluent volume (m3)				Water Depth at Weir (ft)				Wetland Storage Volume (ft^3)				Outflow (cfs)			
	$\bar{e}$	$RMSE$	$\bar{e}/\bar{Y}$	$\bar{S}_e/\bar{S}_y$	$\bar{e}$	$RMSE$	$\bar{e}/\bar{Y}$	$\bar{S}_e/\bar{S}_y$	$\bar{e}$	$RMSE$	$\bar{e}/\bar{Y}$	$\bar{S}_e/\bar{S}_y$	$\bar{e}$	$RMSE$	$\bar{e}/\bar{Y}$	$\bar{S}_e/\bar{S}_y$
13F1	1.06	31.5	0.024	0.422	-0.012	0.114	-0.008	0.221	2517	13400	0.030	0.413	0.002	0.070	0.029	0.230
13F2	1.81	32.3	0.040	0.433	0.001	0.100	0.001	0.195	-5855	15951	-0.069	0.492	0.003	0.054	0.045	0.178
13F3	2.43	34.8	0.054	0.465	0.024	0.118	0.016	0.230	-12412	22135	-0.146	0.682	0.004	0.098	0.059	0.325
13F4	2.35	34.7	0.052	0.465	0.023	0.113	0.016	0.219	-11241	20120	-0.133	0.620	0.004	0.100	0.057	0.331
13F5	2.24	33.6	0.050	0.451	0.003	0.098	0.002	0.191	-8247	17709	-0.097	0.546	0.004	0.070	0.055	0.233
13F6	2.55	35.6	0.057	0.477	0.047	0.165	0.032	0.320	-15151	25030	-0.179	0.771	0.004	0.143	0.062	0.474

Table 5-25 The mean, maximum, minimum, and standard deviation (std) for weekly composite simulated TSS, NH_4^+, and NO_3^- effluent concentrations for all six 13-cell wetland designs.

Trial	**TSS (mg/L)**				**NH4 (mg/L)**				**NO3 (mg/L)**			
	Mean	Min	Max	std	Mean	Min	Max	std	Mean	Min	Max	std
13F1	147	12.7	527	108.8	0.223	0.200	0.393	0.041	0.218	0.001	2.66	0.375
13F2	153	5.44	655	124	0.227	0.200	0.387	0.046	0.205	0.000	2.53	0.355
13F3	153	6.83	703	130	0.225	0.200	0.379	0.044	0.224	0.000	2.71	0.385
13F4	152	6.55	693	129	0.225	0.200	0.394	0.046	0.219	0.000	2.80	0.389
13F5	148	4.06	637	124	0.226	0.200	0.399	0.047	0.194	0.000	2.13	0.320
13F6	156	15.9	726	134	0.224	0.200	0.397	0.043	0.250	0.002	3.37	0.454

Table 5-26 Goodness-of-fit results for six all 13-cell wetland designs for weekly composite concentrations of (1) TSS, (2) NH_4^+, and (3) NO_3^-.

Trial	**TSS (mg/L)**				**NH4 (mg/L)**				**NO3 (mg/L)**			
	$\bar{e}$	*RMSE*	$\bar{e}/\bar{Y}$	$\bar{S}_e/\bar{S}_y$	$\bar{e}$	*RMSE*	$\bar{e}/\bar{Y}$	$\bar{S}_e/\bar{S}_y$	$\bar{e}$	*RMSE*	$\bar{e}/\bar{Y}$	$\bar{S}_e/\bar{S}_y$
13F1	-5.5	192	-0.036	1.20	0.002	0.212	0.008	0.972	0.000	0.278	0.001	0.923
13F2	3.0	196	0.020	1.24	0.005	0.200	0.023	0.930	-0.007	0.259	-0.034	0.867
13F3	3.1	199	0.021	1.26	0.002	0.198	0.011	0.917	0.012	0.273	0.058	0.914
13F4	2.1	200	0.014	1.260	0.003	0.195	0.014	0.905	0.008	0.284	0.036	0.952
13F5	-2.1	197	-0.014	1.243	0.004	0.201	0.016	0.932	-0.017	0.222	-0.082	0.743
13F6	5.8	200	0.039	1.259	0.001	0.193	0.007	0.895	0.038	0.344	0.181	1.154

5.3.4 Effects of cell size

A total of three 26-cell designs were simulated to investigate the effect of cell size and number, especially in characterizing elevations and wetland flowpath. These three simulated 26-cell wetland designs were referred to as 26F1, 26F2, and 26F3 and are shown in Figures 5-48, 5-49, and 5-50. It was observed that increasing the main flowpath within the 26-cell wetland structure improved simulation outputs and corresponding goodness-of-fit statistics. However, because the actual Barnstable 1 wetland was not observed to have a well-defined flowpath (Jordan et al. 2003), it was more difficult to estimate an accurate flowpath due to the increased complexity introduced by the 26-cell structure. The greater flowpath complexity inherent in the 26-cell structure of the 26F1, 26F2, and 26F3 designs also produced different hydrologic and water quality model outputs than the original 13-cell structure 13F1, which suggested that wetland cell size was an important criterion when using the wetland model developed in the current study. Therefore, while the increased number of cells allowed for a more detailed characterization of the wetland elevations, defining a flowpath for the 26-cell design allowed for more error as the flowpath was more complex. The resulting goodness-of-fit statistics for all 26-cell designs are summarized in Table 5-27 through Table 5-30.

5.3.4.1 Wetland Design 26F1

The first 26-cell design 26F1, which is shown in Figure 5-48, had a main flowpath of 7 cells and produced greater outflow and promoted a shorter retention time than the original 13F1 design. Simulated results from the 26F1 design overpredicted outflow rates with respective weekly effluent volume and 1-min effluent rate $\bar{e}/\bar{Y}$ values of 0.064 and 0.071 (see Table 5-28). This increased outflow trend was also evident in the annual peak effluent discharge rates simulated in the 26F1 design of 11.3 and 25.6 cfs for each years 1 and 2. These peak flows were much higher than those produced by the 13F1 design, which were respectively 8.49 and 12.4 cfs for years 1 and 2. Corresponding underpredicted storage variables were also observed in the 26F1 design with 1-min wetland storage volume and water depths at the outlet $\bar{e}/\bar{Y}$ values of -0.260 and -0.100 as compared with the 13F1 values of -0.06 and -0.03. These hydrologic results suggested that the main flowpath of 7 cells in the 26F1 design moved water through the wetland more quickly than both in the 13F1 design and in the actual Barnstable 1 wetland, which further illustrates the trend that the main flowpath length is directly related to the simulated wetland retention time.

The reduced simulated retention time in the 26F1 design also increased effluent TSS, NO_3^-, and NH_4^+ concentrations (see Table 5-30). The resulting 26F1 water quality goodness-of-fit statistics reflected these increased effluent concentrations with respective weekly effluent TSS, NO_3^-, and NH_4^+ concentration $\bar{e}/\bar{Y}$ values of 0.0581, 0.016, and 0.247, all of which were more positive than the corresponding 13F1 values of -0.005, -0.135, and 0.004. The 26F1 $\bar{S}_e/\bar{S}_y$ values were also poorer than those achieved in the 13F1 design for all water constituents, which suggested that the 26F1 did not capture the effluent water quality trends as well as the 13F1 design. Despite the positive biases and increased $\bar{S}_e/\bar{S}_y$ values observed in both the hydrologic and water quality outputs of the design 26F1, it still matched the observed hydrologic data reasonably well given that it was not calibrated. The discrepancies observed between the 13F1 and 26F1 design did, however, suggest that the 26F1 flowpath design was not analogous to the 13F1 flowpath design.

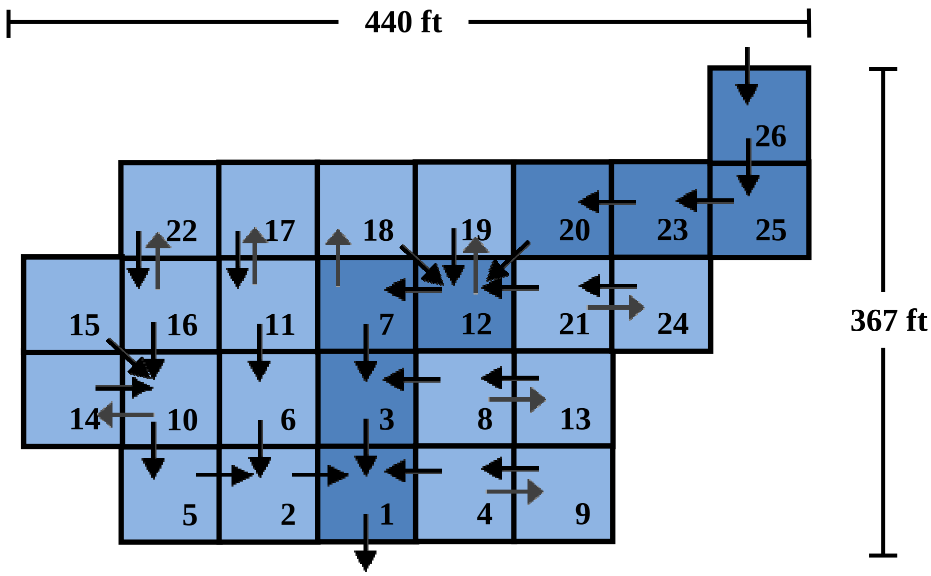

Figure 5-48 Model representation of the Barnstable 1 wetland with a 26-cell configuration and a main primary flowpath of 7 cells. This design was referred to as 26F2. Darkly-shaded cells highlight main flowpath through wetland. The primary flowpath (FID) is shown with black arrows and the secondary flowpath (FID2) is shown with grey arrows.

5.3.4.2 Wetland Design 26F2

A second 26-cell design (26F2) was developed with a main flowpath of 8 cells in order to increase the wetland retention time in the 26F1 design and to better match 13F1 hydrologic and water quality results (see Figure 5-49). The increased flowpath in the 26F2 design was observed to slightly improve both hydrologic and water quality goodness-of-fit statistics, suggesting that the 26F2 retention time was slightly longer than that of the 26F1 design. Despite these improvements in observed data fit and agreement with 13F1 output values, the same hydrologic and water quality trends that were seen in the 26F1 were observed in the 26F2 design. While the 26F2 underestimated wetland storage volumes and depths, it overpredicted outflow volumes and discharge rates as well as corresponding effluent TSS, NO_3^-, and NH_4^+ concentrations due to its shorter retention time.

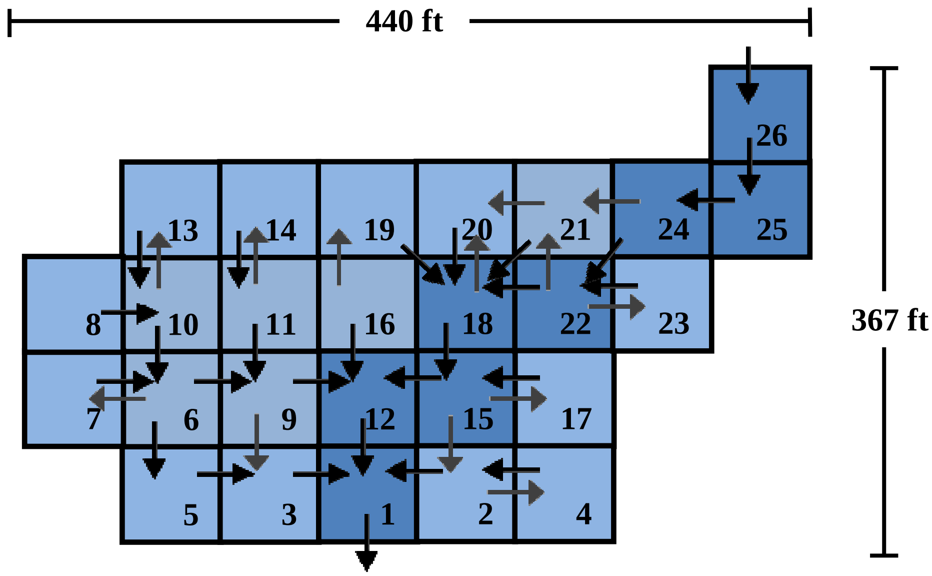

Figure 5-49 Model representation of the Barnstable 1 wetland with a 26-cell configuration and a main primary flowpath of 8 cells. This design was referred to as 26F2. Darkly-shaded cells highlight main flowpath through wetland. The primary flowpath (FID) is shown with black arrows and the secondary flowpath (FID2) is shown with grey arrows.

5.3.4.3 Wetland Design 26F3

A final 26-cell design was developed (26F3) with a main flowpath of 12 cells in order to further reduce the wetland retention in the 26-cell structure. While the 26F3 design matched both 13F1 and the actual Barnstable 1 wetland hydrology, it did not predict wetland water quality performance well. Resulting 26F2 goodness-of-fit statistics for all hydrologic time series except for 1-min water depths at the weir were better than those produced by the 13F1 design (see Table 5-28). These hydrologic fit improvements suggest that the 12-cell flowpath in the 26F3 design promoted a wetland retention time similar to that observed in the actual Barnstable 1 wetland.

Despite this improvement in retention time simulation, the 26F3 design water quality performance was poorer than that of the 13F1 design. This discrepancy in water quality performance suggested that while the 13F1 and 26F3 designs had similar retention times, their respective flowpaths through the wetland were different. Increased effluent NO_3^- concentrations in the 26F3 could, for example, be due to a shorter respective retention time in cells with emergent vegetation, in which denitrification was simulated. Similarly, the relative complexity of the 26F3 flowpath may have also promoted secondary routes from the inlet to the outlet with shorter retention times than the overall mean wetland retention time, which would, in turn, produce higher effluent water quality concentrations. Because the actual flowpath through the Barnstable 1 wetland was not known, neither the 13F1 nor the 26F3 flowpath can be deemed more accurate. Therefore, if calibrated, it seems that the 26F3 design could recreate both the observed hydrologic and water quality performance of the Barnstable 1 wetland.

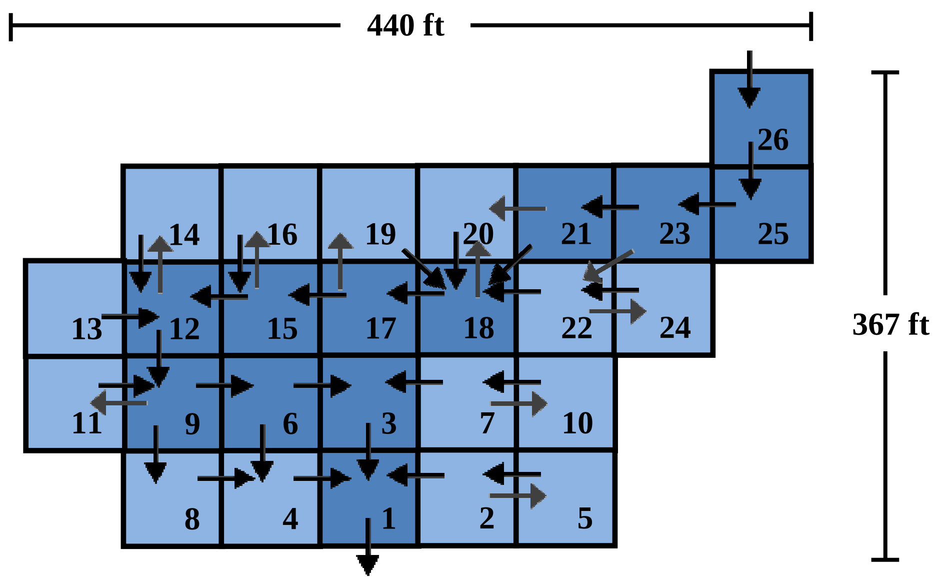

Figure 5-50 Model representation of the Barnstable 1 wetland with a 26-cell configuration and a main primary flowpath of 12 cells. This design was referred to as 26F3. Darkly-shaded cells highlight main flowpath through wetland. The primary flowpath (FID) is shown with black arrows and the secondary flowpath (FID2) is shown with grey arrows.

5.3.4.4 Cell size sensitivity results

While both the 13- and 26-cell design structures predicted observed hydrologic and water quality values reasonably well, the optimal number of cells chosen to represent a given wetland design was found to be dependent on a number of factors including (1) the extent to which a given cell size could characterize the wetland design, (2) the computational time, and (3) flowpath complexity. As with any model, the current model is only as good at the data used to calibrate it. This experiment also reinforced the sensitivity of both hydrologic and water quality performance on wetland flowpath. The flowpath of the Barnstable 1 wetland was not well-defined and a fair amount of error was associated with the elevation map used to estimate cell elevations (see Section 5.1.4.3). Given these data limitations, greater error may be associated with smaller cell sizes simply due to the scale at which elevations and the flowpath were reliably known. Smaller respective cells require greater knowledge of the internal flowpath of a given wetland design. However, it is also necessary to choose a cell size sufficiently small to correctly capture wetland structure. Therefore, the correct cell size should represent general wetland topography and allow for sufficient but not excessive flowpath characterization.

Table 5-27 Resulting annual equivalent depths for wetland outflow, infiltration, ET, change in storage., annual peak and mean discharges for all 26-cell wetland designs (26F1, 26F2, and 26F3) as well as the initial, calibrated design 13F1. Mean flowrates excluded zero-flows.

Trial	Total annual outflow (in.)		Total annual infiltration (in.)		Total annual ET (in.)		Annual change in storage (in.)		Peak outflow discharge (cfs)		Mean outflow discharge (cfs)	
	Year 1	Year 2	Year 1	Year 2	Year 1	Year 2	Year 1	Year 2	Year 1	Year 2	Year 1	Year 2
13F1	10.42	24.20	9.52	12.06	19.31	30.38	0.71	-0.01	8.49	12.44	0.0809	0.114
26F1	10.91	24.54	8.03	10.41	17.14	27.56	-1.37	0.51	11.25	25.56	0.0801	0.121
26F2	10.83	24.21	8.24	10.92	17.94	29.85	-1.43	1.57	9.37	11.65	0.0829	0.132
26F3	10.52	23.89	8.70	11.36	19.64	32.82	0.15	1.95	10.67	11.80	0.0822	0.119

Table 5-28 Hydrologic goodness-of-fit results for all 26-cell wetland designs (26F1, 26F2, and 26F3) as well as the initial, calibrated design 13F1.

Trial	Weekly effluent volume (m3)				Water Depth at Weir (ft)				Wetland Storage Volume (ft^3)				Outflow (cfs)			
	$\bar{e}$	*RMSE*	$\bar{e}/\bar{Y}$	$\bar{S}_e/\bar{S}_y$	$\bar{e}$	*RMSE*	$\bar{e}/\bar{Y}$	$\bar{S}_e/\bar{S}_y$	$\bar{e}$	*RMSE*	$\bar{e}/\bar{Y}$	$\bar{S}_e/\bar{S}_y$	$\bar{e}$	*RMSE*	$\bar{e}/\bar{Y}$	$\bar{S}_e/\bar{S}_y$
13F1	2	32	0.042	0.435	0.038	0.130	0.03	0.25	-5068	23854	-0.06	0.74	0.003	0.060	0.047	0.199
26F1	2.91	34.5	0.064	0.463	-0.150	0.226	-0.10	0.439	-21989	26212	-0.260	0.808	0.005	0.120	0.071	0.395
26F2	2.37	32.8	0.0529	0.439	-0.183	0.228	-0.123	0.444	-15824	18379	-0.187	0.566	0.004	0.053	0.059	0.176
26F3	1.62	29.3	0.036	0.392	-0.174	0.221	-0.12	0.430	-1162	17940	-0.014	0.553	0.003	0.110	0.041	0.362

Table 5-29 Goodness-of-fit results for all 26-cell wetland designs (26F1, 26F2, and 26F3) for weekly composite concentrations of (1) TSS, (2) NH_4^+, and (3) NO_3^-.

Trial	TSS				NH4				NO3			
	$\bar{e}$ (mg/L)	RMSE (mg/L)	$\bar{e}/\bar{Y}$	$\bar{S}_e/\bar{S}_y$	$\bar{e}$ (mg/L)	RMSE (mg/L)	$\bar{e}/\bar{Y}$	$\bar{S}_e/\bar{S}_y$	$\bar{e}$ (mg/L)	RMSE (mg/L)	$\bar{e}/\bar{Y}$	$\bar{S}_e/\bar{S}_y$
13F1	-1.2	549	-0.005	0.785	-0.042	0.276	-0.135	0.670	0.002	0.394	0.004	0.418
26F1	8.72	207	0.0581	1.307	0.00355	0.192	0.0160	0.892	0.052	0.401	0.247	1.343
26F2	-1.40	197	-0.00932	1.25	0.00668	0.199	0.0300	0.925	0.022	0.270	0.106	0.905
26F3	-5.56	195	-0.0366	1.225	0.00831	0.200	0.0373	0.921	-0.030	0.188	-0.140	0.629

Table 5-30 The mean, maximum, minimum, and standard deviation (std) for weekly composite simulated TSS, NH_4^+, and NO_3^- effluent concentrations for all 26-cell wetland designs (26F1, 26F2, and 26F3).

Trial	TSS (mg/L)				NH4 (mg/L)				NO3 (mg/L)			
	Mean	Min	Max	std	Mean	Min	Max	std	Mean	Min	Max	std
13F1	132	7.57	495	108	0.162	0.074	0.368	0.062	0.251	0.001	2.96	0.421
26F1	159	8.44	803	143	0.226	0.200	0.403	0.0466	0.264	0.001	3.84	0.504
26F2	149	4.47	682	123	0.229	0.200	0.409	0.0512	0.234	0.000	2.67	0.383
26F3	146	6.21	543	116.0	0.231	0.200	0.449	0.0581	0.185	0.000	1.61	0.272

Chapter 6: MDE Stormwater Wetland Design

6.1 DESIGN EXAMPLE

In this example, a shallow stormwater wetland was designed following the procedure outlined by MDE (2009) for a wetland at the Clevenger Community Center in Charles County, MD. Given these specifications and location, MDE (2009) concluded that the following design criteria were necessary:

1. The Water quality volume $\boldsymbol{WQ_v}$ **,** which is the storage required to capture and treat runoff from 90% of the average annual rainfall, was required and was defined by MDE (2009) accordingly:

$$WQ_v = \frac{P \cdot R_v \cdot DA}{12} \tag{6-1}$$

   where P is the precipitation depth, R_v is the volumetric runoff coefficient, and DA is the drainage area (acres). Because the wetland site was located in Charles County, MD, a P of 1-in. was required as the site was in the Eastern Rainfall Zone of Maryland (see Figure 6-1). R_v was defined accordingly (MDE 2009):

$$R_v = 0.05 + 0.009 \cdot I \tag{6-2}$$

   where I represents the percent of imperviousness (%) of the drainage area.

2. The recharge volume $\mathbf{Re_v}$ storage was the MDE-define storage volume required in order to account for groundwater recharge lost due to the development in the contributing drainage area. MDE (2009) defined $\mathbf{Re_v}$ in two ways depending on the method used to provide groundwater recharge. If water was infiltrated by a

designed structure such as the wetland, a percent volume method should be used to calculate the volume of water that must infiltrate down to groundwater within the wetland:

$$\mathrm{Re}_v = \frac{S \cdot R_v \cdot DA}{12} \tag{6-3}$$

where S is the soil specific recharge factor (values listed in Table 2-2), and Re_v is the required recharge volume (ac-ft). If Re_v is to be treated non-structurally (i.e., filter strips, grass channels, etc. placed throughout the DA), the percent area method should be used:

$$\mathrm{Re}_v = S \cdot A_i \tag{6-4}$$

where A_i is the impervious area cover (acres), and, in this case, Re_v is equal to the area (acres) of non-structural treatment that must be provided for infiltration of water from the impervious surfaces within the drainage area.

3. MDE (2009) also required that any constructed wetland design control both the volume and discharge rates associated with the 1-yr, 24-hr storm event. This requirement was achieved by sizing wetland designs with the capacity to store and appropriately transfer the storage volume $\boldsymbol{Cp_v}$ (ac-ft) computed by MDE (2009) as the wetland inflow volume produced by the 1-yr, 24-hr storm event.
4. MDE (2009) also required the storage and control of outflow due to the 10-yr, 24-hr storm depending on local jurisdiction. The required wetland storage volume (ac-ft) computed by MDE (2009) to be associated with the 10-yr, 24-hr storm was referred to as $\boldsymbol{Q_p}$. Based on the location of the proposed wetland in the MDE (2009) example, $\boldsymbol{Q_p}$ storage was included in the design.

5. Finally, MDE (2009) required for some locations that constructed wetlands either control or safely transfer the influent volume $\boldsymbol{Q_f}$ (ac-ft) associated with the 100-yr, 24-hr storm. $\boldsymbol{Q_f}$ was not controlled in this example, but was conveyed safely through the wetland.

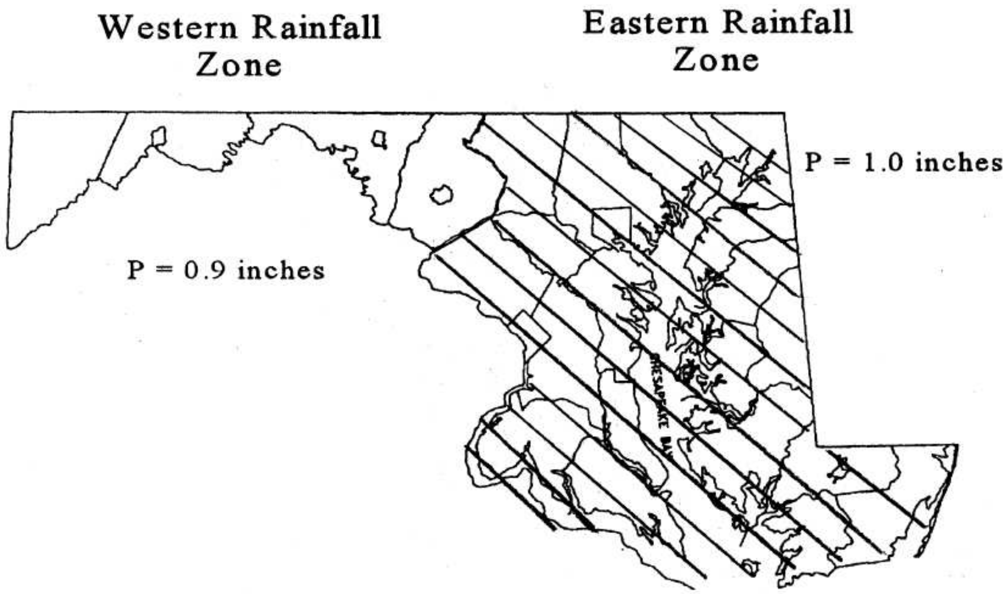

Figure 6-1 Assignment of the precipitation depth *P* used to determine WQ_v for the state of Maryland (MDE 2009).

6.2 PRELIMINARY DESIGN

Given the site specifications, initial values of WQ_v^*, Re_v, Cp_v, and $Q_{p,10}$ were calculated. In order to calculate an initial estimate of WQ_v^*, R_v was calculated accordingly (MDE 2009):

$$R_v = 0.05 + 0.009 \cdot \left(\frac{1.94\,\text{ac}}{5.3\,\text{ac}} \right) \cdot 100 = 0.379 \tag{6-5}$$

Next, an initial wetland WQ_v^* was computed (MDE 2009):

$$WQ_v^* = \frac{(1\,\text{in.})(0.379)(5.3\,\text{ac})}{12\,\text{in./ft}} = 0.167\,\text{ac}-\text{ft} \tag{6-6}$$

Therefore, the wetland must treat a volume of 0.167 ac-ft (7,292 ft^3) in order to meet MDE wetland water quality standards.

Due to a high water table at the site, infiltration was assumed not to be a feasible function of the proposed wetland (MDE 2009). In this case, MDE required that the Re_v volume be collected and infiltrated by a separate facility or be directed to pervious areas within the drainage area. Therefore, the actual wetland will not provide infiltration. An offsite infiltration trench was assumed to infiltrate water structurally in this example (MDE 2009). Therefore, a percent volume Re_v value (see Equation 6-3) was calculated accordingly (MDE 2009):

$$\mathrm{Re}_v = \frac{0.26(0.379)(5.3\,\text{ac})}{12\,\text{in./ft}} = 0.0435\,\text{ac}-\text{ft} \tag{6-7}$$

A total volume of 0.0435 ac-ft (1,896 ft^3) must be provided offsite. Next, the Cp_V, which was the required storage volume for the control of the 1-yr, 24-hr storm, was calculated using TR-55. From TR-55, the drainage area was estimated to have a time of concentration t_c of 0.26 hr (see Section 11.2) and an overall CN of 74. Output from TR-55 is compiled in Table 6-1 (MDE 2009).

Table 6-1 Relevant output values from TR-55 as well as the input rainfall depths for the 1, 10, and 100-yr 24-hr storm events as defined by MDE (2009).

	1-yr, 24-hr (Cp_V)	**10-yr, 24-hr (Q_p)**	**100-yr, 24-hr (Q_f)**
Rainfall depth (in.) *(MDE 2009)*	2.7	5.3	7.5
Post-development runoff depth (in.)	0.72	2.61	4.48
Post-development unit peak discharge (cfs/ac/in.)	0.995	1.10	1.12
Post-development peak discharge (cfs)	3.79	15	27.0
Pre-development runoff depth (in.)	0.18	1.34	2.76
Pre-development unit peak discharge (cfs/ac/in.)	0.460	0.904	0.967
Pre-development peak discharge (cfs)	0.439	6.42	14.1
q_o / q_i ratio	0.115	0.400	0.519

Once MDE (2009) obtained the TR-55 outputs, they were used to compute Cp_V and an initial estimate of the required outlet orifice diameter for the stormwater wetland design. The ratio of post- to pre-development peak 1-yr 24-hr flowrates was calculated in order to determine the corresponding ratio of required Cp_V to total runoff volume from the drainage area. Figure 6-2 was used to determine a suitable q_o / q_i ratio based on the maximum detention time of 24 hrs for a Use I watershed and the unit peak discharge for the 1-yr 24-hr storm event. The term "Use 1" referred to the designated use of the wetland effluent water. Use 1 describes watersheds in which water is designated for general use. As a contrast, watersheds in which water is used either for trout reproduction or for recreation are referred to Use III and Use IV and require a maximum retention time of 12 hr in order to reduce effluent water temperatures, to which trout are very sensitive (MDE 2009). The unit peak discharge was also determined graphically from Figure 6-3 given the drainage area time of concentration of 0.26 hr and the ratio of initial abstraction to rainfall depth for the 1-yr, 24-hr storm event (I_a / P). Initial abstraction I_a was calculated using the SCS Curve Number relationship (MDE 2009):

$$I_a = 0.2\left(\frac{1000}{CN} - 10\right) = 0.2\left(\frac{1000}{74} - 10\right) = 0.703\,\text{in.} \qquad (6\text{-}8)$$

where I_a is the initial abstraction (in.) of the drainage area. I_a / P could then be calculated using the 1-yr 24-hr precipitation depth of 2.7 in. (MDE 2009):

$$I_a / P = 0.703\,\text{in.} / 2.7\,\text{in.} = 0.26 \qquad (6\text{-}9)$$

Given an I_a / P ratio of 0.26 and a time of concentration of 0.26 hr, a 1-yr 24-hr unit peak discharge q_u of 625 csm/in (cfs/mi^2-in.) was derived from Figure 6-3. This value of q_u and the 24-hr extended detention time were then used to extrapolate a q_o / q_i value of 0.030 from Figure 6-2. This q_o / q_i value was input into the following equation to determine the ratio of required Cp_V storage (in.) to runoff depth (in.) V_S / V_R (MDE 2009):

$$\begin{aligned} V_S / V_R &= 0.683 - 1.43(q_o / q_i) + 1.64(q_o / q_i)^2 - 0.84(q_o / q_i)^3 \\ &= 0.683 - 1.43(0.030) + 1.64(0.030)^2 - 0.84(0.030)^3 \\ &= 0.64 \end{aligned} \qquad (6\text{-}10)$$

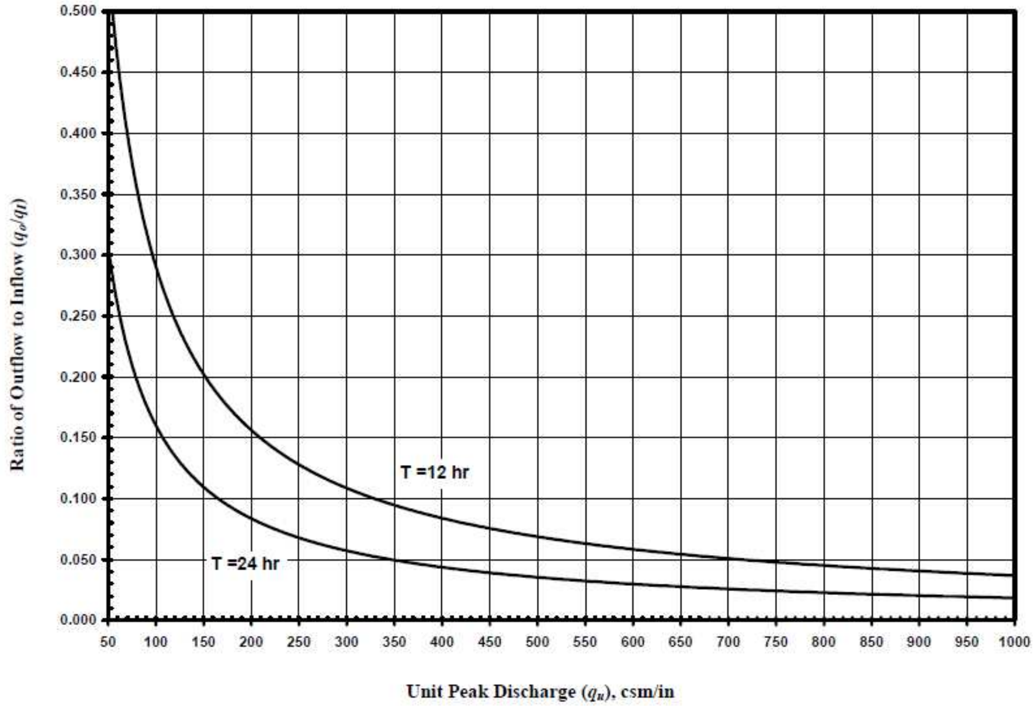

Figure 6-2 Relationship between the ratio of peak outflow to inflow (dimensionless) and the unit peak discharge (csm/in.) as it depends on maximum allowable detention time T (MDE 2009).

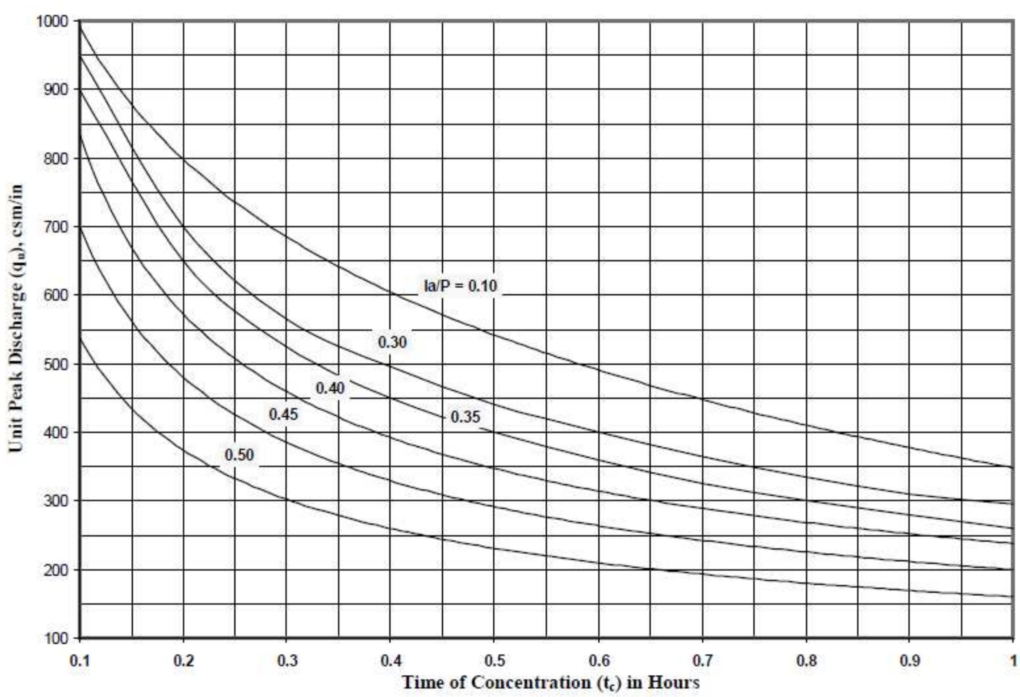

Figure 6-3 Relationship between the unit peak discharge (csm/in.) and the drainage area time of concentration as it depends on the dimensionless ratio of initial abstraction to rainfall depth (MDE 2009).

From the resulting V_S / V_R value of 0.64, the initial estimate of Cp_V was calculated (MDE 2009):

$$Cp_V = \frac{(V_S / V_R) \cdot V_R \cdot A}{12\ \text{in./ft}} = \frac{(0.64)(0.72\ \text{in.})(5.3\ \text{ac})}{12\ \text{in./ft}} = 0.204\ \text{ac} - \text{ft} \qquad (6\text{-}11)$$

Therefore, a total of 0.204 ac-ft (8,865 ft^3) of storage must be provided in the wetland design to meet Cp_V requirements. The q_o to be controlled by the wetland as defined by q_o / q_i and the TR-55 derived post-development 1-yr 24-hr peak flow of 3.7 cfs reported in Table 6-1 (MDE 2009):

$$q_o = (q_o / q_i) \cdot q_i = (0.030)(3.79\ \text{cfs}) = 0.113\ \text{cfs} \qquad (6\text{-}12)$$

Given a required peak outflow of 0.11 cfs for the 1-yr 24-hr, the orifice area was calculated by rearranging the orifice equation (MDE 2009):

$$A_o = \frac{q_o}{C_o\sqrt{2gh_o}} = \frac{0.113\ \text{cfs}}{0.6\sqrt{2(32.2\ \text{ft/s}^2)(3\ \text{ft})}} = 0.013\ \text{ft}^2 \qquad (6\text{-}13)$$

where h_o represents the maximum storage depth associated with Cp_V (ft) and was assumed to equal 3 ft in the MDE example. From this orifice area A_o, an initial orifice diameter d_o was calculated (MDE 2009):

$$d_o = \sqrt{\frac{4 \cdot A_o}{\pi}} = \sqrt{\frac{4(0.013\ \text{ft}^2)}{\pi}} = 0.129\ \text{ft} \qquad (6\text{-}14)$$

In order to produce a maximum outflow rate of 0.11 cfs for a 1-yr 24-hr storm event, a d_o of 0.129 ft (1.54 in.) would be required. MDE, however, requires a minimum orifice diameter of 3 in. to avoid clogging problems. Therefore, the orifice diameter was set to equal 3 in., resulting in an orifice area of 0.0491 ft^2 and an outflow rate of 0.42 cfs when 3-ft of head are present over the orifice. This resulting maximum effluent orifice flowrate was slightly smaller than the corresponding pre-development 1-yr, 24-hr flowrate of 0.434 cfs, which suggests that the orifice was sufficiently sized.

Storage volumes and outflow rates for Q_P were then computed using the same methods used for all Cp_V values. The I_a / P was calculated using the 10-yr, 24-hr precipitation depth of 5.3 in. (MDE 2009):

$$I_a / P = 0.703\,\text{in.} / 5.3\,\text{in.} = 0.13 \tag{6-15}$$

The q_o / q_i ratio for the Q_P values was calculated by MDE (2009) directly based on the q_o and q_i flows of 6 and 15 cfs for the 10-yr, 24-hr storm obtained from Table 6-1 (MDE 2009):

$$q_o / q_i = 6\,\text{cfs} / 15\,\text{cfs} = 0.40 \tag{6-16}$$

From this q_o / q_i value, the corresponding V_S / V_R was calculated (MDE 2009):

$$V_S / V_R = 0.683 - 1.43(0.40) + 1.64(0.40)^2 - 0.84(0.40)^3 = 0.32 \tag{6-17}$$

The Q_P was then calculated (MDE 2009):

$$Q_P = \frac{(0.32)(2.61\,\text{in.})(5.3\,\text{ac})}{12\,\text{in./ft}} = 0.37\,\text{ac} - \text{ft} \tag{6-18}$$

Therefore, a total of 0.37 ac-ft (16,068 ft^3) storage above the WQ_v storage is required to control the 10-yr, 24-hr storm event. Q_f storage was not included in this design example. Generally, 100-yr storm storage is only relevant when building in 100-yr floodplain. The final required storages for the proposed shallow wetland are shown in Table 6-2.

Table 6-2 Summary table of all preliminary storage volumes for the example shallow wetland (MDE 2009).

Storage	Volume Required (ac-ft)	Notes
WQ_v	0.167	---
Re_v	0.0435	Treated offsite (included within WQ_v)
Cp_V	0.204	Cp_V release rate is 0.42 cfs
Q_P	0.370	Q_P release rate is 6 cfs
Q_f	---	provide safe passage of 100-yr storm in final design

6.3 MDE DESIGN CRITERIA COMPUTATION

The wetland surface area and volume was divided according to the general MDE design criteria that the wetland surface area should be at least 1.5% of the drainage area and water depths should be broken down accordingly, a minimum of 35% of surface area must be ≤ 6 in. and at least 65% of it must be ≤ 18 in. to promote sustainable wetland vegetation (MDE 2009). For shallow wetlands, MDE requires that the wetland surface area be at least 1.5% of the drainage area. Therefore, the minimum required wetland surface area was calculated accordingly (MDE 2009):

$$SA_o = 0.015 \cdot 5.3\,\mathrm{ac} = 0.0795\,\mathrm{ac} \tag{6-19}$$

where SA_o represents the minimum wetland surface area (acres), which is equal to 0.0795 acres or 3,463 ft^2. Additionally, because Re_v was assumed to be treated offsite, it could be subtracted from WQ_v as the wetland will not need to treat this volume of water. The updated WQ_v value was calculated (MDE 2009):

$$WQ_v = WQ_v^* - \mathrm{Re}_v = 7{,}292\,\mathrm{ft}^3 - 1{,}896\,\mathrm{ft}^3 = 5{,}396\,\mathrm{ft}^3\ (0.124\,\mathrm{ac\text{-}ft}) \tag{6-20}$$

The wetland requires only a corrected WQ_v volume of 5,396 ft^3 (0.124 ac-ft). Next, the forebay was specified to have a volume (V_F) of 10% of the WQ_v (MDE 2009):

$$V_F = 0.10(5{,}396\ \mathrm{ft}^3) = 540\ \mathrm{ft}^3 \qquad (6\text{-}21)$$

At least 25% of WQ_v was required to be stored in areas with depths of greater than or equal to 4 ft. These areas were called deepwater areas. The minimum total deepwater surface area was calculated accordingly (MDE 2009):

$$V_D = 0.25(5{,}396\ \mathrm{ft}^3) = 1{,}349\ \mathrm{ft}^3\ \ (0.031\ \mathrm{ac-ft}) \qquad (6\text{-}22)$$

where V_D represents the total volume (ft^3) of the deepwater areas within the wetland design. It was acceptable to use the forebay and micropool to fulfill this requirement.

High-marsh areas were defined by MDE (2009) as those whose water depth was less than or equal to 6 in. (0.5 ft). MDE required that these areas comprise at least 35% of the wetland surface area (MDE 2009):

$$SA_H = 0.35(3{,}463\ \mathrm{ft}^2) = 1{,}212\ \mathrm{ft}^2\ \ (0.028\ \mathrm{ac}) \qquad (6\text{-}23)$$

where SA_H represents the surface area of the wetland designated to high-marsh areas. Finally, total marsh areas (low and high) were defined as areas with a water depth of less than or equal to 18 in. (1.5 ft). MDE required that these areas comprise 65% of the total wetland surface area (MDE 2009):

$$SA_T = 0.65(3{,}463\ \mathrm{ft}^2) = 2{,}251\ \mathrm{ft}^2\ \ (0.052\ \mathrm{ac}) \qquad (6\text{-}24)$$

where SA_T represents the wetland surface area with water depth less than or equal to 18 in. (1.5 ft).

6.4 FINAL STORMWATER WETLAND DESIGN

The current study developed a 25-cell shallow stormwater wetland design that met all of the MDE-defined criteria defined in Section 6.3. The resulting wetland was designed with a surface area SA_o, which was computed to be 3,463 ft^2 (0.0795 ac) in Equation 6-19. Therefore, each cell was assigned dimensions of 11.8 x 11.8 ft. A 25-cell structure was chosen in order to best characterize the different areas (i.e., high-marsh, low-marsh, deepwater areas, etc.). Additionally, the 25-cell structure allowed the analyses for differences of ±4% in areal design criteria specified by MDE (2009). The final 25-cell stormwater wetland design was used in sensitivity analyses (see Section 6.10) performed on both MDE design criteria as well as wetland input parameters such as wetland albedo and influent TSS diameter.

As discussed in Section 6.3, MDE (2009) required that the forebay be 10% of the WQ_v. Within the current example, a forebay with a volume of 540 ft^3 was required to serve to meet this criterion. The forebay, in addition to the micropool, also contributed to the deepwater areas (i.e., areas with water depths ≥ 4ft) in the wetland design. The forebay was modeled using one 11.8 x 11.8 ft cell. Therefore, in order to ensure the forebay accounted for ≥10% of the WQ_v and had a depth of ≥ 4 ft, it was assigned a depth of 4 ft. The resulting forebay was 10.3% of the WQ_v. MDE (2009) also required that the forebay be separated from the main wetland by a berm. Therefore, in the stormwater wetland design, a berm was placed after the forebay cell.

The forebay and micropool were assumed to account for all deepwater areas in the wetland as suggested in MDE (2009). According to MDE (2009), these deepwater areas were required have a volume V_D that accounted for greater than or equal to 25% of the WQ_v, which was computed in Equation 6-22 to be 1,349 ft^3. While a specific forebay depth was not specified by MDE, a micropool was defined as having a depth of 3-6 ft (MDE 2009). The micropool was modeled using one 11.8 x 11.8 ft cell and was assigned a depth of 5.75 ft, which resulted in a total deepwater volume of 1,351 ft^3 and accounted for 25.0% of the WQ_v.

Once deepwater areas were identified, high-marsh areas were targeted, all of which were assumed to have a water depth of 0.5 ft. A high-marsh minimum required surface area SA_H of 1,212 ft^2 was computed in Equation 6-23 by MDE (2009). This SA_H represented 35% of the wetland surface area SA_o. Therefore, a total of nine cells were used to model the high-marsh areas with water depths of 0.5 ft resulting in a total high-marsh surface area of 1,247 ft^2, which accounted for 36% of the SA_o.

MDE (2009) required that at least 65% of the SA_o have water depths less than or equal to 1.5 ft. This requirement incorporated both high (water depths ≤ 0.5 ft) and low (water depths between 0.5 and 1.5 ft) marsh areas. MDE (2009) determined that this total (high + low) marsh area should have a minimum surface area SA_T of 2,251 ft^2 (see Equation 6-24). In order to fulfill this requirement, a total of eight low-marsh cells with water depths of 1.25 ft were incorporated into the 25-cell stormwater wetland design. The addition of these low-marsh cells produced a total marsh area of 2,355 ft^2 (17 wetland cells), which accounted for 68.0% of the SA_o.

The forebay and micropool areas were represented by 1 cell each, high-marsh areas by nine cells, and low-marsh areas by eight cells, leaving six remaining cells to be defined. MDE did not provide guidelines for assigning depths to the wetland proportion represented by these six cells. Therefore, each of the six remaining cells was assigned a depth of 3 ft in order meet the WQ_v of 5,396 ft^3. The final wetland volume was 5,835 ft^3, which was slightly oversized so as to allow for sensitivity analyses of MDE volumetric criteria (see Section 6.13) without reducing the wetland volume below WQ_v. All resulting stormwater wetland depths, volumes, surface areas, and associated number of cells are summarized in Table 6-3. The final 25-cell wetland design met all MDE-specified criteria defined in Section 6.3.

Table 6-3 Zone depths, surface areas, volumes, and associated number of cells for the 25-cell stormwater wetland as designed by the procedures and specifications of MDE (2009). Each cell has dimensions of 11.7 by 11.7 ft.

Zone	Depth (ft)	Number of cells	Surface area (ft^2)	Volume (ft^3)
Forebay	4	1	139	554
Micropool	5.75	1	139	797
High-marsh	0.5	9	1247	623
Low-marsh	1.25	8	1108	1385
Other	3	6	831	2494
$\sum$	---	25	3463	5853

A number of other specifications were defined by MDE (2009) with relation to the wetland design configuration. MDE (2009) suggested an internal flowpath L:W ratio of greater than or equal to 1.5:1. Additionally, the forebay and micropool cells were required to be situated respectively at the inlet and outlet of the wetland. Aside from these requirements, however, MDE did not specify strict quantitative or qualitative criteria on the design and arrangement water depths within a stormwater wetland. Therefore, the 25-cell stormwater design developed in the current study was arranged in order to maximize the L:W ratio of the main flowpath through the wetland. The resulting 25-cell stormwater wetland design is shown in Figure 6-4. Additionally, Table 6-4 summarizes all of the flowpath, water depth, vegetation type, bottom elevation, and berm height specifications for each cell within the design depicted in Figure 6-4.

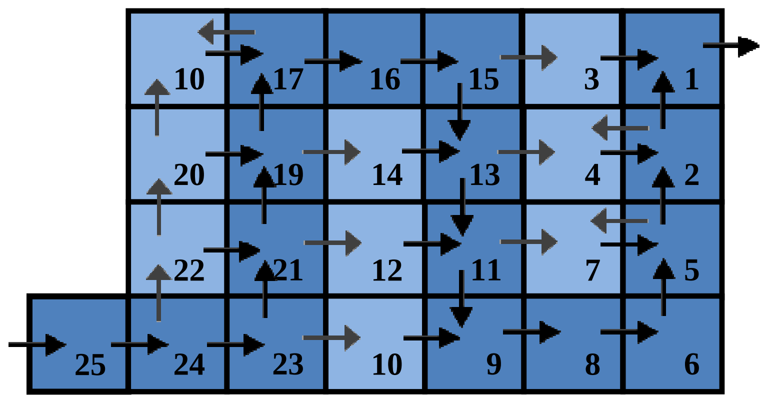

Figure 6-4 Diagram of wetland flowpath with numbers in each cell representing its location in the primary flowpath FID. The micropool outlet cell is labeled as 1 and the main wetland flowpath is highlighted in the darkly-shaded cells. Lightly shaded cells represent high-marsh cells. Additionally, a berm was situated between cell 25 (the forebay) and cell 24. Each cell has dimensions of 11.8 x 11.8 ft. Black arrows represent the primary flowpath FID while grey arrows indicate the secondary flowpath FID2.

Because the model uses the Rational method to estimate runoff from the contributing drainage area, the rational C was estimated. According to MDE (2009), the drainage area was comprised completely of type B soils with 1.94 acres of impervious surfaces, 0.3 acres of woods in good condition, and 3.06 acres of open space (lawns, parks, etc.) in good condition. Additionally the drainage area was defined by MDE (2009) to have a slope of 0.013 ft/ft. Given this description, a composite rational C value of 0.36 was calculated accordingly:

$$C = \frac{(1.94\,\text{ac})(0.85) + (3.06\,\text{ac})(0.08) + (0.3\,\text{ac})(0.08)}{5.3\,\text{ac}} = 0.36 \qquad (6\text{-}25)$$

The corresponding Rational C values for each land use type were defined according to McCuen (2005).

Table 6-4 Constructed wetland cell specifications for FID1, FID2, vegetation type (VEG), initial design depth in ft SS, and cell elevation above a datum EL in ft. Vegetation descriptor values of 0, 1, 2 indicate respectively that a given cell has no vegetation, emergent vegetation, and submerged vegetation.

Cell	FID	FID2	SS (ft)	VEG	EL (ft)	BERM (ft)
1	0	1	5.75	0	0	0
2	1	4	1.25	1	4.5	0
3	1	3	0.5	1	5.25	0
4	2	4	0.5	1	5.25	0
5	2	7	1.25	1	4.5	0
6	5	6	3	2	2.75	0
7	5	7	0.5	1	5.25	0
8	6	8	3	2	2.75	0
9	8	9	1.25	1	4.5	0
10	9	10	0.5	1	5.25	0
11	9	7	1.25	1	4.5	0
12	11	12	0.5	1	5.25	0
13	11	4	3	2	2.75	0
14	13	14	0.5	1	5.25	0
15	13	3	3	2	2.75	0
16	15	14	1.25	1	4.5	0
17	16	18	1.25	1	4.5	0
18	17	18	0.5	1	5.25	0
19	17	14	3	2	2.75	0
20	19	18	0.5	1	5.25	0
21	19	12	3	2	2.75	0
22	21	20	0.5	1	5.25	0
23	21	10	1.25	1	4.5	0
24	23	22	1.25	1	4.5	0
25	24	25	4	0	1.75	5.75

6.5 MDE OUTLET DESIGN

MDE designed an orifice to control the 1-yr, 24-hr flow and a weir to control the 10-yr, 24-hr flow exiting the wetland. The wetland outlet structure was located in the micropool. The 1-yr, 24-hr orifice was designed to control the Cp_V volume with a 3-in. diameter. It was situated directly above normal pool depth. The Q_P riser, which was designed to control the 10-yr, 24-hr flood, was located directly above the total storage depth associated with Cp_V.

In order to size the outlet weir structures, the storage depths associated Cp_V and Q_P with were calculated by dividing the storage volumes Cp_V and Q_P by the total wetland surface area (3,563 ft^2). The surface area was assumed not to increase as storage increased above the normal pool level for simplicity of calculations. In reality, the wetland would have sloped edges, causing the wetland surface area to increase as the wetland filled with more and more water. For this simplified example, the edges of the wetland were assumed vertical. Based on the water depth h_o of 2.56 ft over the wetland surface area associated with the Cp_V storage volume of 8,865 ft^3, the current study computed the maximum effluent rate from the wetland for future reference by rearranging Equation 6-13:

$$q_o = C_o A_o \sqrt{2gh_o} = 0.6 \cdot 0.0491\,\text{ft}^2 \sqrt{2(32.2\ \text{ft/s}^2)(2.56\ \text{ft})} = 0.379\ \text{cfs} \qquad (6\text{-}26)$$

where q_o (cfs) represents the maximum effluent orifice flow exiting the wetland design.

The Cp_V orifice invert was situated at the normal pool depth (5.75 ft above the micropool bottom) and the Q_P weir was situated 2.56 ft above the normal pool depth

(8.31 ft above the bottom of the micropool). The following (weir + orifice) flow equation was used to determine the Q_P weir length (MDE 2009):

$$q_{i,10} = C_w L_{10} h_{w,10}^{3/2} + C_o A_o \sqrt{2gh_o} \tag{6-27}$$

where C_w is the weir coefficient (3.1), L_{10} is the weir length (ft), $h_{w.10}$ is the depth of head over the Q_P weir (ft), and $q_{i,10}$ is the 10-yr, 24-hr inflow rate (cfs) that the weir must control. In the case of the Q_P weir, $h_{w.10}$ represents the difference between the depth of the Q_P and Cp_V storages (2.56 ft) and h_o represents the head above the centerline of the Cp_V orifice up to the top of the Q_P weir (4.64 ft). The q_i flowrate associate with Cp_V was 6 cfs. Therefore, the weir length L for Cp_V control could be solved for (MDE 2009):

$$L_{10} = \frac{q_{i,10} - C_o A_o \sqrt{2gh_o}}{C_w h_{w,10}^{3/2}} = \frac{6.42 \text{ cfs} - (0.6)(0.05 \text{ ft}^2)\sqrt{2(32.2 \text{ ft/s}^2)(2.56)}}{(3.1)(2.08)^{3/2}} = 0.650 \text{ ft} \tag{6-28}$$

Therefore, a weir length of 0.605 ft and a weir height of 2.08 ft were required to control the 10-yr, 24-hr flood. A similar method was used to determine the Q_f weir length. MDE assumed the orifice to be clogged during a 100-yr, 24-hr storm and, therefore, used the following two-stage weir flow equation to estimate the Q_f weir length (MDE 2009):

$$q_{i,100} = C_w L_{100} h_{w,100}^{3/2} + C_w L_{10} h_{w,10}^{3/2} \tag{6-29}$$

where $q_{i,100}$ represents the inflow 100-yr, 24-hr flow (27 cfs), L_{100} is the Q_f weir length, and $h_{w,100}$ is the head above the Q_f weir during a 100-yr flood, which was assumed to equal 0.5 ft in the MDE example. In this case $h_{w,10}$ referred to the head over the Q_P weir during a 100-yr flood (1.77 ft). L_{100} was then computed:

$$L_{100} = \frac{q_{i,100} - C_w L_{10} h_{w,10}^{2/3}}{C_w h_{w,100}^{3/2}} = \frac{27\text{ cfs} - (3.1)(0.650\text{ ft})(2.58\text{ ft})^{3/2}}{(3.1)(0.5)^{3/2}} = 17.0\text{ ft} \quad (6\text{-}30)$$

Therefore, the Q_f weir should have a length of 14.75 ft and a height of 0.5 ft. Figure 6-5 shows the resulting riser and orifice design at the outlet of the micropool.

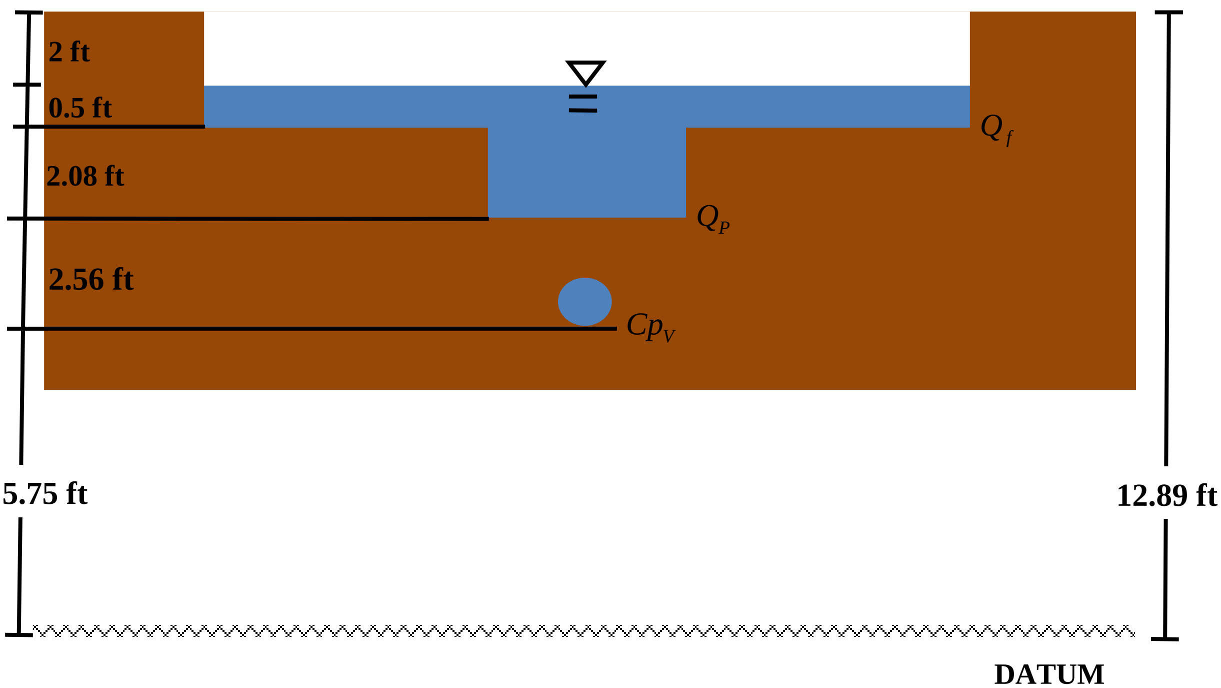

Figure 6-5 Outlet orifice and double riser design through the MDE (2009) method. The datum represents the bottom of the micropool, which was designed to have a depth of 5.75 ft. The designated Cp_V, Q_P, Q_f depths represent the water depths corresponding to the 1-yr, 10-yr, and 100-yr, 24-hr floods within the wetland.

6.6 FESABILITY CHECK

Once the wetland storages were calculated, MDE required that a wetland feasibility test be run to ensure that the wetland would not dry out over a 30-day drought. In order to determine if the wetland was feasible, the estimated wetland inflow was compared with the maximum drawdown within the wetland due to total evaporation.

Wetland monthly inflow was estimated by multiplying the drainage area runoff efficiency by the drainage area and the 2-yr, 24-hr rainfall depth for Charles County, MD (3.3 in.). The runoff efficiency was defined as the ratio of runoff to rainfall for the 2-yr, 24-hr for the drainage area. As calculated by TR-55, the estimated runoff from the drainage area for a 2-yr, 24-hr storm event was found to equal 1.1 in. The runoff efficiency (*E*) was then calculated as $E = 1.1\,\text{in.}/3.3\,\text{in.}$ by MDE (2009). A final estimate of wetland monthly inflow was calculated (MDE 2009):

$$Inflow = \frac{(3.3\,\text{in.})(0.33)(5.3\,\text{ac})}{12\,\text{in./f}t} = 0.48\,\text{ac} - \text{ft/month} \tag{6-31}$$

Outflow from the wetland via evaporation was then estimated using the highest monthly evaporation rate (see Table 6-5), which was that for the month of July with a value of 0.54 ft/month (MDE 2009):

$$ET = \frac{(3{,}463\,\text{ft}^2)(0.54\,\text{ft/month})}{43{,}560\,\text{ft}^2/\text{ac}} = 0.043\,\text{ac} - \text{ft/month} \tag{6-32}$$

Because the monthly outflow due to evaporation was an order of magnitude less than the monthly inflow to the wetland, it was concluded that the wetland should maintain water levels during normal conditions (MDE 2009). The MDE procedure next estimated the total evaporation loss and associated wetland drawdown during a 45-day dry period. Again, the maximum evaporation rate of 0.54 ft/month was used as a conservative estimate of the evaporation rate. Because July has 31 days, this monthly rate translates roughly to a daily rate of 0.017 ft/day. Therefore, over 45 days, a total depth of 0.78 ft (9.36 in.) would be evaporated.

Over a 45-day dry period at this evaporation rate, the wetland would experience a drawdown of 9.36 in. While the high-marsh areas would be dry during this period, the wetland would still have 5.64 in. (0.47 ft) of water in the low-marsh areas and at least 3.22 ft of water in the deepwater zones. Wetland vegetation that is tolerant of such water depth changes should be selected for this wetland design in order to ensure vegetation survival through potential extended dry periods.

Table 6-5 Precipitation and evaporation monthly rates for Maryland (MDE 2009)

	April	May	June	July	August	September
Precipitation (ft)	0.30	0.35	0.32	0.36	0.38	0.31
Evaporation (ft)	0.36	0.44	0.52	0.54	0.46	0.35

6.7 MEETING POND CODE STANDARDS

All wetland designs must meet Code 378 Pond Standards in order to ensure that berms are correctly sized and constructed, and that the appropriate permits are acquired before construction is begun. According to Code 378 Pond Standards, the following minimum requirements must be met for any embankment or excavated pond (MDE 2009):

1. Failure of the dam will not cause loss of life, or damage to private or public properties.
2. The product of the pond storage (ac-ft) times the effective height of the dam is less than 3,000 where the effective dam height is measured from the lowest bottom elevation of the pond to the height of the emergency spillway.

3. A dam with an effective height of 35 ft or less is considered a class “a” dam hazard in rural areas. Dams with effective heights of 20 ft or less in urban areas are also considered class “a” hazards. Class “a” refers to structures that are of least concern.

In addition to these requirements, a breach flow must be calculated for the proposed pond (MDE 2009):

$$Q_{\max} = 3.2 H_w^{5/2} \tag{6-33}$$

where H_w represents the depth of water (ft) at the dam at the time of failure (measured to the crest of the emergency spillway or to design high water if a emergency spillway is not present) and $Q_{\max}$ is the resulting peak breach discharge from the pond (cfs). Given $Q_{\max}$ and a topographic map of the design site, the resulting flood depth *d* (ft) would be calculated. If *d* was found to be less than or equal to 1.5 ft, the dam would still be considered a class “a” hazard. However, *d* depths greater than 1.5 ft were considered either “b” or “c” hazards. Wetland designers should ensure that their designs adhere to all Code 378 Pond Standards before going forward in construction. The stormwater wetland designed in the current section was assumed to meet these requirements given that a detailed topographic map was not provided.

6.8 STORMWATER WETLAND CALIBRATION

Subjective optimization was used to calibrate the example stormwater wetland. All inputs related to wetland ET were assigned to the corresponding mean literature values recorded in Section 2.3.3.1. Base K_{NIT} and K_{DNT} were set to the initially estimated mean values derived from the literature of 0.01 and 0.0208 hr^{-1} (see Section 5.2.8). Estimated influent TSS, NH_4^+, and NO_3^- concentrations were estimated to equal the mean values reported by Leisenring et al. (2012), which were based on a total of 11 stormwater wetlands located in the mid-Atlantic region and are shown in Table 6-6. The initial and final user input parameters are listed in Table 7-5. Calibration trials were made with simulation periods of 5 years in order reduce calibration time. All trials and their corresponding parameter changes are summarized in Table 6-8.

Table 6-6 Influent and effluent median concentrations for stormwater wetland basins as reported by the BMP database. The 25% percentile and 75% values are shown in parentheses (Leisenring et al. 2012).

Constituent	**Leisenring et al. (2012) – Chesapeake Bay area BMPDB**		
	In (mg/L)	**Out (mg/L)**	**Removal (%)**
BOD	---	---	---
TSS	43.2 (21.4-91.8)	15.2 (8.5-33.3)	64.8
TN	1.88 (1.06-2.52)	1.40 (0.84-2.27)	25.5
TKN	---	---	---
Organic N*	1.25	1.07	14.4
NH_3/NH_4	0.13 (0.08-0.24)	0.08 (0.04-0.18)	38.5
NO_3	0.50 (0.28-0.93)	0.25 (0.12-0.67)	50.0

* Organic nitrogen EMC values estimated by subtracting ammonia and nitrate values from total nitrogen assuming all corresponding values are based on the same volumes

Table 6-7 All user inputs for the stormwater wetland design, their assigned initial values, and final values after calibration.

User input	Initial value	Final value
Number of years of simulation	25	25
Contributing drainage area (sc)	5.3	5.3
Cell length (ft)	11.8	11.8
Number of cells in wetland design	26	26
FID vector	See Table 6-4	---
Vegetation specification for each cell (no vegetation = 0, emergent = 1, submerged = 1)	See Table 6-4	---
Initial water depth in each cell	See Table 6-4	---
Bottom elevation in each cell	See Table 6-4	---
Berm height at exit of each cell	See Table 6-4	---
Orifice or Weir (Orifice = 1, Weir = 2)	1	1
Orifice area (ft^2)	0.0491	0.0491
10-yr weir length (ft)	0.605	0.605
100-yr weir length (ft)	17.5	17.5
Orifice invert height H_I (ft)	5.75	5.75
10-yr weir invert height (ft)	2.56	2.56
100-yr weir invert height (ft)	4.64	4.64
Hydraulic conductivity K_V (ft/d)	0	0
Shelter factor f_s	0.75	0.75
Wetland albedo a	0.159	0.159
Leaf area index LAI	6.5	6.5
Maximum leaf conductance C^*_{leaf} (mm/s)	9.7	9.7
Emergent vegetation height z_v (m)	1.65	1.65
Wind speed measurement height z_m (m)	2	2
Maximum photosynthesis rate $PMAX$ (mg-O_2/m^2-hr)	910	910
TSS particle diameter D (m)	$9.5x10^{-6}$	$1.2x10^{-6}$
Initial water temperature $T_{w(o)}$ (°C)	15.5	15.5
Nitrification reaction rate K_{NIT} (hr^{-1})	0.01	0.004
Denitrification reaction rate K_{DNT} (hr^{-1})	0.0208	0.055
TSS wetland background concentration TSS_o (mg/L)	3	3
NH_4^+ wetland background concentration $NH4_o$ (mg/L)	0	0
NO_3^- wetland background concentration $NO3_o$ (mg/L)	0	0
DO initial concentration in wetland DO_o (mg/L)	7.5	7.5
Influent DO concentration DO_{in} (mg/L)	7.5	7.5
Influent TSS concentration TSS_{in} (mg/L)	43.2	43.2
Influent NH_4^+ concentration $NH4_{in}$ (mg/L)	0.13	0.13
Influent NH_3^- concentration $NO3_{in}$ (mg/L)	0.50	0.50
Wetland perimeter (ft)	259	259
Number of wetland habitat types	3	3
Number of habitat islands	0	0
Goal high-marsh design depth (ft)	1.5	1.5
Goal low-marsh design depth (ft)	3	3

Table 6-8 Stormwater wetland calibration trials and corresponding results.

Trial	Change made	Change rationale	Mean annual rainfall (in.)	Mean annual inflow (in.)	Mean annual outflow (in.)	Mean annual ET (in.)	Daily mean effluent TSS conc. (mg/L)	Daily mean effluent NH_4^+ conc. (mg/L)	Daily mean effluent NO_3^- conc. (mg/L)
1	---	---	39.1	14.0	13.9	31.1	3.02	0.05	0.36
2	TSS particle diameter decreased from 9.5x10^{-6} m to 1.0 x10^{-6} m	Increase effluent TSS concentrations	45.4	16.2	16.2	30.8	18.8	0.05	0.36
3	TSS particle diameter increased from 1.0x10^{-6} m to 1.2 x10^{-6} m	Decrease effluent TSS concentrations	43.2	15.4	15.4	31.2	15.2	0.05	0.36
4	K_{NIT} decreased from 0.01 to 0.008 hr^{-1}	In crease NH_4^+ concentrations	40.7	14.5	14.5	31.0	15.3	0.05	0.36
5	K_{NIT} decreased from 0.008 to 0.004 hr^{-1}	In crease NH_4^+ concentrations	44.5	15.9	15.9	31.0	16.3	0.08	0.35
7	K_{DNT} increased from 0.0208 to 0.05 hr^{-1}	Decrease NO_3^- concentrations	42.5	15.2	15.1	31.0	15.6	0.08	0.26
8	K_{DNT} increased from 0.05 to 0.055 hr^{-1}.	Decrease NO_3^- concentrations	37.6	13.4	13.3	31.2	15.2	0.08	0.25

6.9 STORMWATER WETLAND PERFORMANCE EVALUATION

The overall performance of the stormwater wetland design was evaluated through the computation and assessment of performance criteria (PC) and metric values based on the PC and PTM functions defined in Section 3.4. A total of five final Wetland Sustainability Indices (WSI's) were then computed resulting from (1) equally weighting all metrics, (2) weighting only water quality metrics, (3) weighting only flood control and downstream hydrologic regime metrics, (4) weighting all water quality and hydrologic metrics, and (5) weighting only wildlife habitat and aesthetics metrics. The metric weighting scheme 4 was referred to as the Best Management Practice (BMP) weighted design, as it equally weighted all water quality and hydrologic metrics, which were the metrics on which BMP design is focused. Because the base stormwater wetland design was a BMP facility designed according to MDE requirements, this weighting scheme was assumed to align best with the intended functions of the wetland.

6.9.1 Performance criteria and metric computation

A total of 16 performance criteria (PC values) were developed (see Section 3.4) in order to quantify the performance of a given wetland design with respect to the seven wetland functions defined in the current study. These wetland functions were (1) wildlife habitat, (2) flood control, (3) downstream hydrologic regime maintenance, (4) wetland water balance maintenance, (5) groundwater recharge and baseflow maintenance, (6) aesthetics, and (7) water quality. Within the current section, all 16 of these PC values for the base stormwater wetland design were computed. All wetland outputs relevant to PC computation are summarized in Table 6-9; resulting PC values are shown in Table 6-10.

Table 6-9 All relevant outputs resulting from the 25-yr simulation of the base stormwater design discussed in Section 6.11.

Model output	**Base design**
Mean daily effluent TSS Conc. (mg/L)	15.4
Mean daily effluent DO Conc. (mg/L)	10.3
Mean daily effluent NH4 Conc. (mg/L)	0.0782
Mean daily effluent NO3 Conc. (mg/L)	0.258
Mean Annual Rainfall depth (in.)	43.4
Mean annual ET depth (in.)	31.0
Mean Annual Inflow depth (in.)	15.5
Mean Annual Outflow depth (in.)	15.5
Mean Annual Infiltration depth (in.)	0
Annual Mean Influent volume (ac-ft)	6.85
Annual Mean Effluent volume (ac-ft)	6.93
Annual Mean pre-development volume (ac-ft)	2.65
Mean high-marsh water depth (ft)	0.52
Mean low-marsh water depth (ft)	1.27
Daily Influent flow CV	2.10
Daily Effluent flow CV	0.73
Daily pre-development flow CV	2.10
2-yr Inflow rate (cfs)	0.947
2-yr Outflow rate (cfs)	0.338
2-yr pre-development rate (cfs)	0.366
Mean inflow non-zero flow days	105
Mean outflow non-zero flow days	262
Mean pre-development non-zero flow days	105
Influent zero-flow exceednce probability	0.128
Effluent zero-flow exceedence probability	0.575
Pre-development zero-flow exceedence probability	0.128
Influent bankfull flow exceedence probability	0.0152
Effluent bankfull flow exceedence probability	0.000797
Pre-development bankfull flow exceedence probability	0.000252

Table 6-10 All computed performance criteria (PC) values computed for the base stormwater wetland design.

Performance criterion	Base design
Mean daily TSS conc. (mg/L)	15.4
Mean daily DO conc. (mg/L)	10.3
Mean daily NH4 conc. (mg/L)	0.0782
Mean daily NO3 conc. (mg/L)	0.258
Vegetative cover PC	0.68
Habitat island PC	0
High-marsh PC	0.962
Low-marsh PC	0.983
GW recharge PC	0.00
Wetland perimeter PC (ft)	1.24
Wetland diversity PC	3
Wetland area PC (acres)	0.0795
High-flow PC	0.316
Low-flow PC	0.223
Flow variation PC	2.90
Flood control PC	0.382

6.9.1.1 Wildlife habitat

Two wildlife habitat PC values were defined in the current study (see Section 3.4.1), one of which evaluated the proportion of emergent vegetated area in the wetland available for marsh wren habitat (PC_{H1}) and the second of which evaluated the number and distribution of habitat islands in a given wetland design (PC_{H2}). A total of 17 out of 25 cells had emergent vegetation in the base design, resulting in a PC_{H1} of 17/25 or 0.680. The base design did not, however, include any habitat islands, and, therefore, had a PC_{H2} value of 0. Applying the corresponding habitat PTM relationships defined in Equations 3-4 and 3-5, corresponding habitat metrics M_{H1} and M_{H2} of 0.712 and 0 were computed:

$$M_{H1} = \frac{1}{1+10890\exp(-15\cdot PC_{H1})} = \frac{1}{1+10890\exp(-15\cdot 0.680)} = 0.712 \qquad (6\text{-}34)$$

$$M_{H2} = 0 \qquad (6\text{-}35)$$

Therefore, while the base stormwater wetland design performed reasonably well with respect to wren marsh habitat, it did not provide any additional waterfowl habitat via habitat islands. Based on these habitat metrics, the stormwater wetland design does not represent an optimal design for the sustainable maintenance of wildlife habitat as defined in the current study.

6.9.1.2 Flood control

The flood control performance criterion PC_{FC} was equal to the proportion of computed pre-development mean annual volume $\overline{V}_{PRE}$ to that of the wetland outflow $\overline{V}_{OUT}$ (see Section 3.4.2). This ratio evaluated the extent to which annual wetland effluent volumes compared with annual volumes estimated to runoff from an analogous pre-developed area. The base stormwater wetland design computed $\overline{V}_{OUT}$ and $\overline{V}_{PRE}$ values of 302,020 and 115,279 ft^3 (6.93 and 2.65 ac-ft), which resulted in a PC_{FC} of 0.382:

$$PC_{FC} = \frac{\overline{V}_{PRE}}{\overline{V}_{OUT}} = \frac{115{,}279\ \text{ft}^3}{302{,}020\ \text{ft}^3} = 0.382 \qquad (6\text{-}36)$$

Therefore, $\overline{V}_{PRE}$ only represents 38.2% of $\overline{V}_{OUT}$, indicating that the wetland produces 61.8% more runoff than pre-developed conditions. Because PC_{FC} was less than 2, the final flood control metric M_{FC} was computed according to Equation 3-7:

$$M_{FC} = -PC_{FC}^2 + 2\cdot PC_{FC} = -(0.382^2) + 2\cdot 0.382 = 0.618 \qquad (6\text{-}37)$$

These resulting PC_{FC} and M_{FC} suggest that the performance of the stormwater wetland design was not effective in reducing the runoff volume entering the downstream natural area. This poor flood control performance is typical of stormwater wetlands, which are generally designed with a greater focus on water quality improvement and peak flow reduction than on volume reduction.

6.9.1.3 Downstream hydrologic regime

A total of three downstream hydrologic regime PC values ($PC_{H(H)}$, $PC_{H(L)}$, and $PC_{H(CV)}$) were developed in Section 3.4.3. The $PC_{H(H)}$, the high-flow PC, is a ratio of the proportion of 1-min pre-development to 1-min outflow discharge rates that exceeded the computed pre-development 2-yr (i.e., estimated bankfull flow) discharge for a given drainage area. The $PC_{H(L)}$, the low-flow PC, is a ratio of the proportion of 1-min pre-development to 1-min outflow flowrates that exceeded zero. Finally, the $PC_{H(CV)}$, the flow variation PC, represents the ratio of the mean pre-development daily flowrate coefficient of variation $\overline{CV}_P$ to the is the mean wetland effluent daily flowrate coefficient of variation $\overline{CV}_E$ over the simulation period. These three PC values were computed accordingly for the base stormwater wetland design:

$$PC_{H(H)} = \frac{p_{PRE(H)}}{p_{OUT(H)}} = \frac{0.000252}{0.000797} = 0.316 \tag{6-38}$$

$$PC_{H(L)} = \frac{p_{PRE(L)}}{p_{OUT(L)}} = \frac{0.128}{0.575} = 0.223 \tag{6-39}$$

$$PC_{H(CV)} = \frac{\overline{CV}_P}{\overline{CV}_E} = \frac{2.10}{0.727} = 2.90 \tag{6-40}$$

The same PTM functional form, which is shown in Equation 3-1, was used to convert all three downstream hydrologic regime PC values to their corresponding metrics $M_{H(H)}$, $M_{H(L)}$, and $M_{H(CV)}$:

$$M_{H(H)} = -PC_{H(H)}^2 + 2 \cdot PC_{H(H)} = -(0.335^2) + 2 \cdot 0.335 = 0.558 \tag{6-41}$$

$$M_{H(L)} = -PC_{H(L)}^2 + 2 \cdot PC_{H(L)} = -(0.223^2) + 2 \cdot 0.223 = 0.396 \tag{6-42}$$

$$M_{H(CV)} = 0 \tag{6-43}$$

These metric values revealed that while the wetland did a fair job of mitigating the frequency and duration of flow exceeding bankfull and a poor job of mimicking pre-development zero-flow frequency and duration. The wetland also failed to recreate the variation in flow observed under pre-developed conditions.

These discrepancies between wetland outflow and pre-development hydrologic regimes are due to the manner in which the wetland outlet orifice and weir structure controls flow. The outlet orifice was designed according to MDE (2009) requirements to reduce effluent flowrates to a maximum effluent rate of 0.379 cfs. However, because the wetland did not reduce influent volumes proportionally to these reduced peak flows, the resulting effluent hydrologic regime did not match that of pre-development conditions. Therefore, while peak effluent flowrates were restricted by orifice and weir flow, higher flows were maintained over longer durations than under corresponding pre-development conditions. As a result of these longer flow durations, wetland effluent flows exceeded zero 57.5% of the time or an average of 262 d/yr while pre-developed flows only exceeded zero 12.8% of the time or 105 d/yr. Because flows were maintained over longer durations, the day-to-day variation in effluent flowrates, which was represented by a $\overline{CV}_E$ of 0.727, was much lower than that of pre-developed conditions, which was represented by a $\overline{CV}_P$ of 2.90. The three hydrologic regime metrics were designed to evaluate the frequency, duration, and magnitude of wetland effluent flow as it compared to pre-developed values with the hopes of promoting stormwater wetlands that better mimic pre-developed hydrologic conditions.

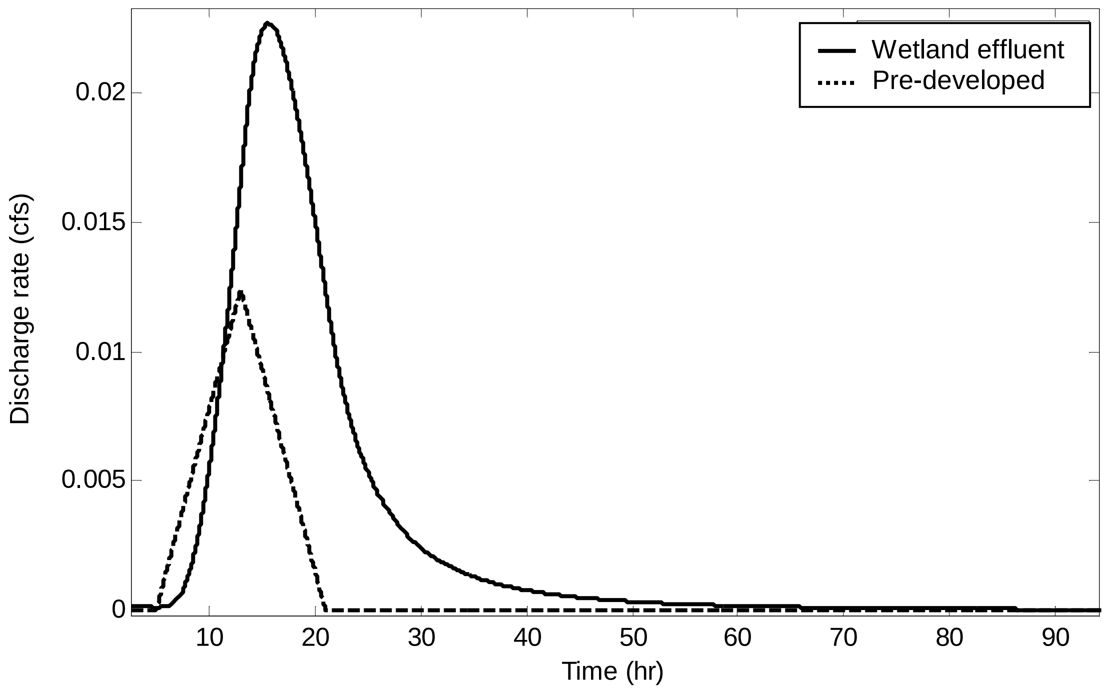

Figure 6-6 Comparison of example effluent (solid line) and pre-developed (dashed line) hydrographs. The pre-developed hydrograph has a total duration of 16 hours while the effluent hydrograph flows over a total of 125 hours. The effluent volume is also almost double that of the pre-developed runoff.

6.9.1.4 Wetland water balance

Two wetland water balance performance criteria, $PC_{WB(H)}$ and $PC_{WB(L)}$, were defined in Section 3.4.4. $PC_{WB(H)}$ is the ratio of a user-defined goal high-marsh water depth to the actual mean high-marsh water depth over the simulation period. $PC_{WB(L)}$ is the ratio of a user-defined low-marsh depth to the actual simulated mean low-marsh depth. Both wetland water balance PC values were computed according to Equations 3-17 and 3-18:

$$PC_{WB(H)} = \frac{SS_{GOAL(H)}}{\overline{SS}_H} = \frac{0.50}{0.52} = 0.962 \qquad (6\text{-}44)$$

$$PC_{WB(L)} = \frac{SS_{GOAL(L)}}{\overline{SS}_L} = \frac{1.25}{1.27} = 0.983 \qquad (6\text{-}45)$$

Given than optimal $PC_{WB(H)}$ and $PC_{WB(L)}$ values were 1.0, these values show that the wetland maintained design water levels very well over the simulation. Final wetland water balance metrics $M_{WB(H)}$ and $M_{WB(L)}$ were also computed using the same PTM relationship (see Equation 3-1) as all of the downstream hydrologic regime PC values:

$$M_{WB(H)} = -PC_{WB(H)}^2 + 2 \cdot PC_{WB(H)} = -(0.962^2) + (2 \cdot 0.962) = 0.999 \qquad (6\text{-}46)$$

$$M_{WB(L)} = -PC_{WB(L)}^2 + 2 \cdot PC_{WB(L)} = -(0.983^2) + (2 \cdot 0.983) = 1.00 \qquad (6\text{-}47)$$

These resulting metrics also show that the base wetland design maintained the design water levels very well over the 25-yr simulation period.

6.9.1.5 Groundwater recharge and baseflow maintenance

The current study developed one groundwater recharge and baseflow maintenance PC value, PC_{GW} (see Section 3.4.5), which was a ratio comparing the estimated pre-development annual infiltration over the drainage area with the simulated annual infiltration volume within the wetland. The base stormwater design incorporated an impervious liner, and therefore, did not provide any infiltration. As a result, the PC_{GW} and corresponding metric for the base design were both 0 based on Equations 3-231 and 3-24.

6.9.1.6 Aesthetics

Three aesthetics performance criteria were specified in Section 3.4.6, which evaluate the extent to which the (1) wetland perimeter irregularity PC_{PI}, (2) wetland-type diversity PC_{WD}, and (3) total wetland surface area PC_A contributed to the overall aesthetic appeal of a given wetland design. The following aesthetic PC values were computed for the base stormwater wetland design according to Equations 3-25, 3-26, and 3-27:

$$PC_{PI} = \frac{PR}{2\sqrt{\pi(43{,}560 \cdot A_w)}} = \frac{259\,\text{ft}}{2\sqrt{\pi(43{,}560 \cdot 0.0795\,\text{ac})}} = 1.24 \qquad (6\text{-}48)$$

$$PC_{WD} = N = 3 \qquad (6\text{-}49)$$

$$PC_A = A_w = 0.0795\,\text{ac} \qquad (6\text{-}50)$$

where PR is the wetland perimeter (ft), A_w is the wetland surface area in acres, and N is the total number of wetland habitat types present in the wetland design. The wetland perimeter PR was equal to 259 ft and the area A_w was 0.0795 ac (3,463 ft^2). The base design was also assumed to have three total wetland habitat types N, which were high-marsh areas, low-marsh areas, and areas with water depths between 1.5 and 4 ft. Deepwater areas were not included in N as they did not provide habitat, but were rather intended to provide areas for additional TSS settling. From these PC values, final aesthetics metrics M_{PI}, M_{WD}, and M_A were then computed based on Equations 3-28, 3-29, and 3-30:

$$M_{PI} = \frac{PC_{PI}}{5} = \frac{1.24}{5} = 0.248 \qquad (6\text{-}51)$$

$$M_{WD} = \frac{PC_{WD}}{5} = \frac{3}{5} = 0.60 \quad (6\text{-}52)$$

$$M_A = \frac{PC_A}{10} = \frac{0.0795\,\text{ac}}{10\,\text{ac}} = 0.00795 \quad (6\text{-}53)$$

These final metrics indicate that the base wetland design did not have a large aesthetic value with respect to perimeter irregularity and surface area. It did, however, provide some variation in wetland habitat types, which resulted in a final M_{WD} of 0.60. Overall, because aesthetics were the not a main intended function of this stormwater wetland, these poor aesthetics metrics were excepted.

6.9.1.7 Water quality

Water quality PC values were defined in Section 3.4.7 to equal the mean daily effluent TSS, dissolved oxygen, NH_4^+, and NO_3^- concentrations over the simulation period, which were respectively 15.4, 10.3, 0.0782, and 0.258 mg/L and are compiled in

Table **6-10**. From these concentrations, Equations 3-32 through 3-34 were used to compute corresponding water quality metrics M_{NO3}, M_{NH4}, M_{DO}, and M_{TSS}:

$$M_{NO3} = 1 \quad (6\text{-}54)$$

$$M_{NH4} = 1.15 \cdot \exp(-9.96 \cdot NH4) = 1.15 \cdot \exp(-9.96 \cdot 0.0782) = 0.528 \quad (6\text{-}55)$$

$$M_{DO} = 1 \quad (6\text{-}56)$$

$$M_{TSS} = 3 \cdot TSS^{-0.81} = 3 \cdot (15.4^{-0.81}) = 0.328 \quad (6\text{-}57)$$

These metrics were based on a study done by McNett et al. (2010) that related water quality concentrations with different levels of natural stream benthic health. The resulting metrics showed that the wetland did an excellent job of achieving healthy effluent concentrations of DO and NO_3^-. However, effluent TSS and NH_4^+, concentrations were still high compared to healthy levels reported by McNett et al. (2010) to correspond with natural streams.

6.9.1.8 Final WSI computations

Once all performance criteria and metrics were computed for the base wetland design, a total of five metric weighting schemes were developed in order to show to effect of weighting on the final Wetland Sustainability Index (WSI). As defined in Section 3.5, the WSI is the weighted sum of all wetland metrics. The weights assigned to each metric are subject to user discretion based on the intended function(s) of a given wetland design. Therefore, weighting schemes were developed within the current section to compute the final WSI of the base design with respect to (1) all wetland functions, (2) water quality functions, (3) flood control, wetland water balance, and downstream hydrologic regime functions (i.e., all relevant hydrologic functions), (4) habitat and aesthetic functions, and (5) both water quality and hydrologic functions. The final weighting scheme, referred to as the BMP weighting scheme, was assumed to be most relevant to the base design as its main purpose was to control the effluent hydrologic regime, prevent downstream flooding, and to improve stormwater water quality. The BMP weighting scheme was further used in the sensitivity analyses of all design criteria and input parameters performed later in this section.

As shown in Table 6-11, the five different weighting schemes resulted in varying WSI scores for the base wetland design. The wetland performed worse, with a WSI of 0.314, when only habitat and aesthetic metrics were weighted. This low WSI score reflects the poor aesthetics and habitat metrics produced by the wetland. Conversely, the wetland performed best when only water quality metrics were weighted, with a WSI of 0.714. The final BMP weighting scheme resulted in a WSI of 0.640, which reflected good water quality performance and fair hydrologic performance. Therefore, while stakeholders most interested in wetland habitat and aesthetics would be disappointed with this wetland design, those concerned solely with water quality performance would favorably assess the performance. Based on these results, the WSI weighting method appears to reflect the performance of the wetland with respect to all defined wetland functions.

While the different weighting schemes were successful in producing WSI scores indicative of the performance of the wetland with respect to different wetland functions, the significance of the differences between these scores remains to be defined. As defined in Sections 3.4 and 3.5, both the performance metrics and WSI scores had ranges of 0 to 1, with 0 representing failure to meet a given function and 1 representing optimal performance of that function. However, it is difficult to define what represents a significant difference in these metrics and resulting WSI scores. For example, if two wetland designs return respective WSI scores of 0.600 and 0.700, does the second design perform significantly better than the first? Answering this question requires detailed knowledge of (1) the intended functions for the proposed wetland and (2) the sensitivity of these wetland functions to the differences observed in the WSI scores of the two wetlands. While intended wetland functions can be well defined, it often difficult to define the sensitivity of such functions due to lack of data.

The current study explores this issue in the following sections as well as in Chapters 7, 8, and 9 by computing the PC values, metrics, and corresponding WSI scores for a number of different wetland designs and types.

Table 6-11 Computed metrics, weights and final Wetland Sustainability Indices (WSI's) resulting from three different metric weighting schemes.

Performance criterion	Raw metrics	Weights				
		Equally-weighted	Water quality-weighted	Hydrology-weighted	Habitat/ Aesthetic-weighted	BMP-weighted
Mean daily TSS Conc. (mg/L)	0.328	0.0625	0.25	0	0	0.1
Mean daily DO Conc. (mg/L)	1	0.0625	0.25	0	0	0.1
Mean daily NH4 Conc. (mg/L)	0.528	0.0625	0.25	0	0	0.1
Mean daily NO3 Conc. (mg/L)	1	0.0625	0.25	0	0	0.1
Vegetative cover PC	0.712	0.0625	0	0	0.2	0
Habitat Island PC	0	0.0625	0	0	0.2	0
High-marsh PC	0.999	0.0625	0	0.167	0	0.1
Low-marsh PC	1.000	0.0625	0	0.167	0	0.1
GW Recharge PC	0.000	0.0625	0	0	0	0
Wetland Perimeter PC (ft)	0.248	0.0625	0	0	0.2	0
Wetland Diversity PC	0.600	0.0625	0	0	0.2	0
Wetland Area PC (acres)	0.008	0.0625	0	0	0.2	0
High-marsh PC	0.533	0.0625	0	0.167	0	0.1
Low-Flow PC	0.396	0.0625	0	0.167	0	0.1
Flow Variation PC	0.000	0.0625	0	0.167	0	0.1
Flood Control PC	0.618	0.0625	0	0.167	0	0.1
Final *WSI* score	---	0.498	0.714	0.591	0.314	0.640

6.10 STORMWATER WETLAND DESIGN SENSITIVITY ANALYSIS

A sensitivity analysis was performed on the MDE-based stormwater wetland designed in Section 6.4 with the final objectives:

1. To assess the importance of design criteria changes (e.g., decreasing high-marsh area from 35% to 40%, removing the forebay, etc.) on resulting model performance criteria.

2. To assess the importance of model inputs (e.g., drainage area, runoff coefficient, leaf area index, etc.) on resulting model performance criteria.

Within the current study, the importance of a given input parameter or design criteria with respect to each wetland performance criterion was quantified as the relative sensitivity (S_X):

$$S_X = \frac{\delta PC / PC}{\delta X / X} \tag{6-58}$$

where *X* is the input parameter or design criteria for a given simulation, PC is the model performance criteria, and S_X is the model relative sensitivity (dimensionless) with respect to PC. As a review, the final model sustainability metrics are normalized (range from 0 to 1) representations of raw model performance criteria. The current study performed a sensitivity analysis on the raw performance criteria as the relationships linking the performance criteria and final, normalized metrics are subject to changes made by the model user.

In addition to the S_X values, deviation sensitivity D_X values were also computed for each input parameter/design criteria and output PC pair:

$$D_X = \frac{\delta PC}{\delta X} \Delta X = \delta PC \tag{6-59}$$

where ΔX, with the same units as *X*, represents the error associated with X. Because ΔX and δX were both equal to the introduced deviation in *X*, the final D_X values were equal to the resulting deviation observed in a given PC value. Therefore, D_X values had units equal to those of the corresponding PC value.

6.11 STORMWATER WETLAND DESIGN CRITERIA

The example stormwater wetland design used in the sensitivity analysis was designed according to the specifications and procedures developed by the Maryland Department of the Environment (MDE). Each step of this design was recorded and discussed in detail in Section 6.4. A required wetland surface area SA_o of 3,463 ft^2 (1.5% of the drainage area) and a water quality volume WQ_v, the MDE-specified minimum wetland volume for stormwater quality improvement, of 5,396 ft^3 (0.124 ac-ft) was computed according to MDE (2009) specifications in Section 6.4. The final stormwater wetland design met all MDE-specified design criteria (see Table 6-12).

The final wetland design included a total of four MDE-required wetland zones, the forebay, micropool, high-marsh, and low-marsh (see Table 6-13). The forebay was designed with a depth of 4 ft, which resulted in a forebay volume that was 10.3% of the total WQ_v while a volume of greater or equal to 10% was required by MDE (2009). The micropool had a depth of 5.75 ft. Together, the forebay and the micropool composed all of the deepwater (i.e., water depths $\geq$ 4 ft) areas within the wetland and accounted for 25.0% of the WQ_v, which met the MDE requirement that deepwater areas must compose at least 25% of the WQ_v. High-marsh areas, which had water depths less than or equal to 0.5 ft, accounted for a total of 36% of the MDE-required surface area SA_o, meeting the MDE-required proportion of 35%. Low-marsh areas, which had water depths between 0.5 and 1.5 ft, accounted for a total of 32% of the SA_o. The total high and low-marsh area was 68% of the SA_o, which met the MDE-required proportion of 65%. In order to meet the WQ_v required storage, the remaining wetland volume was allotted to water with a depth of 3 ft, which did not match any of the MDE-defined depth categories. MDE (2009) did not specify any requirements for such areas with water depths between 1.5 and 4 ft. Therefore, it was assumed acceptable to include these 3-ft depth areas in order to meet the required WQ_v while also matching the minimum SA_o proportions.

Table 6-12 MDE (2009) wetland criterion and corresponding values for stormwater wetland designed in current study.

Wetland feature	MDE (2009) criterion	Current study design
Wetland surface area	≥ 1.5% contributing drainage area (SA_o)	1.5% contributing drainage area
Storage volume at normal pool level	WQ_v (5,396 ft^3)	5,853 ft^3
High-marsh (water depths ≤ 0.5 ft)	≥ 35% of required wetland surface area SA_o	36% of SA_o
High + Low-marsh (water depths ≤ 1.5 ft)	≥ 65% of required wetland surface area SA_o	68% of SA_o
Forebay volume	≥ 10% of WQ_v	10.3% of WQ_v
Deepwater zones (water depths ≥ 4 ft)	≥ 25% of WQ_v	25.0% of WQ_v

Table 6-13 Wetland zones and corresponding depths as defined by MDE (2009).

Wetland Zone	MDE (2009) depth definition
High-marsh	≤ 0.5 ft
Low-marsh	> 0.5 ft and ≤ 1.5 ft
Deepwater	≥ 4 ft
Micropool	3-6 ft
Forebay	No depth specifications
Other	> 1.5 ft and < 4 ft

6.12 SIMULATION PERIOD DETERMINATION

It was important to find a simulation period that sufficiently characterized the hydrologic regime of the wetland and its contributing drainage area. However, there was a computational price to be paid for longer simulation periods. While the 25-cell stormwater wetland model structure discussed in Section 6.11 takes less than one minute to simulate 1 year of flow, it requires about 1.57 hours to simulate 100 years (see Table 6-14). The current study performed a sensitivity analysis to determine a model simulation period that fully characterized the hydrology of the wetland while minimizing computational time.

Table 6-14 Simulation periods and corresponding computation times for the 25-cell shallow stormwater wetland designed in the current study using a 64-bit operating system with a Intel® Core™ i5-4440 CPU at 3.10 GHz.

Simulation period (yrs)	Computation time (min)
1	0.951
5	4.66
10	9.50
25	23.4
50	46.3
100	94.2 (1.57 hr)

In order to evaluate the sensitivity of model performance to simulation period duration, flow was first simulated through the calibrated shallow stormwater wetland over a 100-yr period (see Figure 6-8). The annual maximum flowrates (cfs) for wetland inflow, wetland outflow, and the estimated pre-developed drainage area (including the wetland area) were output for each year of simulation. While both the inflow and pre-development annual maxima series followed similar trends with respective standard skews of -0.625 and -0.735, the wetland outflow annual maxima series had a positive skew of 0.609. The difference in the shape of the wetland outflow flood-frequency curve is due to the storage characteristics of the wetland cells and the characteristics of the outlet structure of the wetland.

The stormwater wetland outlet was designed according to MDE (2009) standards, with an orifice to control the 1-yr, 24-hr storm; a weir to control the 10-yr, 24-hr storm; and a second and final weir to transfer the 100-yr storm event through the wetland (see Figure 6-7). As described in Section 6.4, the outlet orifice had a diameter of 3 in., was situated at normal pool level, and was designed to control the 1-yr, 24-hr storm event. The 1-yr, 24-hr storm event was computed to equal 2.56 ft of storage above the normal pool level, with a corresponding outflow rate of 0.379 cfs as computed from the orifice equation:

$$q_o = A_o C_o \sqrt{2gh_o} = 0.0491\,\text{ft} \cdot 0.6\sqrt{2(32.2\,\text{ft/s}^2)(2.56\,\text{ft})} = 0.379\,\text{cfs} \qquad (6\text{-}60)$$

where h_o represents the maximum storage depth (ft) associated with the 1-yr, 24-hr storm (2.57 ft), A_o ,is the orifice area (ft^2), C_o is the orifice coefficient, and q_o is the maximum orifice flowrate (cfs). A weir with a length of 0.650 ft was situated 2.56 ft above the normal pool level (see Figure 6-7), which was designed to control the influent 10-yr, 24-hr storm event with an outflow rate of 6.42 cfs.

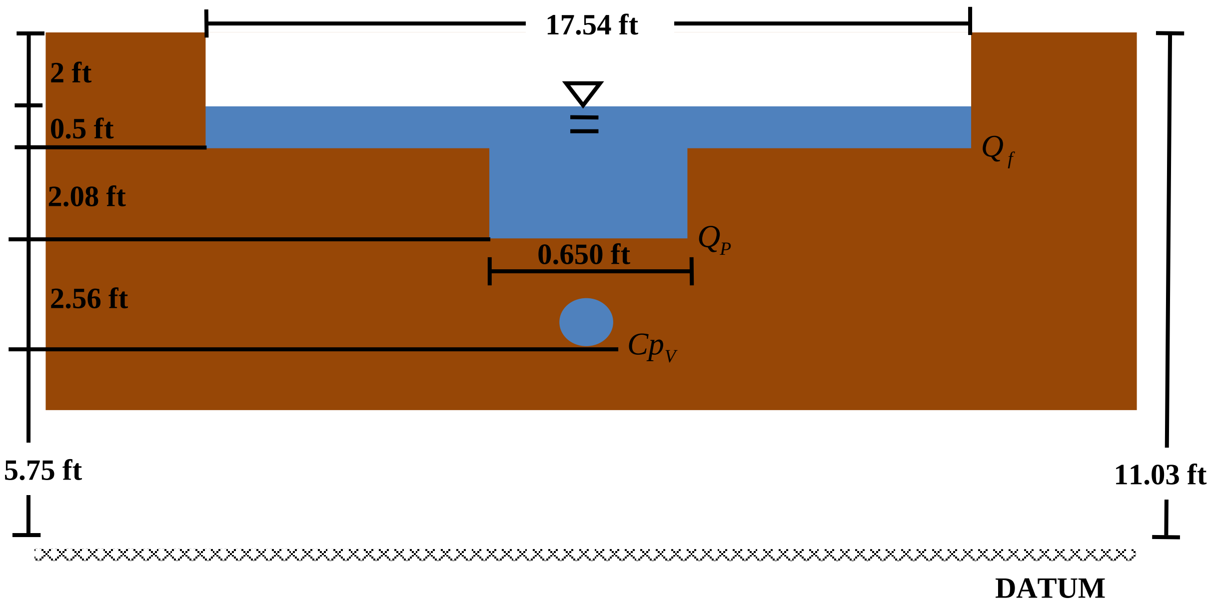

Figure 6-7 Outlet orifice and double riser design through the MDE (2009) method. The datum represents the bottom of the micropool, which was designed to have a depth of 5.75 ft. The designated Cp_V , Q_P , Q_f depths represent the water depths corresponding to the 1-yr, 10-yr, and 100-yr, 24-hr floods within the wetland.

The outflow annual maxima series experienced an abrupt increase in flowrates and slope after an exceedence probability of about 0.54, which corresponds to an outflow rate of 0.352 cfs. This jump in flowrates and change in slope in the outflow maxima series are due to the change from orifice flow, with a maximum computed flowrate of 0.379 cfs, to two-stage orifice -weir flow, with a maximum flowrate of 6 cfs. Therefore, as seen in Figure 6-8, wetland outflow rates approach those of inflow rates for storm events with flowrates and/or inflow volumes exceeding those of the 1-yr, 24-hr influent storm event. Figure 6-9 shows the resulting Log-Pearson III (LP3) curves and corresponding data points for weir outflow and orifice outflow rates, showing that the weir flow annual maxima series has a much steeper slope than that of the orifice flow annual series maxima. As with most BMP facilities, this stormwater wetland was designed with the purpose of controlling smaller storm events (with inflow storm events with recurrence intervals of 1 year or less).

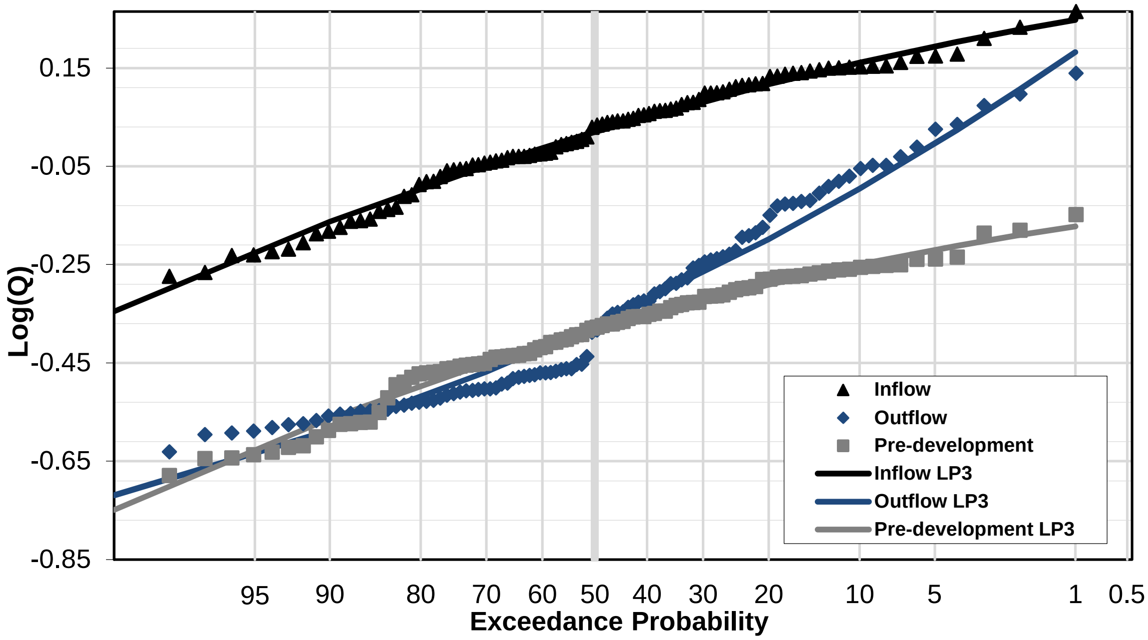

Figure 6-8 Resulting annual maxima for wetland inflow (black triangles), outflow (blue diamonds), and estimated pre-developed drainage area (grey squares) resulting from 100 years of simulation. Corresponding Log-Pearson III curves were computed for inflow (black line), outflow (blue line), and pre-developed (grey) annual maxima flow series.

The annual maxima series were analyzed separately as one 100-yr period, two consecutive 50-yr periods, and four consecutive 25-yr periods. Log-Pearson III curves were then computed for each of the seven resulting simulation periods (one 100-yr, two 50-yr, and four 25-yr periods). The generated LP3 curves for all simulation periods for inflow, outflow, and pre-developed conditions are plotted in Figures 6-10, 6-11, and 6-12. Curves from all simulation periods followed similar trends for each three flow series (i.e., inflow, outflow, and pre-development). As expected, the resulting 25-yr LP3 curves showed greater deviation from the 100-yr curve than the 50-yr curves. Therefore, as sample size decreases, variation increases.

In order to evaluate the significance of the variation observed between the 100, 50, and 25-yr LP3 curves for each flow series, the 100, 50, 25, and 2-yr flows were computed from the corresponding LP3 for all simulation periods and all flow series. Computed flows for all 50 and 25-yr simulation periods were also compared with corresponding computed flows generated from the 100-yr simulation period in the form of relative errors e/Y_{100}:

$$e/Y_{100} = \frac{Y_s - Y_{100}}{Y_{100}} \tag{6-61}$$

where Y_{100} represents the flowrate (cfs) for a given LP3 return period (i.e., 2, 25, 50, or 100 yrs) computed from the 100-yr simulation period and Y_S is the flowrate (cfs) for a given LP3 return period computed from one of the six other simulation periods (two 50-yr and four 25-yr simulation periods). Return period flows and e/Y_{100} for all simulation periods are summarized in Table 6-15 for wetland inflow, in Table 6-16 for wetland outflow, and Table 6-17 for pre-developed conditions. Resulting e/Y_{100} values had magnitudes of less than 25% for all simulation and computed return periods. More variation between simulation periods was observed in wetland outflow values (maximum e/Y_{100} of 0.239 for the 100-yr flow generated from the 25-yr simulation period #1) versus the inflow and pre-development values with corresponding e/Y_{100} values of 0.161 and 0.119 for the same 100-yr flow for the 25-yr simulation period #1. This increased variation in wetland outflow rates is likely due to the increased flowpath complexity within the wetland. While inflow and pre-developed flowrates depend directly on rainfall rates and their respective runoff coefficients of 0.36 and 0.137, outflow rates depend on rainfall rates, initial wetland storage, and ET rates. Relative errors e/Y_{100} for LP3 2-yr return period flowrates were low for all flow series, with maximum respective values of 0.059, 0.083, and 0.065 for inflow, outflow, and pre-development LP3 curves.

Given these results, the 25-yr simulation period appears to sufficiently characterize the hydrology of the site based on the relatively low (within the context of hydrology) variation observed between the 100-yr and 25-yr generated LP3 curves and return period flood events. However, it is difficult to predict the magnitude of these variations with respect to the variations that will be produced by changing input parameters and design criteria. Ideally, the hydrologic variation between simulations would be zero, which would guarantee that parameter/design criteria changes were responsible for all variations in simulation runs. Given this concern, a large number of the performance criteria outputs were designed as ratios of pre-development over outflow hydrologic values. These PC values should not be greatly affected by variation between simulations, as both the pre-development and the outflow hydrology would experience the same conditions (although extreme outflow and pre-development values may not necessarily be produced by the same storm event). The comparative nature of these PC ratios should, in general, normalize hydrologic variations between simulations. Additionally, the water quality PC values are the daily effluent TSS, NH_4^+, and NO_3^- concentrations, which seemed to be fairly resilient to hydrologic variation through calibration (see Table 6-8) with 5-yr simulation periods. The only PC values of concern are the wetland water balance PC values, which measure the mean high and low-marsh water depths. While the wetland is dependable in maintaining the design depths, dry periods and large storm events can temporarily decrease or increase water depths. However, over a simulation period of 25 years, it may be reasonable to assume that these fluctuations would equilibrate, even with extreme events.

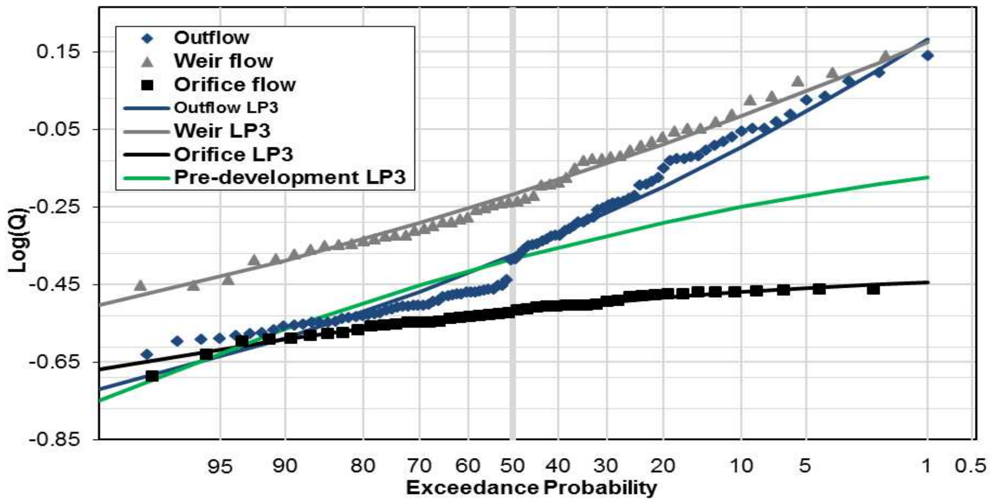

Figure 6-9 Annual maxima series for outflow (blue diamonds), all outflow values with weir flow (grey triangles), and outflow flows with orifice flow (black squares). Corresponding Log-Pearson III curves were computed for the total outflow series (blue line), weir flow outflow (grey line), orifice flow outflow (black line), and pre-developed (green line).

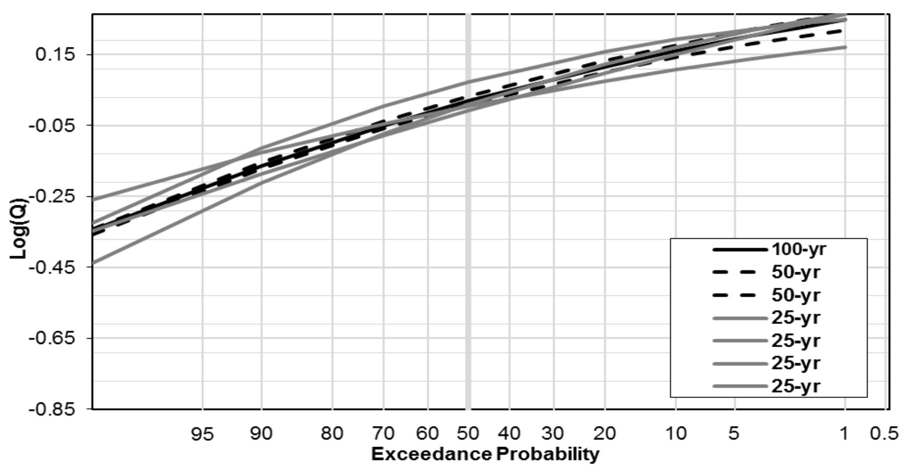

Figure 6-10 Computed Log-Pearson III curves for inflow annual maximum series based simulation periods of 100 yr (solid black line), 50 yr (dashed black lines), and 25 yr (grey lines). All simulation periods were taken from one 100-yr simulation. Therefore, two 50-yr and four 25-yr periods are shown.

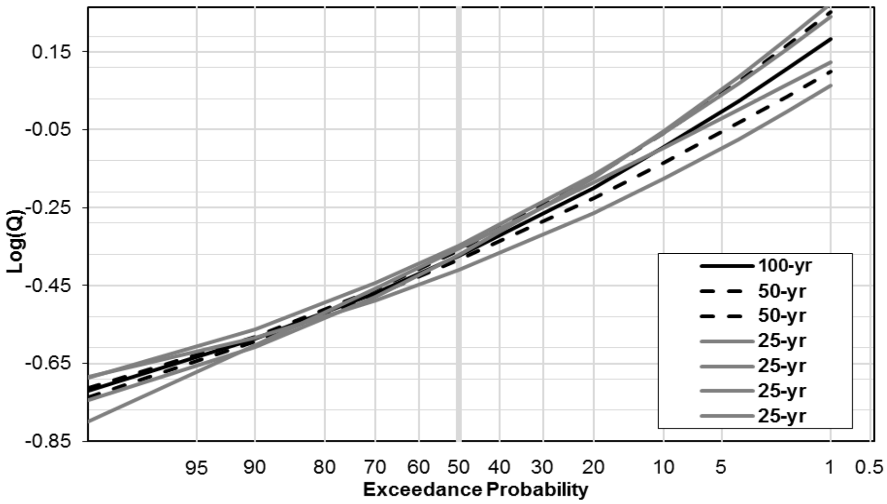

Figure 6-11 Computed Log-Pearson III curves for outflow annual maximum series based simulation periods of 100 yr (solid black line), 50 yr (dashed black lines), and 25 yr (grey lines). All simulation periods were taken from one 100-yr simulation. Therefore, two 50-yr and four 25-yr periods are shown.

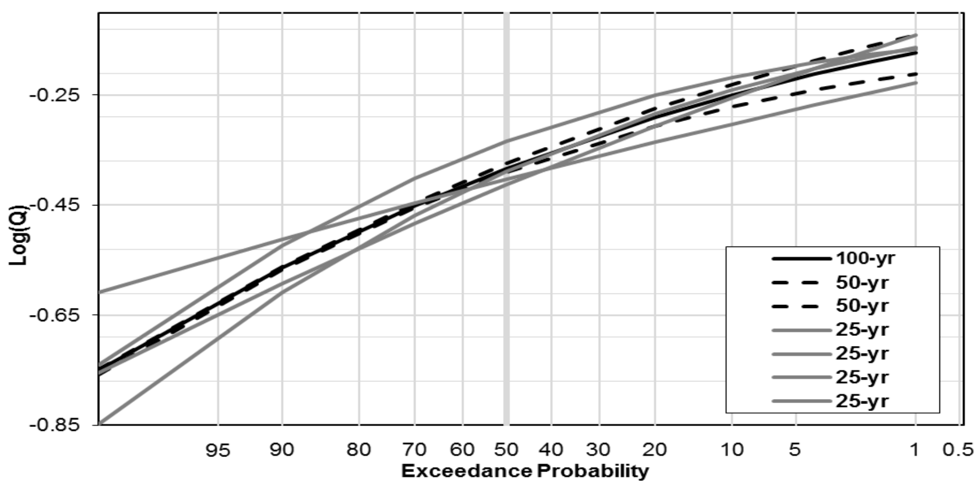

Figure 6-12 Computed Log-Pearson III curves for pre-development annual maximum series based simulation periods of 100 yr (solid black line), 50 yr (dashed black lines), and 25 yr (grey lines). All simulation periods were taken from one 100-yr simulation. Therefore, two 50-yr and four 25-yr periods are shown.

Table 6-15 LP3 curve computed 100, 50, 25, and 2-yr flowrates for wetland inflow for all seven simulation periods. Relative errors $\bar{e}/\bar{Y}$ values were computed with flowrates computed from the 100-yr simulation period as the "observed" values.

Inflow LP3 Return Period (yr)	**Simulation Time period (yrs)**												
	100	50 #1		50 #2		25 #1		25 #2		25 #3		25 #4	
	Flow (cfs)	Flow (cfs)	$\bar{e}/\bar{Y}$	Flow (cfs)	$\bar{e}/\bar{Y}$	Flow (cfs)	$\bar{e}/\bar{Y}$	Flow (cfs)	$\bar{e}/\bar{Y}$	Flow (cfs)	$\bar{e}/\bar{Y}$	Flow (cfs)	$\bar{e}/\bar{Y}$
100	1.77	1.66	-0.062	1.85	0.044	1.49	-0.161	1.85	0.045	1.78	0.006	1.84	0.038
50	1.69	1.59	-0.056	1.76	0.042	1.44	-0.150	1.76	0.042	1.73	0.026	1.72	0.020
25	1.60	1.52	-0.050	1.66	0.041	1.38	-0.136	1.65	0.037	1.67	0.047	1.60	0.002
2	1.05	1.02	-0.025	1.08	0.030	1.01	-0.031	1.02	-0.023	1.18	0.130	0.99	-0.059

Table 6-16 LP3 curve computed 100, 50, 25, and 2-yr flowrates for wetland outflow for all seven simulation periods. Relative errors $\bar{e}/\bar{Y}$ values were computed with flowrates computed from the 100-yr simulation period as the "observed" values.

Outflow LP3 Return Period (yr)	**Simulation Time period (yrs)**												
	100	50 #1		50 #2		25 #1		25 #2		25 #3		25 #4	
	Flow (cfs)	Flow (cfs)	$\bar{e}/\bar{Y}$	Flow (cfs)	$\bar{e}/\bar{Y}$	Flow (cfs)	$\bar{e}/\bar{Y}$	Flow (cfs)	$\bar{e}/\bar{Y}$	Flow (cfs)	$\bar{e}/\bar{Y}$	Flow (cfs)	$\bar{e}/\bar{Y}$
100	1.52	1.26	-0.175	1.79	0.176	1.16	-0.239	1.33	-0.126	1.88	0.236	1.74	0.142
50	1.27	1.08	-0.149	1.46	0.149	0.99	-0.221	1.16	-0.087	1.52	0.195	1.43	0.126
25	1.06	0.93	-0.123	1.19	0.124	0.84	-0.202	1.00	-0.049	1.22	0.154	1.17	0.110
2	0.42	0.41	-0.024	0.44	0.031	0.39	-0.083	0.44	0.042	0.42	0.002	0.45	0.060

Table 6-17 LP3 curve computed 100, 50, 25, and 2-yr flowrates for pre-developed conditions for all seven simulation periods. Relative errors $\bar{e}/\bar{Y}$ values were computed with flowrates computed from the 100-yr simulation period as the "observed" values.

Pre-development LP3 Return Period (yr)	**Simulation Time period (yrs)**												
	100	50 #1		50 #2		25 #1		25 #2		25 #3		25 #4	
	Flow (cfs)	Flow (cfs)	$\bar{e}/\bar{Y}$	Flow (cfs)	$\bar{e}/\bar{Y}$	Flow (cfs)	$\bar{e}/\bar{Y}$	Flow (cfs)	$\bar{e}/\bar{Y}$	Flow (cfs)	$\bar{e}/\bar{Y}$	Flow (cfs)	$\bar{e}/\bar{Y}$
100	0.67	0.62	-0.085	0.72	0.075	0.59	-0.119	0.69	0.020	0.68	0.013	0.72	0.075
50	0.65	0.60	-0.075	0.69	0.067	0.57	-0.121	0.66	0.023	0.67	0.031	0.68	0.049
25	0.61	0.57	-0.065	0.65	0.059	0.54	-0.121	0.63	0.025	0.64	0.050	0.63	0.024
2	0.41	0.41	-0.016	0.42	0.020	0.40	-0.044	0.41	-0.011	0.46	0.121	0.39	-0.065

6.12.1 Evaluating flow variation through culvert diameter design

Culvert diameters were designed from LP3-computed 100, 50, 25, 10, and 2-yr flows for all seven simulation periods that were discussed in the previous section. The resulting variation in design culvert diameters was used to determine the importance of the initial flow variation observed between simulation periods. The culvert diameters, D_C (ft) were designed for an unsubmerged inlet and outlet with the following Manning-derived equation (McCuen 2005):

$$D_C = 1.333(Q \cdot n)^{3/8} S^{-3/16} \tag{6-62}$$

where Q is the flowrate (cfs) for a given return period and simulation period, n is the pipe roughness coefficient, and S is pipe slope (ft/ft). Within this example, the pipe slope S was set equal to the drainage area slope of 0.013 ft/ft. The pipes were assumed to be corrugated metal with a pipe roughness coefficient n of 0.023 (McCuen 2005). Computed D_C values and corresponding ASTM standard corrugated pipe diameters are summarized for inflow, outflow, and pre-developed conditions in Tables 6-18, 6-19, and 6-20. All pre-development design flows yielded a design culvert diameter of 8 in. Additionally, the ranges in the computed D_C values across simulation periods for each return period were between 0.44 and 0.52 in. (0.0367 and 0.0433 ft). These small D_C ranges suggest that pre-development flows did not vary significantly between different simulation periods. Therefore, a 25-yr simulation period would be sufficient in characterizing pre-development hydrology.

Table 6-18 Computed culvert diameters and corresponding design pipe diameters (based on ASTM A760 standard corrugated pipe diameters) for wetland inflow for all LP3-compute return period flowrates and all seven simulation periods.

Inflow LP3 Return Period (yr)	**Simulation Time period (yrs)**													
	100		50 #1		50 #2		25 #1		25 #2		25 #3		25 #4	
	Culvert diameter		Culvert diameter		Culvert diameter		Culvert diameter		Culvert diameter		Culvert diameter		Culvert diameter	
	Computed (in.)	Design (in.)	Computed (in.)	Design (in.)	Computed (in.)	Design (in.)	Computed (in.)	Design (in.)	Computed (in.)	Design (in.)	Computed (in.)	Design (in.)	Computed (in.)	Design (in.)
100	10.8	12	10.6	12	11.0	12	10.2	12	11.0	12	10.9	12	11.0	12
50	10.7	12	10.4	12	10.8	12	10.0	12	10.8	12	10.8	12	10.7	12
25	10.4	12	10.2	12	10.6	12	9.87	10	10.6	12	10.6	12	10.4	12
10	10.1	12	9.90	10	10.2	12	9.62	10	10.2	12	10.3	12	9.98	10
2	8.91	10	8.82	10	9.01	10	8.80	10	8.83	10	9.33	10	8.71	10

Table 6-19 Computed culvert diameters and corresponding design pipe diameters (based on ASTM A760 standard corrugated pipe diameters) for wetland outflow for all LP3-compute return period flowrates and all seven simulation periods.

Outflow LP3 Return Period (yr)	**Simulation Time period (yrs)**													
	100		50 #1		50 #2		25 #1		25 #2		25 #3		25 #4	
	Culvert diameter		Culvert diameter		Culvert diameter		Culvert diameter		Culvert diameter		Culvert diameter		Culvert diameter	
	Computed (in.)	Design (in.)	Computed (in.)	Design (in.)	Computed (in.)	Design (in.)	Computed (in.)	Design (in.)	Computed (in.)	Design (in.)	Computed (in.)	Design (in.)	Computed (in.)	Design (in.)
100	10.2	12	9.54	10	10.9	12	9.25	10	9.74	10	11.1	12	10.8	12
50	9.59	10	9.03	10	10.1	12	8.73	10	9.27	10	10.3	12	10.0	12
25	8.94	10	8.51	10	9.33	10	8.21	10	8.77	10	9.43	10	9.29	10
10	8.07	8	7.79	8	8.33	10	7.52	8	8.06	8	8.36	10	8.33	10
2	6.34	8	6.29	8	6.42	8	6.14	8	6.44	8	6.35	8	6.48	8

Table 6-20 Computed culvert diameters and corresponding design pipe diameters (based on ASTM A760 standard corrugated pipe diameters) for wetland outflow for all LP3-compute return period flowrates and all seven simulation periods.

Pre-developed LP3 Return Period (yr)	**Simulation Time period (yrs)**													
	100		50 #1		50 #2		25 #1		25 #2		25 #3		25 #4	
	Culvert diameter		Culvert diameter		Culvert diameter		Culvert diameter		Culvert diameter		Culvert diameter		Culvert diameter	
	Computed (in.)	Design (in.)	Computed (in.)	Design (in.)	Computed (in.)	Design (in.)	Computed (in.)	Design (in.)	Computed (in.)	Design (in.)	Computed (in.)	Design (in.)	Computed (in.)	Design (in.)
100	7.5	8	7.30	8	7.8	8	7.20	8	7.60	8	7.58	8	7.8	8
50	7.43	8	7.22	8	7.6	8	7.08	8	7.49	8	7.52	8	7.6	8
25	7.29	8	7.11	8	7.45	8	6.95	8	7.36	8	7.43	8	7.36	8
10	7.06	8	6.93	8	7.18	8	6.74	8	7.12	8	7.26	8	7.03	8
2	6.29	8	6.25	8	6.34	8	6.18	8	6.27	8	6.57	8	6.13	8

More variation in computed and design diameters was observed for both inflow and outflow flows. Design diameters for wetland inflow rates varied for the 25- and 10-yr return periods. The respective ranges in computed D_C values across simulation periods for the 25- and 10-yr return period flows were 0.73 and 0.68 in., which were larger than the ranges seen in pre-developed-designed pipe diameters. These ranges, however, were similar to the ranges observed in the D_C values computed for the return period flows that resulted in constant design pipe diameters. LP3 computed flows for 100-, 50-, and 2-yr return periods respectively required design pipe diameters of 12, 12, and 10 in. The ranges in computed D_C values across simulation periods for each of these return periods were 0.80, 0.80, and 0.62 in. Based on these ranges, larger variation in computed D_C values did not always translate to variation in corresponding design diameters. The largest variations in computed D_C values were observed in the 100- and 50-yr return periods. However, because these variations occurred within the interval of 10 and 12 in., all resulting pipes were assigned a design diameter of 12 in. Even this variation of 0.80 in. (0.0667 ft) is relatively small compared to the 2-in range of each design culvert category. In other words, a design pipe diameter of 12 in. is used for all computed D_C values between 10 and 12 in. Therefore, if computed D_C values were within 2 in. of each other, they were assumed to represent upstream areas that were hydrologically the same. Based on this assumption, a 25-yr simulation period characterized wetland inflow hydrology within a reasonable range of variation.

Design diameters for wetland outflow rates varied for all but the 2-yr return period, which reflected the large variation in outflow rates observed in Table 6-16 and

Figure 6-11. Larger ranges in computed D_C values were also observed for outflow flows than in pre-development and inflow values, which were expected as the wetland increases the complexity of flow. As seen in the inflow-designed pipe diameters, larger variation in computed diameters did not necessarily translate differences in resulting design diameters. However, variation did consistently increase with increasing return period. The return periods of 25- and 2-yrs resulted in constant design diameters across simulation periods and had respective computed D_C ranges of 0.34 and 1.22 in. The 100-yr return period had the largest variation in D_C values with a range of 1.85 in. While these ranges were higher than those seen in the pre-developed and inflow D_C values, they were still relatively small and within a 2-in. range by which culvert pipes are designed according to ASTM standards.

These results indicated that a 25-yr simulation period was sufficient for the current study given that the 25-yr simulation periods above produced wetland inflow, wetland outflow, and pre-developed with comparable hydrology to those of a 100-yr simulation period. Additionally, stormwater wetlands are often designed for the control of smaller (≤ 1-yr, 24-hr) storm events. Variations in 2-yr flows were much lower than those observed for the larger return periods. All simulation periods produced equal-sized design culvert diameters for 2-yr flows, suggesting that a 25-yr simulation period reliably characterizes flows with a return period of 2 years or less. Based on these results, a simulation period of 25 years was chosen to perform all sensitivity analyses.

6.13 MDE DESIGN CRITERIA CHARACTERIZATION

The first portion of the sensitivity analysis assessed the importance of MDE-defined design guidelines summarized in Table 6-12. Example wetland designs were developed to test the sensitivity of each MDE-specified criterion. At least two designs, one representing an increase and the other a decrease in the parameter associated with a given criterion, were developed to evaluate each criterion. The sensitivity of model performance to the wetland surface area criterion (the wetland surface are must be ≥ 1.5 % of the drainage area), for example, was tested by increasing the drainage area so that one design had a surface area 2.5% of the drainage area and a second with a surface of 1.25% of the contributing drainage area.

The current study observed a number of trends in wetland performance with respect to design criteria changes. Throughout the sensitivity analyses of the design criteria, the relative retention time allocated to each cell type (i.e., high-marsh, low-marsh, deepwater, etc.) was the dominant factor that affected wetland water quality performance. Cell retention time was, in turn, controlled by cell depth, velocity, and vegetation type. The location of a deepwater cell was found to significantly affect TSS removal. Wetland hydrologic performance, however, was generally less sensitive than water quality performance and was most sensitive to the contributing drainage area size. Each design and associated criterion is discussed in detail in the following sections.

6.13.1 Wetland surface area

MDE (2009) requires that shallow stormwater wetlands be sized to have a surface area of greater than or equal to 1.5% of their contributing drainage area. The base design developed in the current study had a surface area of exactly 1.5% of the contributing drainage area. To test model sensitivity to wetland surface area, the contributing drainage area was (1) decreased to 3.18 ac and (2) increased to 6.36 ac in order to simulate sites where the base wetland design, respectively, represented 1.25 and 2.5% of the drainage area. Within the current section, the design with a drainage area of 3.18 ac was referred to as design DA1 and the design with a drainage area of 6.36 ac was called design DA2. While the cell configuration defined for the base design (see Figure 6-4 and Table 6-4) remained constant for design DA1 and DA2, their outlet structures were adjusted so as to properly control and transfer inflow from their respective drainage areas.

The MDE (2009) procedure used to design the base stormwater wetland design, which is outlined in Sections 6.2 and 6.5, was used to design outlet structures for the DA1 and DA2 designs according to MDE (2009) standards. Resulting outlet specifications for all three wetland designs are summarized in Table 6-21. The final outlet structures used for the DA1 and DA2 designs are also illustrated in Figures 6-13 and 6-14. All calculations leading to the DA1 and DA2 design outlet structure dimensions are shown in the Appendix 11.2.

It was noted that while both the 1-yr, 24-hr outlet orifice diameter d_o and the 100-yr, 24-hr weir length L_{100} increased with increasing drainage area, the 10-yr, 24-hr weir length L_{10} decreased with increasing drainage area. This difference in trends was due to the MDE (2009) design criteria specified for each outlet structure. The 1-yr, 24-hr outlet orifice diameter d_o was designed by MDE (2009) to drain in 24 hr. Therefore as the drainage area increased and input greater water volumes into the wetland, the corresponding d_o had to be increased accordingly in order to drain the increasing influent volume in the same 24-hr period. The 100-yr, 24-hr weir L_{100} was designed by MDE (2009) to safely transfer the 100-yr, 24-hr storm event through the wetland and was not used to control such large flows. The influent 100-yr, 24-hr storm increased with increased drainage area and, therefore, required a larger L_{100} to be safely transfer through the wetland. Finally, the 10-yr, 24-hr weir L_{10} was designed to reduce influent 10-yr, 24-hr discharge rate to TR-55-estimated pre-development discharge rate. Therefore, the L_{10} had to be decreased as the influent flowrates increased with increasing drainage area.

Table 6-21 Computed outlet structure dimensions and contributing parameters for the base, DA1, and DA2 designs. DA t_c is the drainage area time of concentration (hr), d_o is the outlet diameter (ft), Cp_v is the influent volume (ac-ft) associated with the 1-yr, 24-hr storm, Q_p is the influent volume (ac-ft) associated with the 10-yr, 24-hr storm, L_{10} is the length of the weir controlling Q_p, and L_{100} is the weir length used to transfer the 100-yr 24-hr storm through the wetland. Orifice diameter d_o values in parenthesis represent actual computed values while 3 is the minimum MDE-required orifice diameter.

	Drainage area (ac)	DA t_c (hr)	d_o (in.)	Cp_v (ac-ft)	Q_p (ac-ft)	L_{10} (ft)	L_{100} (ft)
DA1	3.18	0.16	3 (1.45)	0.125	0.221	0.831	9.21
Base	5.30	0.26	3 (1.65)	0.204	0.370	0.650	17.5
DA2	6.36	0.29	3 (1.72)	0.244	0.443	0.595	20.5

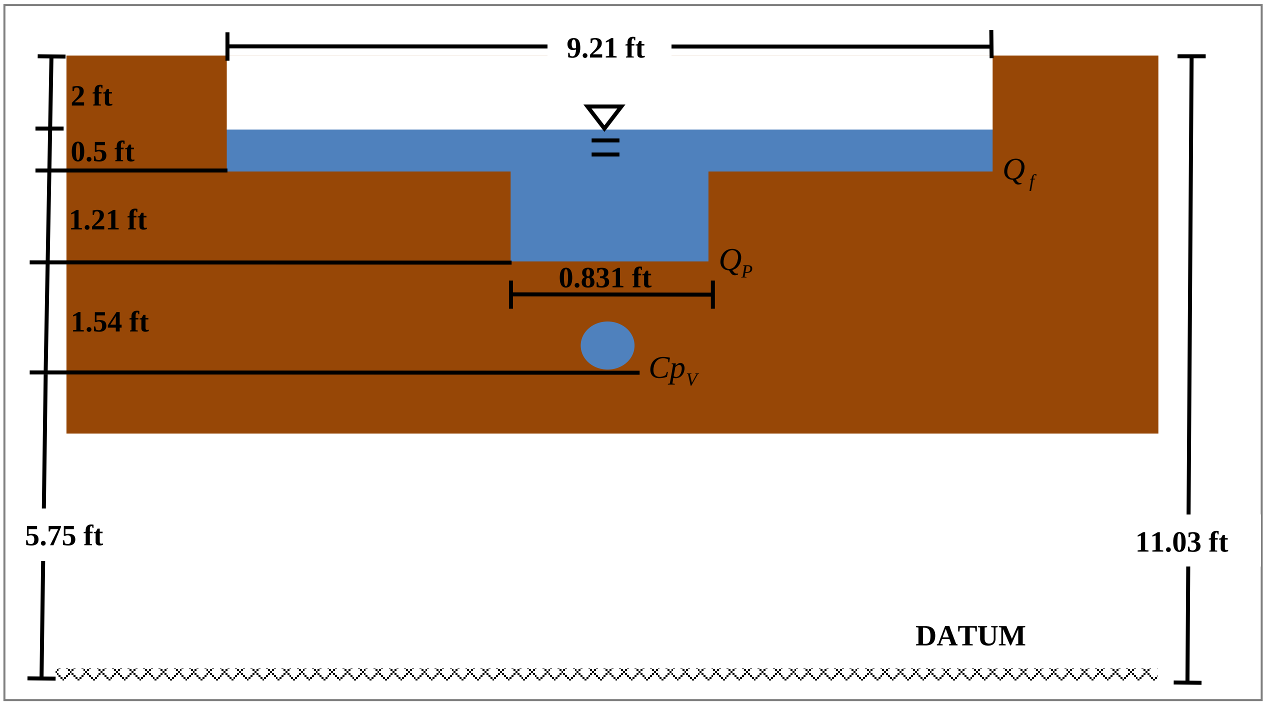

Figure 6-13 DA1 design outlet orifice and double riser design through the MDE (2009) method. The datum represents the bottom of the micropool, which was designed to have a depth of 5.75 ft. The designated Cp_V, Q_P, Q_f depths represent the water depths corresponding to the 1-yr, 10-yr, and 100-yr, 24-hr floods within the wetland.

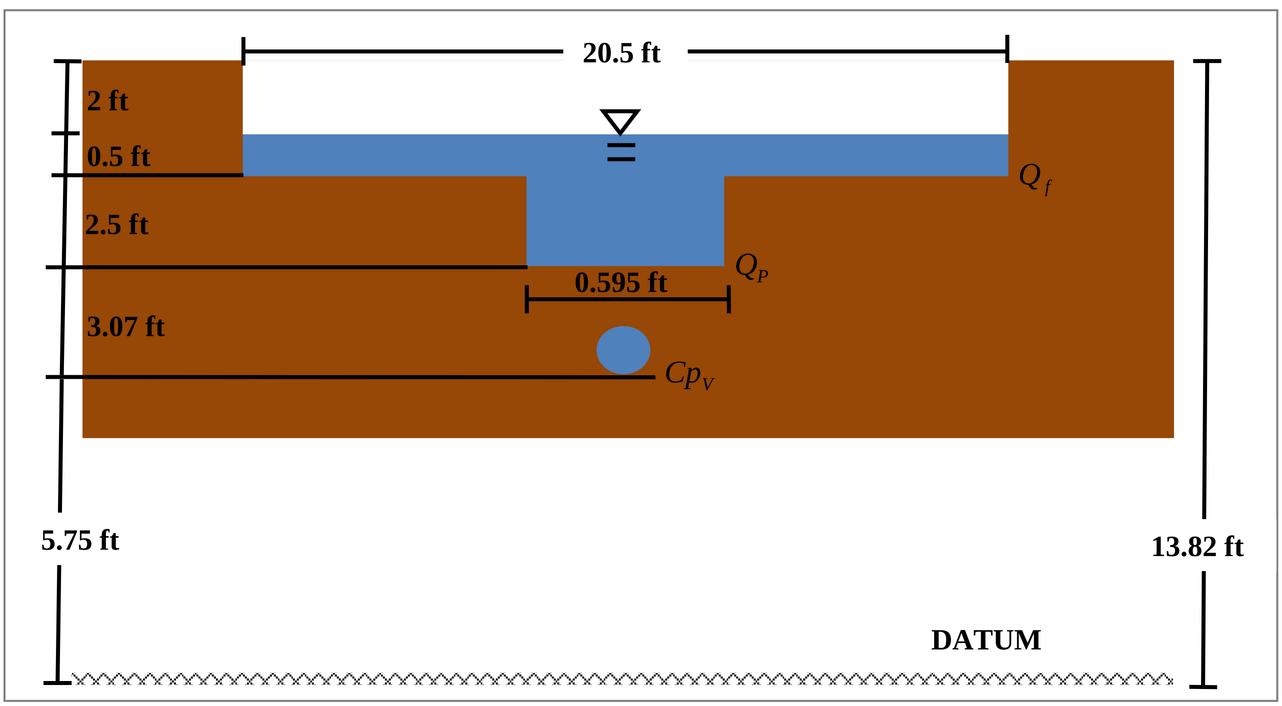

Figure 6-14 DA2 design outlet orifice and double riser design through the MDE (2009) method. The datum represents the bottom of the micropool, which was designed to have a depth of 5.75 ft. The designated Cp_V, Q_P, Q_f depths represent the water depths corresponding to the 1-yr, 10-yr, and 100-yr, 24-hr floods within the wetland.

6.13.1.1 Sensitivity analysis results

Wetland performance was found to be sensitive to wetland surface area relative to drainage area within the context of both effluent hydrology and water quality. Based on the resulting relative (S_X) and deviation (D_X) sensitivities, the effluent TSS and high-flow PC values were most sensitive to the changes in the size of the contributing drainage area (see Table 6-22). Effluent TSS concentrations increased with increasing drainage area and decreased with decreasing drainage area. This TSS trend indicated that the larger relative drainage area simulated in design DA2 produced larger inflow volumes and discharge rates, which, in turn, produced faster internal wetland velocities and reduced the overall wetland retention time, allowing for less TSS settling as evidenced by the resulting mean daily TSS concentration of 17.2 mg/L versus that of the base design of 15.4 mg/L. Conversely, the smaller relative drainage area simulated in design DA1, produced lower effluent TSS concentrations with a mean daily concentration of 10.4 mg/L as a result of the smaller inflow discharge rates and volumes entering the wetland. Similar, but weaker trends were also seen in the other water quality constituents DO, NH_4^+, and NO_3^-, which had respective mean S_X values of -0.046, 0.372, and 0.439. However, TSS was the most sensitive water quality constituent with a mean S_X of 0.697. The large relative TSS S_X with respect to the other water quality constituent S_X values may have been due to the fact that TSS settling was solely and directly related to cell retention time, while DO, NH_4^+, and NO_3^- transformations depended on a number of factors in addition to cell retention time including water depth, internal cell constituent concentrations, and cell vegetation type.

The changes made to the contributing drainage area in the DA1 and DA2 designs were found to have a complex effect on the magnitude and frequency of effluent wetland discharge rates. The high-flow PC was very sensitive to changes in drainage area size, with a mean S_X of 1.29. This PC value was a ratio of the proportion of 1-min pre-development to 1-min outflow flowrates that exceeded the computed pre-development 2-yr (bankfull flow) flowrate for a given drainage area. Therefore, a high-flow PC less than 1.0 indicates that outflow rates exceed bankfull flow more often than pre-developed flowrates. A high-flow PC greater than 1.0 indicates that pre-development flowrates exceed bankfull flow more often than outflow rates. This PC was used to evaluate the extent to which receiving natural streams would overflow if they were downstream from the wetland versus downstream from the pre-developed drainage area. As evidenced by the high-flow PC S_X of 1.29, increasing the drainage area significantly decreased the frequency with which bankfull flow events occurred at the outlet of the wetland.

Table 6-22 Relevant PC values, relative sensitivities, and deviation sensitive for the base, DA1, and DA2 stormwater wetland designs which served respective drainage areas of 5.3, 3.81, and 6.63 ac. The relationship direction indicates where the PC value increases (+) or decreases (-) with a corresponding increase in drainage area surface area.

Performance Criteria	Base design	DA1	DA2	S_x	$\lvert D_x \rvert$	Relationship direction
Mean daily TSS Conc. (mg/L)	15.4	10.4	17.2	0.697	3.38	+
Mean daily DO Conc. (mg/L)	10.3	10.5	10.2	-0.046	0.15	-
Mean daily NH4 Conc. (mg/L)	0.08	0.06	0.08	0.372	0.01	+
Mean daily NO3 Conc. (mg/L)	0.258	0.203	0.276	0.439	0.04	+
High-Marsh PC	0.962	0.990	0.947	-0.074	0.02	-
Low-Marsh PC	0.983	0.995	0.976	-0.033	0.01	-
High-Flow PC	0.316	0.170	0.406	1.29	0.12	+
Low-Flow PC	0.223	0.229	0.221	-0.059	0.00	-
Flow-Variation PC	2.90	3.03	2.85	-0.095	0.09	-
Flood-Control PC	0.382	0.382	0.382	-0.004	0.00	-

Because wetland outflow in all three designs was controlled by an outlet structure consisting of an orifice and a double stage weir, a complex relationship existed between effluent flowrates and their corresponding high-flow PC values, and drainage area size. It was observed that high-flow PC value increased with increasing drainage area, with respective DA1, base, and DA2 values of 0.170, 0.316, and 0.406. This trend suggested that as the drainage area increased the wetland produced fewer outflow discharge rates exceeding the estimated pre-development bankfull flow over the simulation period. Conversely, as the contributing drainage area decreased, the wetland produced more outflow discharge rates exceeding the pre-development bankfull flowrate. The simulated outflow 2-yr flowrates for the DA1, base, and DA2 designs were 0.236, 0.338, and 0.390 cfs. Similarly, the simulated pre-development 2-yr flows (i.e., bankfull flows) for the DA1, base, and DA2 designs were 0.222, 0.366, and 0.438. It was noted that the DA1 design 2-yr outflow rate was greater than the corresponding pre-development bankfull flowrate while the 2-yr outflow rates for the base and DA2 designs were less than their corresponding pre-development bankfull discharge rates. In order to determine if these 2-yr outflow rates were due to orifice or weir flow, the orifice flow resulting from the storage depths of 1.54 and 3.07 ft associated with $Cp_V(DA1)$ and $Cp_V(DA2)$ were computed. The maximum outflow orifice flow q_o for the base design was already computed to be 0.379 cfs in Equation 6-26. The resulting $q_o(DA1)$ and $q_o(DA2)$ flows were also computed using the orifice equation:

$$q_o(DA1) = A_o C_o \sqrt{2gh_o} = 0.0491\,\text{ft} \cdot 0.6\sqrt{2(32.2\text{ ft/s}^2)(1.54\text{ ft})} = 0.293\,\text{cfs} \quad (6\text{-}63)$$

$$q_o(DA2) = A_o C_o \sqrt{2gh_o} = 0.0491\,\text{ft} \cdot 0.6\sqrt{2(32.2\text{ ft/s}^2)(3.07\text{ ft})} = 0.414\,\text{cfs} \quad (6\text{-}64)$$

Given these maximum orifice flowrates, it was concluded that the 2-yr outflow rates for all three designs were due to orifice flow as they were all less than the corresponding maximum design orifice flowrates. All three designs employed an orifice diameter of 3 in., which was the minimum allowable diameter in MDE stormwater wetland designs. The actual computed diameters for the DA1, base, and DA2 designs were respectively 1.34, 1.65, and 1.72 in. Therefore, as the computed orifice diameter approached 3 in., effluent discharge rates became smaller relative to corresponding pre-development discharge rates. This trend suggests that stormwater wetlands may perform better hydrologically when designed for drainage areas producing sufficient runoff to require an orifice diameter of 3 in. or greater if clogging in smaller orifices is of concern. Additionally, the results from the DA1 and DA2 designs suggest that wetland hydrologic performance is sensitive to the outlet diameter size. Therefore, much care should be taken when sizing wetland outlet structures in order to ensure an optimal wetland effluent hydrologic regime.

Based on these results, it was concluded that NH_4^+, and NO_3^- effluent concentrations were moderately sensitive to changes in drainage area to wetland area ratio with respective S_X values of 0.372 and 0.439. TSS effluent concentrations were the most sensitive water quality PC value, with a S_X of 0.697, which was credited to the direct dependence of TSS settling on cell residence time (see Equations 4-74 and 4-75). It was also noted that the high-flow PC value was most sensitive to changes in the contributing drainage are size with a S_X of 1.29. High-flow PC values improved with decreasing drainage area due to the diminishing effect of the oversized outlet orifice diameter. These high-flow PC results suggest that stormwater wetlands may be more effective hydrologically when designed for a larger/more impervious drainage area or, in the case of the DA1 and DA2 designs, when designed with smaller outlet orifice diameters.

Final metrics were also computed for each wetland design (see Table 6-23). These metrics were computed based on the PTM (Performance-to-Metric) relationships defined in Section 3.4. For this example, all ten PC values weighted in the BMP-weighting scheme defined in Section 6.9.1.8 were weighted equally and summed to compute final Wetland Sustainability Indices (WSI's) for the DA1, base, and DA2 designs, which had respective values of 0.639, 0.640, and 0.646. These WSI scores indicate while increasing the drainage area slightly improved the overall wetland WSI score, the effects were minimal in the case of the DA1 and DA2 designs. In this example, a larger wetland area with respect to its contributing drainage area performed better with respect to TSS and NH_4^+ reduction based on the respective TSS and NH_4^+ metrics reported for the DA1, base, and DA2 designs in Table 6-23. The corresponding reduction in the mean daily effluent NO_3^- concentrations associated with a larger respective wetland area was not significant as reflected by its constant metric value of 1 for all three designs. This wetland NO_3^- performance implies that downstream health and sustainability is not sensitive to changes in NO_3^- concentrations below the threshold of 0.36 mg/L. Additionally, the high-flow PC and metric values worsened with decreasing drainage area due to the increasing effect of the oversized orifice. Based on these results, the current study suggests that stormwater wetlands be designed for drainage areas producing sufficient runoff to require an outlet orifice with a diameter of greater than or equal to 3 in. or with smaller orifices designed with anti-clogging controls. The following section further evaluates the effect of drainage area size on wetland performance when the initial computed orifice diameters of 1.45, 1.65, and 1.72 were employed in the DA1, base, and DA2 designs. These analyses were performed in order to better understand the true effect of the relative drainage area size on wetland performance rather than the effect of the relative over-sizing of the outlet orifice.

Table 6-23 Resulting normalized metrics and final wetland sustainability indices (WSI's) for the base, DA1, and DA2 stormwater wetland designs. All WSI scores were computed assuming equal weights for all water quality and hydrologic metrics.

Performance Criteria	Metric weights	Base design metrics	DA1 metrics	DA2 metrics
Mean daily TSS Conc. (mg/L)	0.1	0.328	0.449	0.300
Mean daily DO Conc. (mg/L)	0.1	1	1	1
Mean daily NH4 Conc. (mg/L)	0.1	0.528	0.608	0.505
Mean daily NO3 Conc. (mg/L)	0.1	1	1	1
High-Marsh PC	0.1	0.712	0.712	0.712
Low-Marsh PC	0.1	0	0	0
High-Flow PC	0.1	0.999	1.000	0.997
Low-Flow PC	0.1	1.000	1.000	0.999
Flow-Variation PC	0.1	0.000	0.000	0.000
Flood-Control PC	0.1	0.248	0.248	0.248
Final *WSI* score	---	0.640	0.639	0.646

6.14 Outlet orifice diameter

Given that the MDE minimum allowable outlet orifice diameter of 3 in. complicated the effect of the respective contributing drainage area, a second set of test designs were made to better evaluate model sensitivity to drainage area size. Within this section, the DA1, base, and DA2 wetland designs were modified to incorporate the initial computed outlet orifice diameters (d_o) of 1.45, 1.65, and 1.72 in. rather than the MDE-required 3-in. diameter orifice. The resulting designs were respectively defined as OD1, ODB, and OD2. OD1 served the same drainage area of 3.18 ac as the DA1 design and had an outlet orifice diameter of 1.45 in. ODB represented the base wetland design, serving a drainage area of 5.3 acres with an outlet orifice diameter of 1.65 in. Finally, the OD2 design served a drainage area of 6.36 ac with an outlet orifice diameter of 1.72 in. The smaller outlet orifice diameters altered the 10- and 100-yr, 24-hr weir lengths, L_{10} and L_{100}, that were used in the OD1, ODB, and OD2 designs with respect to the base, DA1, and DA2 designs. Equations 6-28 and 6-30 were used to compute L_{10} and L_{10} for each design. All design orifice and weir dimensions for the OD1, ODB, and OD2 designs are summarized in Table 6-24.

Table 6-24 Computed d_o, L_{10}, and L_{100} values for the OD1, ODB, and OD2 stormwater wetland designs.

Design	d_o (in.)	L_{10} (ft)	L_{100} (ft)
OD1	1.45	0.919	8.79
ODB	1.65	0.678	17.2
OD2	1.72	0.617	20.1

6.14.1.1 ODB design results

The effect of the outlet orifice diameter was evaluated first by comparing wetland performance in the base stormwater design and the ODB design. As shown in Table 6-25, decreasing the orifice diameter from 3 to 1.65 in. in the ODB design had a significant impact on both hydrologic and water quality wetland performance. The smaller orifice in the ODB design reduced the effluent orifice discharge rate, which promoted a longer retention time as well as higher water levels within the wetland compared with those observed in the base design. Slower effluent discharge rates also increased the duration of effluent flow, which resulted in more non-zero effluent flows. The increased water height in the wetland also increased the frequency of flow events that over-topped the 10-yr, 24-hr weir as evidenced by the decrease in the high-flow PC, which was the most sensitive PC value with a S_X value of 0.691. Increased wetland retention times in the ODB design also reduced mean daily effluent TSS, NH_4^+, and NO_3^- concentrations, which had respective S_X values of 0.064, 0.0248, and 0.257.

All hydrologic PC values except for the flood control PC were relatively sensitive to the decreased outlet orifice diameter in the ODB design. This model sensitivity to orifice diameter was reflected in the resulting effluent discharge rates. As computed in Equation 6-26, the maximum orifice discharge rate for the base design was 0.379 cfs. The ODB design had a smaller orifice diameter and, therefore, a smaller maximum discharge rate of 0.113 cfs:

$$q_o(ODB) = A_o C_o \sqrt{2gh_o} = 0.0147\,\text{ft}^2 \cdot 0.6\sqrt{2(32.2\,\text{ft/s}^2)(2.56\,\text{ft})} = 0.113\,\text{cfs} \quad (6\text{-}65)$$

As a result of the smaller discharge rates associated with the smaller orifice in the ODB design, water flowed out of the wetland at a slower rate and, therefore, promoted a longer wetland retention time, as well as a slightly decreased internal wetland velocity of 0.000607 ft/s in the ODB design vs 0.000658 ft/s in the base design. Decreased effluent rates also caused wetland water depths to increase slightly. Mean high and low-marsh depths increased from 0.520 and 1.27 ft in the base design to 0.578 and 1.33 ft in the ODB design, resulting in worse respective high and low-marsh PC values of 0.865 and 0.940 vs the base design values of 0.962 and 0.983. Both high and low-marsh PC values were ratios of the design to actual mean depth for a wetland cell type. Therefore, as actual mean depths approached design depths, the corresponding marsh PC value approached 1 and as actual mean depths exceeded design depths more and more, the corresponding marsh PC value approached 0. The reductions observed in high and low-marsh PC values were also characterized by respective S_X values of 0.224 and 0.097, which indicated that the decreased orifice diameter in the ODB design slightly increased wetland water depths. The larger high-marsh PC S_X value of 0.224 also suggested that shallower water depths were more sensitive to the change in orifice diameter. Within the context of the current study these small changes in water depth were not significant under the assumption that they would not affect the ability of the wetland to sustainably promote vegetation growth, pollutant removal, etc. However, the significance of such changes may change based on the conditions required in a given wetland design to achieve stakeholder goals.

Higher water depths in the wetland also promoted more frequent flow over the 10-yr weir, which resulted in a larger effluent wetland 2-yr discharge rate of 0.436 cfs in the ODB design versus that of 0.366 in the base design. The maximum orifice flow in ODB design was 0.113 cfs and that of the base design was 0.379 cfs. Therefore, while the 2-yr flow in the base design occurred as orifice flow, the 2-yr flow in the ODB design occurred as weir flow over the 10-yr weir, which produced a maximum effluent discharge rate of 6 cfs (see Table 6-1). As a result of the increased frequency of larger weir flows produced in the ODB design, the high-flow PC decreased from 0.316 in the base design to 0.218, resulting in a S_X of 0.691. This PC value was a ratio of the proportion of 1-min pre-development to 1-min outflow flowrates that exceeded the computed pre-development 2-yr (bankfull flow) flowrate for a given drainage area. Therefore, a high-flow PC less than 1.0 indicates that outflow rates exceed bankfull flow more often than pre-developed flowrates. A high-flow PC greater than 1.0 indicates that pre-development flowrates exceed bankfull flow more often than outflow rates. Therefore, while both the base and ODB designs produced a higher proportion of effluent flows that exceeded the estimated downstream bankfull flow, the ODB design proportion was greater. These results suggest that the decreased orifice diameter in the ODB design would promote more bankfull flows downstream than the base design, which, in turn, could cause greater erosion of downstream banks (Dunne and Leopold 1978).

For the ODB design the effluent flow duration increased due to the slower discharge rates associated with the smaller orifice diameter. This trend was evidenced by the increased number of non-zero flow days of 267 days in the ODB design vs. 262 days in the base design. The low-flow PC value, which was a ratio of the proportion of pre-development to effluent 1-min flows greater than 0, decreased slightly from 0.223 in the base design to 0.216 in the ODB design. As low-flow PC values approached 0, they indicated that a larger and larger proportion of effluent flows were greater than 0 given that the pre-development proportion for a given drainage area remained constant. Therefore, the decreased low-flow PC value in ODB design further illustrated the increased duration of effluent flow caused by its smaller orifice diameter. The observed increase in the low-flow PC was small, with a S_X of 0.0728. Significance of changes in the low-flow PC values should be dependent on their relative impact on wetland functions. For example, the small decrease in the low-flow PC value observed in the ODB design could be significant to sensitive stream species that require specific seasonal dry periods for reproduction. Conversely, such a decrease in zero-flow periods may not have any effect on less sensitive species.

Wetland effluent flow variation was also observed to decrease in the ODB design due to the increased effluent flow duration associated with the smaller diameter. The longer effluent orifice flow durations associated with the higher water depths in the ODB design promoted more constant effluent discharge rates as well as more frequent zero-flow periods. Both of these trends promoted more uniform effluent discharge rates and consequently less flow variation. This decrease in variation was captured by the corresponding increase in the flow variation PC value from 2.90 to 3.16 in the base and ODB designs. The flow variation PC value was, again, a ratio that compared the pre-development to effluent flow variation. Therefore, flow variation PC values greater than 1 indicated that flow variation in the pre-developed drainage area was greater than that in the wetland outflow. Flow variation was fairly sensitive to the decreased orifice diameter in the ODB design with a S_X of -0.206. Given these values, ODB design performed worse than the base design in mimicking pre-development flow variation.

Overall wetland water quality performance was found to increase slightly with decreasing outlet orifice diameter. The increased wetland retention time associated with the slower effluent discharge rates in the ODB design allowed for a longer duration over which TSS, NH_4^+, and NO_3^+ could be reduced via settling, nitrification, and denitrification. Resulting mean daily TSS, NH_4^+, and NO_3^+ relative sensitivities S_X were all positive, implying that they decreased with decreasing orifice diameter, and had respective values of 0.064, 0.0248, and 0.257. While effluent TSS and NH_4^+ concentrations were not sensitive to the decreased orifice diameter in the ODB design, effluent NO_3^- concentrations did decrease significantly with the decreased orifice diameter. This sensitivity of effluent NO_3^- concentrations was hypothesized to be due to an increased retention time in cells that contain emergent vegetation, which were the only cells in which denitrification was simulated. The slight increase in retention time experienced in these cells did not affect TSS, NH_4^+ as significantly given that they were removed in all cells. Additionally, the calibrated denitrification rate constant was an order of magnitude larger than the nitrification rate constant, which promoted faster denitrification within the appropriate cells.

Based on the improved water quality and worsened hydrologic performance of the ODB design with respect to the base design, the current study suggests that the outlet orifice diameter be sized in order to balance wetland water quality and hydrologic performance. While the smaller orifice in the ODB design reduced orifice discharge rates, it also increased internal water depths, promoted more frequent erosive flows via weir flow, and produced longer flow durations. Conversely, the smaller orifice in the ODB design also slightly improved effluent water quality, producing lower mean daily effluent TSS, NH_4^+, and NO_3^- concentrations. Overall, however, the hydrologic PC values appeared to be more sensitive to the decrease in orifice diameter than the water quality PC values. Therefore, the negative hydrologic effects observed in the ODB design may outweigh the corresponding positive water quality effects.

The worsened hydrologic performance in the ODB design suggested that current stormwater wetland design does not promote effluent flows that mimic pre-development hydrology. Currently, MDE sizes outlet orifices to drain the influent 1-yr, 24-hr flow over a period of 24 or 12 hours, depending on the intended use of the effluent water. While this sizing method has been successful in flood control and peak flow reduction throughout the literature, it does not mimic the goal pre-development hydrologic regime successfully. However, the underlying problem associated with stormwater runoff is the excess of runoff volume produced by drainage areas that do not promote sufficient infiltration. Therefore, altering the outlet orifice dimensions can only control the rate at which water exits the wetland. With an excess of volume entering the wetland, a smaller orifice, as seen in the ODB design, promotes slower effluent discharge rates over a longer period of time. Conversely, a larger orifice promotes faster effluent discharge over a shorter period of time. Both designs could have adverse effects on downstream health as neither matches the flow duration and variation of the corresponding pre-developed hydrologic regime. Therefore, in order to improve hydrologic performance, greater effluent volume reduction in addition to peak reduction should be emphasized in wetland design.

Table 6-25 Relevant PC values, relative sensitivities, and deviation sensitivities observed between the base and ODB stormwater wetland designs. The relationship direction indicates where the PC value increases (+) or decreases (-) with a corresponding decrease in orifice diameter.

Performance Criteria	Base	ODB	S_x	$\lvert D_x \rvert$	Relationship direction
Mean daily TSS Conc. (mg/L)	15.4	14.9	0.064	-4.86	+
Mean daily DO Conc. (mg/L)	10.3	10.4	-0.018	0.20	-
Mean daily NH4 Conc. (mg/L)	0.0782	0.073	0.0248	-0.01	+
Mean daily NO3 Conc. (mg/L)	0.258	0.228	0.257	-0.05	+
High-marsh PC	0.962	0.865	0.224	0.06	+
Low-marsh PC	0.983	0.940	0.097	0.03	+
High-Flow PC	0.316	0.218	0.691	0.02	+
Low-Flow PC	0.223	0.216	0.0728	0.01	+
Flow Variation PC	2.90	3.16	-0.206	0.04	+
Flood Control PC	0.382	0.382	0.000	0.00	---

6.14.1.2 Comparing ODB with the OD1 and OD2 designs

Increasing the contributing drainage area to the wetland increased influent volumes, which, in turn, increased internal wetland velocities and water depths, and reduced overall wetland retention time and worsened water quality performance. Conversely, decreasing the contributing drainage area decreased influent volumes, internal wetland velocities, and water depths, while increasing wetland retention time and improving water quality performance. All resulting water quality PC values and associated sensitivities are shown in Table 6-26. Despite these clear trends in wetland behavior, the effects of altering the drainage area size on wetland hydrologic performance, specifically on the high-flow PC, were complicated by the wetland outlet structure. However, overall, it was found that the OD1 design, which served the smallest drainage area of 3.18 ac, performed the best with a final WSI score of 0.651 (see Table 6-27).

The high-marsh and low-marsh PC values were both inversely related to drainage area with respective mean S_X values of -0.183 and- 0.0811. Therefore, the increased drainage area and corresponding increased influent volume in the OD2 design produced deeper internal wetland depths. Conversely, decreased influent volumes associated with a smaller drainage area, as was seen in the OD1 design, resulted in shallower water depths than those observed in the ODB design. This trend in wetland water depths indicated that an appropriately-sized drainage area is necessary to ensure design wetland depths are maintained.

The low-flow PC was found to decrease with increasing drainage area with a mean S_X value of -0.076. This trend indicated that a larger proportion of effluent flows were greater than zero in the OD2 design than in the ODB design. Therefore, the larger volume of water moving through the OD2 design due to the increased drainage area produced longer effluent flow durations and shorter zero-flow periods. Similarly, the OD1 design effluent experienced zero-flow periods more frequently than the ODB design as less inflow entered the OD1 design, which produced less effluent flow and more zero-flow periods.

Both the OD1 and OD2 high-flow PC values, which had respective values of 0.238 and 0.231, were greater than that of the ODB design, which was 0.218. These high-flow PC values suggest that both the OD1 and OD2 designs produced fewer 1-min effluent flows exceeding the pre-development bankfull flow than the ODB design. Two different mechanisms were found to cause such a decrease in effluent discharge rates exceeding pre-development bankfull flow. Firstly, higher water depths in OD2 design promoted more frequent weir flow, which was faster than orifice flow, resulting in high flows with short durations. Therefore, while the large (greater than bankfull flow) effluent discharge rates produced by the OD2 design were greater than those produced by the ODB design, they occurred over shorter time periods. Secondly, the lower water depths in the OD1 designs produced smaller effluent discharge rates and, therefore, did not produce flows exceeding the estimated downstream bankfull flow as frequently as did the ODB design. Overall, the high-flow PC value was relatively sensitive to drainage area size with a mean S_X value of 0.266, suggesting that the relative drainage area size had a significant impact on the duration and frequency of erosive flows produced by the wetland.

Finally, wetland water performance improved as the contributing drainage area decreased due to the corresponding decrease in inflow volumes. The reduced influent volume associated with the OD1 design also reduced the mean internal wetland velocity to 0.000326 ft/s from 0.000606 ft/s in the ODB design. Conversely, the increased influent volume in the OD2 design increased the mean internal wetland velocity to 0.000754 ft/s. This trend in internal velocity suggested that the wetland retention time decreased with increasing drainage area. Therefore, the OD1 design had a longer retention time than the ODB design while the OD2 design had a shorter retention time than that of the ODB design. As discussed in Section 6.14.1.1, a longer retention time promoted greater TSS, NH_4^+, and NO_3^- reduction. Therefore the OD1 design improved ODB water quality performance while the OD2 design worsened water quality performance. Resulting mean daily effluent TSS, NH_4^+, and NO_3^- mean S_X values were 0.715, 0.379, and 0.431, showing that the effect of drainage area on water quality performance was significant. It was hypothesized that rhe TSS S_X value was greater than those of NH_4^+, and NO_3^- because it was solely and directly related to cell retention time while NH_4^+, and NO_3^- transformations were dependent on a number of other wetland cell factors.

Based on these results, decreasing the contributing drainage area slightly improved both water quality and hydrologic wetland performance as evidenced by the high WSI score of the OD1 design of 0.651. For comparison, the final WSI scores for the base, ODB, and OD2 designs were computed to be 0.640, 0.624, and 0.621 (see Table 6-28). The significance of changes in the final wetland WSI scores is dependent on (1) the stakeholder goals for a given wetland design and (2) the wetland conditions required to meet these goals. Unfortunately, a lack of data and knowledge currently limit the quantification such significance. Within the context of the current study, the resulting WSI scores for the ODB, OD1, and OD2 were assumed not to be significantly different given that these changes did not appear to affect the ability of the wetland design to promote sustainable water quality and hydrologic outputs.

The current study hypothesized that similar hydrologic results could also be obtained by requiring greater infiltration either within the contributing drainage area or within the wetland in order to reduce effluent wetland volumes. As discussed in Section 6.14.1.1, it may be necessary to size and design wetland orifices differently than is currently done by MDE. However, based on the relative performance of the ODB, OD1, and OD2 designs, the wetland water quality and hydrologic performance is sensitive to the contributing drainage area size. The optimal wetland surface area for a given drainage area, however, should depend on the hydrology and water quality characteristics of the drainage area runoff as well as the hydrologic and water quality goals for the wetland outflow.

Table 6-26 Relevant PC values, relative sensitivities, and deviation sensitive for the ODB, OD1 and OD2 stormwater wetland designs. The relationship direction indicates where the PC value increases (+) or decreases (-) with a corresponding increase in drainage area. S_X values in parenthesis were computed using the absolute values of those from OD1 and OD2 because they had both positive and negative effects on the corresponding PC value.

Performance Criteria	ODB	OD1	OD2	S_x	$\lvert D_x \rvert$	Relationship direction
Mean daily TSS Conc. (mg/L)	14.9	10.07	16.8	0.715	3.35	+
Mean daily DO Conc. (mg/L)	10.4	10.6	10.3	-0.041	0.134	-
Mean daily NH4 Conc. (mg/L)	0.0773	0.0630	0.0819	0.379	0.0094	+
Mean daily NO3 Conc. (mg/L)	0.228	0.180	0.244	0.431	0.0317	+
High-marsh PC	0.865	0.929	0.834	-0.183	0.0477	-
Low-marsh PC	0.940	0.970	0.924	-0.081	0.0228	-
High-Flow PC	0.218	0.238	0.231	(0.266)	0.0165	+/-
Low-Flow PC	0.216	0.224	0.213	-0.076	0.00527	-
Flow Variation PC	3.16	3.20	3.16	-0.020	0.0221	-
Flood Control PC	0.382	0.382	0.382	-0.004	0.0005	-

Table 6-27 Resulting normalized metrics and final wetland sustainability indices (WSI's) for the base, ODB, OD1, and OD2 stormwater wetland designs. All WSI scores were computed assuming equal weights for all water quality and hydrologic metrics.

Performance Criteria	Metric weights	Base design metrics	ODB metrics	OD1 metrics	OD2 metrics
Mean daily TSS Conc. (mg/L)	0.1	0.328	0.336	0.460	0.306
Mean daily DO Conc. (mg/L)	0.1	1	1	1	1
Mean daily NH4 Conc. (mg/L)	0.1	0.528	0.533	0.613	0.509
Mean daily NO3 Conc. (mg/L)	0.1	1	1	1	1
High-marsh PC	0.1	0.999	0.982	0.995	0.972
Low-marsh PC	0.1	1.000	0.996	0.999	0.994
High-Flow PC	0.1	0.533	0.390	0.426	0.411
Low-Flow PC	0.1	0.396	0.385	0.399	0.381
Flow-Variation PC	0.1	0.000	0.000	0.000	0.000
Flood-Control PC	0.1	0.618	0.618	0.619	0.618
Final *WSI* score	---	0.640	0.624	0.651	0.619

6.14.1 Forebay specifications

MDE (2009) requires that the forebay account for at least 10% of the WQ_v and that it be separated from the main wetland by a berm. The forebay in the base wetland design accounted for 10.3% of the WQ_v. In order to evaluate the importance of the forebay and its storage, a total of three designs were simulated, which included designs that incorporated (1) a forebay sized as 25% of the WQ_v, (2) a forebay sized as 1.8% of the WQ_v, and (3) no forebay by way of removing the berm separating forebay and the main wetland. These three forebay designs were referred respectively as designs FB1, FB2, and FB3. The resulting upper and lower limits were chosen based on the relative insensitivity of wetland performance to smaller changes to the forebay volumes. The volume of the forebay in designs FB1 and FB2 was adjusted by respectively increasing and decreasing the base design forebay depth of 4 ft to 9.75 and 0.7 ft. While a forebay depth of 9.75 ft may not be reasonable for many real-world wetland designs, it was chosen for the FB1 design to illustrate the insensitivity of wetland performance to such a large change in the forebay volume. Additionally, the forebay in the FB2 design was limited to 1.8% of the WQ_v in order to maintain a total wetland volume equal to the WQ_v.

6.14.1.1 Sensitivity analysis results

The base stormwater wetland was not significantly affected by any of the changes made to the forebay in the FB1, FB2, and FB3 designs. The maximum mean S_X values produced by designs FB1 and FB2 were -0.038 and -0.018 for daily effluent NH_4^+ and TSS concentrations (see Table 6-22). Both of these S_X values were negative, which indicated that both effluent NH_4^+ and TSS effluent concentrations decreased with increasing forebay volume. These trends are rational, as a larger forebay promotes greater forebay storage and, therefore, a longer forebay retention time, which, in turn, promotes greater TSS settling and nitrification of NH_4^+. Conversely, a smaller forebay provides less storage and, therefore, promotes less TSS settling and nitrification of NH_4^+. The FB3 design produced a maximum S_X value of 1.34×10^{-4} for the high-marsh PC. Given its low resulting S_X values, the removal of the berm that separated the forebay from the main wetland did not appear to affect model performance. The insensitivity of wetland performance to the removal of the forebay berm in design FB3 suggests that water levels in the wetland were generally constant, and the forebay generally full. As a result of the low sensitivity of the model to forebay volume in designs FB1 and FB2, and to the exclusion of a forebay berm in design FB3, the final metrics and WSI values reported in Table 6-23 did not differ significantly from those of the base design due to the manner in which TSS was simulated within the model.

Table 6-28 Relevant PC values, relative sensitivities, and deviation sensitive for the base, FB1and FB2 stormwater wetland designs (Design FB3 saw no S_x greater than1.5x10^{-4}). The relationship direction indicates where the PC value increases (+) or decreases (-) with a corresponding increase in forebay volume.

Performance Criteria	Base design	FB1	FB2	S_x	$\lvert D_x \rvert$	Relationship direction
Mean daily TSS Conc. (mg/L)	15.4	15.3	15.7	-0.018	0.037	-
Mean daily DO Conc. (mg/L)	10.3	10.3	10.3	-0.001	0.004	-
Mean daily NH4 Conc. (mg/L)	0.078	0.075	0.080	-0.038	0.010	-
Mean daily NO3 Conc. (mg/L)	0.258	0.259	0.258	0.003	0.001	+
High-marsh PC	0.962	0.962	0.962	-1.73x10^{-16}	3.36 x10^{-5}	-
Low-marsh PC	0.983	0.983	0.983	-2.08x10^{-16}	1.38 x10^{-5}	-
High-Flow PC	0.316	0.316	0.316	0.00	5.29 x10^{-7}	-
Low-Flow PC	0.22	0.223	0.223	0.00	5.29 x10^{-7}	-
Flow-Variation PC	2.90	2.897	2.897	1.13 x10^{-14}	2.16E x10^{-7}	+
Flood-Control PC	0.382	0.382	0.382	1.60 x10^{-17}	1.13 x10^{-7}	+

Table 6-29 Resulting normalized metrics and final wetland sustainability indices (WSI's) for the base, FB1, FB2, and FB3 stormwater wetland designs. All WSI scores were computed assuming equal weights for all water quality and hydrologic metrics.

Performance Criteria	Metric weights	Base design metrics	FB1 metrics	FB2 metrics	FB3 metrics
Mean daily TSS Conc. (mg/L)	0.1	0.328	0.330	0.322	0.328
Mean daily DO Conc. (mg/L)	0.1	1	1	1	1
Mean daily NH4 Conc. (mg/L)	0.1	0.528	0.546	0.517	0.528
Mean daily NO3 Conc. (mg/L)	0.1	1	1	1	1
High-marsh PC	0.1	0.999	0.999	0.999	0.999
Low-marsh PC	0.1	1.000	1.000	1.000	1.000
High-Flow PC	0.1	0.533	0.533	0.533	0.533
Low-Flow PC	0.1	0.396	0.396	0.396	0.396
Flow-Variation PC	0.1	0.000	0.000	0.000	0.000
Flood-Control PC	0.1	0.618	0.618	0.618	0.618
Final *WSI* score	---	0.640	0.642	0.638	0.640

The insensitivity of wetland performance to changes in the forebay volume were hypothesized to be a function of the model use of a single mean input TSS particle diameter. MDE (2009) requires a forebay for the initial removal of larger particles in stormwater before it enters the main wetland. The current model only incorporated one mean TSS particle diameter input. Therefore, all influent TSS particles were assigned a diameter of 1.2×10^{-6}m, which was determined through calibration (see Section 6.8). Due to the use of this mean TSS particle diameter, all TSS particles behaved the same as they were routed through the wetland. In reality, a distribution of TSS particle sizes would enter the wetland, the heaviest of which would be settled out in the forebay. However, because the model did not simulate these different particle sizes, the resulting wetland performance only reflected that of the input mean TSS diameter of 1.2×10^{-6}m. Additionally, the purpose of the forebay is to provide an area in which sediment build-up from the larger TSS particles can be removed periodically. Because the forebay is typically designed with a concrete bottom, removal of sediment is much easier and less disruptive in the forebay than within the main wetland area, which is vegetated and often sustains aquatic wildlife. The overall wetland trap efficiency also does not distinguish between TSS settled within the forebay versus TSS settled in the main wetland area. Therefore, within the current model, the importance of the forebay with respect to TSS settling could not be reliably evaluated.

While the model did not fully capture all functions of the forebay, the sensitivity analyses did show that neither the forebay volume nor the forebay berm had significant bearing on wetland hydrologic performance. These analyses also revealed that changes to the forebay and berm structure did not significantly affect wetland water quality performance with respect to NH_4^+ and NO_3^- reduction. Despite these conclusions, the results from the forebay analyses cannot be used neither to negate nor confirm the current MDE (2009) requirements that the forebay be 10% of the WQ_v and that it be separated from the main wetland by a berm.

Based on these results, a number of model improvements were proposed in order to better evaluate the importance and structure of the forebay. Influent TSS particle diameters should enter the wetland as a distribution rather than a single mean value to simulate the initial settling of larger particles in the forebay. If these TSS distributions were defined and used as model inputs, the model could also compute the estimated sediment build-up within the forebay as well as within the main wetland. This computation would also the user to evaluate how quickly sediments build-up in the main wetland given different forebay designs.

6.14.2 Storage in deepwater areas

Deepwater areas in the original stormwater wetland accounted for 25.0% of the total WQ_v, which was the MDE requirement for deepwater areas. Two wetland designs were employed to assess the sensitivity of this deepwater design criterion. The first design reduced the volume of deepwater areas to 20.5% of the WQ_v while the second design increased the volume of deepwater areas to 30.0% of the WQ_v. These deepwater wetland designs were referred to respectively as designs DW1 and DW2. Deepwater zones accounting for 20.5% of the WQ_v were achieved in design DW1 by reducing the micropool (cell 1 in Figure 6-4) depth from its base value of 5.75 to 4 ft, which was the minimum depth required for deepwater areas. Similarly, deepwater zones that account for 30.0% of the WQ_v were achieved in design DW2 by increasing the micropool depth to 7.72 ft.

While changes to the micropool depth in designs DW1 and DW2 did not affect model hydrologic outputs, they did impact wetland water quality performance. All resulting PC, S_X , and D_X values for designs DW1 and DW2 are shown in Table 6-30. The lack of hydrologic change in the DW1 and DW2 designs was rational given that flow out of the micropool was controlled by the outlet orifice and weir structure rather than micropool depth. The water quality performance of the relative water quality performance of designs DW1 and DW2 did, however, demonstrate that the micropool depth affected a number of wetland water quality functions. Of the water quality PC values, the mean daily effluent TSS and NO_3^- concentrations were most sensitive to changes in micropool depth with respective S_X values of 0.229 and 0.142. These positive S_X values also indicated that both TSS and NO_3^- mean daily effluent concentrations increased with increasing micropool depth and subsequent increasing deepwater volume. The increasing trend seen in TSS concentrations was opposite of that observed in Section 6.14.1.1, in which mean daily effluent TSS concentrations decreased slightly (S_X of -0.018) with increasing forebay depth. Water quality trends, however, for DO, NO_3^-, and NH_4^+ in designs DW1 and DW2 were consistent with those observed in the FB1 and FB2 designs in Section 6.14.1.1.

Table 6-30 Relevant PC values, relative sensitivities, and deviation sensitive for the base, DW1, and DW2 stormwater wetland designs. The relationship direction indicates where the PC value increases (+) or decreases (-) with a corresponding increase in deepwater storage in the wetland.

Performance Criteria	Base design	DW1	DW2	S_x	$\|D_x\|$	Relationship direction
Mean daily TSS Conc. (mg/L)	15.4	14.6	16.0	0.229	0.678	+
Mean daily DO Conc. (mg/L)	10.3	10.4	10.2	-0.0624	0.124	-
Mean daily NH4 Conc. (mg/L)	0.0782	0.0793	0.0770	-0.0743	0.00	-
Mean daily NO3 Conc. (mg/L)	0.258	0.250	0.264	0.142	0.00709	+
High-marsh PC	0.962	0.962	0.962	0.00	0.00	+
Low-marsh PC	0.983	0.983	0.983	0.00	0.00	+
High-Flow PC	0.316	0.316	0.316	0.00	0.00	-
Low-Flow PC	0.223	0.223	0.223	0.00	0.00	-
Flow Variation PC	2.90	2.90	2.90	0.00	0.00	-
Flood Control PC	0.382	0.382	0.382	0.00	0.00	+

6.14.2.1 TSS behavior

Daily effluent TSS concentrations in designs DW1 and DW2 showed trends opposite of effluent TSS loads in the FB1 and FB2 designs discussed in Section 6.14.1.1. The DW1 design, which had a shallower micropool than the base design, produced a lower mean daily effluent TSS concentration of 14.6 mg/L versus that of the base design of 15.4 mg/L. Conversely, the DW2 design, which had a deep micropool, produced a larger mean daily effluent TSS concentration of 16.0 mg/L. While these results suggested that the increased deepwater storage provided in the DW2 design did not allow for longer TSS retention as was observed in designs FB1 and FB2, the total settled and effluent TSS loads over the simulation indicated the opposite. The DW1, base, and DW2 designs settled out a total of 4.47, 4.49, and 4.50 Mg over the 25-simulation period. Additionally, the DW1 design produced an effluent TSS load of 4.90 Mg, the base design a TSS effluent load of 4.88 Mg, and the DW2 design a TSS effluent load of 4.87 Mg. These settled and effluent TSS loads revealed that the total settled TSS load increased with micropool depth. As a result, over the 25-year simulation period, the DW2 design produced less effluent TSS and the DW1 design produced more effluent TSS load than the base design.

This trend in TSS is result of the model relationship between effluent flow and TSS concentrations. TSS settling, as computed by Stoke's Law, was constant for a given TSS particle diameter as all other inputs to were assumed constant (see Section 4.4.2.1):

$$v_s = 3.28 \cdot \frac{(\rho_p - \rho_w)}{18\mu} g_m \cdot D^2 \tag{6-66}$$

where v_s is the settling velocity (ft/s), ρ_p is the particle density (kg/m^3), ρ_w is the water density (kg/m^3), μ is the fluid dynamic viscosity (N-s/m^2), g_m is gravity (m/s^2), and D is the particle diameter (m). Because the current study assumed that ρ_p, ρ_w, μ, and g_m were constant, the settling velocities within the three designs were all equal as they incorporated the same D value of 1.2x10^{-6} m. Corresponding trap efficiencies were then calculated for each cell for a given time interval assuming completely mixed conditions:

$$TE = 60 \cdot \frac{v_s \cdot \Delta t}{SS} \tag{6-67}$$

where Δt represents the time interval (min), *SS* is the cell surface water depth (ft), and *TE* is the resulting cell trap efficiency of TSS. Based on these TSS computations, two factors controlled TSS settling in a wetland cell, (1) the retention time and (2) the cell water depth. As observed in the analysis of designs FB1 and FB2 in Section 6.14.1.1, increasing the cell depth also increased the retention time within a cell. The cell TE, however, represented a smaller and smaller proportion of the total TSS load in a cell as the cell water depth was increased. Therefore, when little or no flow occurred in a cell the retention time factor dominated TSS settling while during high flow periods, the magnitude of the *TE* proportion dominated settling. These factors implied that shallower cells would promote greater TSS settling during high flow periods while deeper cells with greater storage would promote greater TSS settling during periods of low- and zero-flow. This trend was observed in the DW1 and DW2 designs. As shown in the pollutographs in Figure 6-15, the greater storage in design DW2 produced lower effluent TSS concentrations during the rising limb of the pollutographs, which reflected the greater settling capacity of the deeper DW2 micropool during zero-and low-flow periods preceding storm events. However, after the peak flow and into the falling limb of the pollutographs, effluent concentrations from the DW1 design decreased faster than those of the DW2 design due to the high proportion of TSS being removed as *TE* in the shallower micropool in the DW1 design.

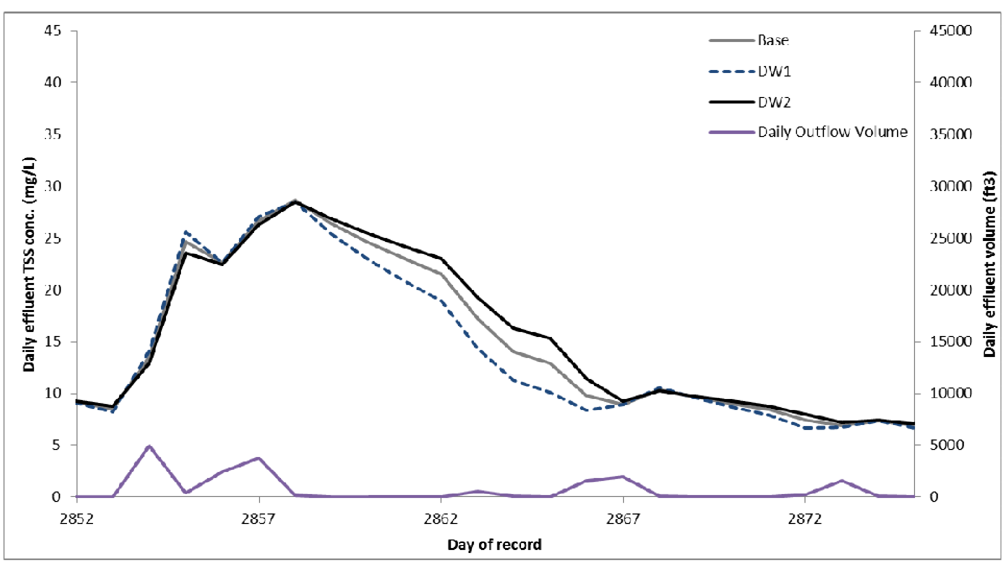

Figure 6-15 Base (grey line), DW1 (dotted blue line), DW2 (black line) daily effluent TSS concentrations. Daily effluent volume also shown in purple solid line. Low effluent volumes are all above zero.

Based on the trends in TSS behavior in the DW1 and DW2 designs, the current study observed that while the mean daily effluent TSS concentration increased slightly with increasing micropool depth, the overall effluent TSS load decreased with increasing micropool depth. Therefore, while the trap efficiency *TE* proportion was the dominating factor controlling TSS settling over a shorter time scale of a day, the cell retention time and storage controlled TSS settling over the longer time scale of 25 years.

While both the retention time and the trap efficiency proportionality factors were evident in designs DW1 and DW2, the retention time factor appeared to dominate in the FB1 and FB2 designs in which the depth of the forebay was varied (see Section 6.14.1.1). The likely causes of this difference in wetland performance were (1) the relative locations of the forebay and the micropool and (2) the velocities of flows leaving the forebay and micropool cells. Because the forebay was located at the inlet of the wetland, the short-term effects of trap efficiency proportionality could be dampened by the flowpath through the wetland. Conversely, the micropool was located at the wetland outlet and all short and long term TSS factors were observed directly in the wetland outflow. Additionally, the respective mean forebay and micropool effluent discharge rates of the 25-yr simulation period were 0.0140 and 0.0143 cfs in the base design, indicating that, on average, water flowed out of the forebay at a slightly slower rate than out of the micropool, which was likely due to the respective outflow devices controls used. Therefore, the corresponding short term effects of trap efficiency proportionality should have been slightly lower in the forebay.

In reality, TSS as well as the other water quality constituents would most likely not be completely mixed within each wetland cell, which would diminish the effect of the trap efficiency proportionality of cell depth. Influent TSS would most likely stay at or near the surface during high flow events, which would result in similarly high TSS effluent concentrations. Consequently, most TSS settling would occur during low- and zero-flow periods, which was shown by the model to be enhanced by a deeper micropool, which increased the micropool retention time. Based on the model results and their interpretation, the benefits of the increased deepwater storage in DW2 outweigh the apparent benefits of the decreased deepwater storage in DW1.

6.14.2.2 General water quality trends

The remaining water quality constituents DO, NH_4^+, and NO_3^- followed similar but stronger trends to those observed in the FB1 and FB2 designs. The increased retention time associated with the deeper micropool in design DW2 allowed for more nitrification, producing a slightly lower daily mean effluent NH_4^+ concentration of 0.0770 mg/L versus the base design concentration of 0.0782 mg/L. The shallower micropool in design DW2, in turn, produced slightly higher effluent NH_4^+ concentrations with a mean daily value of 0.0793 mg/L. These slight changes in NH_4^+ effluent concentrations resulted in a mean S_X of -0.0743 and were not found to be significant in the current study under the assumption that they had the same effect on downstream ecosystem health. However, these changes could be significant for a wetland design in which downstream species are extremely sensitive to NH_4^+ concentrations. Because the NH_4^+ reduction in the model was controlled by a simple first-order nitrification equation (see Equation 4-81), NH_4^+ levels in the wetland were a direct function of cell DO and NH_4^+ concentrations, and wetland water temperature. As a result, the nitrification rate itself was not directly affected by cell depth as was TSS settling. While NH_4^+ concentrations decreased , NO_3^- concentrations increased due to the increased nitrification in design DW2. Additionally, NO_3^- was not removed from the micropool because denitrification was not simulated within it as denitrification was only simulated in cells with emergent vegetation. This increase in mean daily effluent NO_3^- concentrations was evidenced by a mean S_X of 0.142.

Daily mean dissolved oxygen concentrations of the effluent were slightly less sensitive than daily effluent NH_4^+ concentrations , producing a mean S_X of -0.0624. The resulting negative S_X value for DO indicated that DO levels marginally decreased with increasing micropool depth. Because the micropool did not contain submerged vegetation, it did not receive oxygen from photosynthesis. Therefore, DO levels within the micropool were controlled by surface aeration. As discussed in Section 4.4.2.3.2, surface aeration within the model is a function of cell water depth, velocity, and the difference between the maximum saturation and current DO concentrations within a cell. While water temperature and velocity in the micropool were fairly unchanged in designs DW1 and DW2, the micropool depth was significantly altered. Surface aeration within the model was inversely related to cell depth (see Equation 4-93), which causes surface aeration to increase with decreasing cell depth. This trend was observed in the DW1 and DW2 designs as the shallower micropool in design DW1 resulted in slightly higher effluent DO concentrations while the deeper micropool in design DW2 produced slightly lower effluent DO concentrations than the base design. Because DO levels never reached levels near 2 mg/L, which was the minimum DO concentration for which nitrification was simulated, NH_4^+ concentrations were not affected by the trends observed in DO levels in the DW1 and DW2 wetland designs, but rather only by micropool retention time.

6.14.2.3 Deepwater storage volume conclusions

While changing the micropool depth did not significantly affect neither wetland hydrology not wetland water quality, a number of water quality trends were observed. The increased micropool storage provided in design DW2 promoted increasing trends in TSS settling and nitrification, which resulted in lower effluent TSS loads and NH_4^+ concentrations, and higher effluent NO_3^- concentrations. Additionally, effluent DO concentrations decreased slightly with increasing micropool depth as a result of the inverse relationship between surface aeration and cell depth. Due to the model structure, TSS settling was affected by both relative micropool retention time and of the trap efficiency proportion of micropool depth, which resulted in a higher effluent mean daily effluent TSS concentration in the shallower DW1 design. Despite these results, the current study assumed that in reality the retention time factor would most likely dominate in all wetland designs given that complete mixing of TSS would not occur in the micropool.

Based on these results, the model appeared to be weakly sensitive to changes in deepwater storage. While increased deepwater storage generally improved wetland TSS and NH_4^+ performance, it produced larger effluent NO_3^- concentrations and slightly lower effluent DO concentrations. As shown in Table 6-31, the final resulting wetland WSI scores were not significantly different with respective values of 0.641, 0.640, and 0.640 for the DW1, base, and DW2 designs. The current study hypothesized based on the model results that wetland performance was fairly insensitive to the MDE (2009) requirement that at least 25% of the WQ_v.

Furthermore, the these results suggest that greater focus should be put on determining the appropriate storage volumes and depths within each cell type (i.e., deepwater, high-marsh, and low-marsh) in order to achieve retention times within each cell that, combined, would achieve desired water quality and hydrologic performance.

Table 6-31 Resulting normalized metrics and final wetland sustainability indices (WSI's) for the base, DW1 and DW2 stormwater wetland designs. All WSI scores were computed assuming equal weights for all water quality and hydrologic metrics.

Performance Criteria	**Metric weights**	**Base design metrics**	**DW1 metrics**	**DW2 metrics**
Mean daily TSS Conc. (mg/L)	0.1	0.328	0.342	0.318
Mean daily DO Conc. (mg/L)	0.1	1	1	1
Mean daily NH4 Conc. (mg/L)	0.1	0.528	0.522	0.534
Mean daily NO3 Conc. (mg/L)	0.1	1	1	1
High-marsh PC	0.1	0.999	0.999	0.999
Low-marsh PC	0.1	1.000	1.000	1.000
High-Flow PC	0.1	0.533	0.533	0.533
Low-Flow PC	0.1	0.396	0.396	0.396
Flow-Variation PC	0.1	0.000	0.000	0.000
Flood-Control PC	0.1	0.618	0.618	0.618
Final *WSI* score	---	0.640	0.641	0.640

6.14.3 Location of deepwater areas

Based on the performance of the wetland designs FB1, FB2, DW1, and DW2, it was hypothesized that the location of deepwater areas within the wetland played a role in effluent water quality, especially with respect to TSS concentrations. In order to evaluate the sensitivity of wetland performance to the location of deepwater areas, three additional wetland designs were developed and simulated, which were referred to as MPB, MP1, and MP2. The MPB design switched the micropool cell (cell 1) with cell 15, which had a water depth of 3 ft. Therefore, in the MPB design, cell 15 had a depth of 5.75 ft and a smooth bottom without vegetation while cell 1, the outlet cell, had a water depth of 3 ft with submerged vegetation. By moving the micropool to the middle of the wetland, the current study aimed to analyze the effect of deepwater locations within the wetland. Both the MP1 and MP2 designs followed the same cell structure as that of the MPB design, with the micropool located in cell 15. The MP1 design incorporated a relocated micropool with a depth of 4 ft and the MP2 design incorporated a relocated micropool with a depth of 7.72 ft. Therefore the MP1 and MP2 designs are analogous to the DW1 and DW2 designs, the only difference being the location of the micropool within the wetland.

6.14.3.1 MPB design results

The MPB design performance revealed that deepwater location within the wetland had a significant impact on wetland water quality performance, but minimal effect on hydrologic performance. MPB design hydrologic PC values were changed by the relocation of the micropool in the MPB design as illustrated through the computed relative errors E_X comparing the base and MPB hydrologic PC values in Table 6-33. E_X values rather S_X values were used to quantify changes associated with the MPB design as the relocation of the micropool did not translate to a quantifiable input change δX that was necessary for the computation of S_X via Equation 6-58. Despite the hydrologic PC insensitivities, the internal wetland velocity did increase from 0.000658 ft/s in the base design to 0.000722 ft/s in the MPB. This increased internal velocity was due to the replacement of the initial cell 15, which had submerged vegetation, with the micropool, which was deeper and did not include vegetation. Within the model, cells without vegetation were assigned a roughness coefficient of 0.017 while the roughness coefficients for cells with submerged vegetation could range from 0.135 to 21.9, decreasing with depth.

Increased TSS settling was also observed in the MPB design as evidenced by the MPB mean daily effluent TSS concentration of 13.9 mg/L versus the base value of 15.4 mg/L. This decrease in effluent TSS in the MPB design was due to the effect of the increased cell 15 depth coinciding with water containing higher TSS concentrations. Deepwater cells appeared to be more effective in the middle of the wetland because there was a greater amount of TSS in the water. The increased cell storage appeared to outweigh the effect of the slightly increased velocity that resulted from the exclusion of vegetation in the internal cell. These results suggest that the incorporation of deepwater areas throughout the wetland, not solely at the inlet and outlet, could significantly improve wetland TSS performance. The current model, however, did not support the removal of deepwater areas at the outlet despite model results due to possible TSS resuspension associated with shallower areas (MDE 2009).

The effluent NH_4^+ concentrations did not change in the MPB design. This NH_4^+ insensitivity to micropool location was likely due to the low reaction rate of nitrification relative to TSS settling and denitrification rates. As a result, the relocation of the micropool did not have the same effect on NH_4^+ concentrations as it did on TSS concentrations. Effluent NO_3^- concentrations in the MPB design were, however, marginally lower than those in the base design. While denitrification was not simulated in the micropool cell, the increased velocity in cell 15 in the MPB design may have pushed more water into high and low-marsh cells, in which denitrification was simulated, allowing for a longer net retention time in denitrifying cells.

Effluent DO concentrations increased marginally from 10.3 mg/L in the base design to 10.5 mg/L in the MPB design. This slight increase in effluent DO concentrations was due to the addition of submerged vegetation in the outlet cell, which promoted photosynthesis in addition to surface aeration. Because less NH_4^+ was present in the outlet cell, this photosynthesis-generated DO was not used up at the same rate as when it was generated in cell 15 in the base design. This effect was not significant, as it did not affect the ability of the wetland to perform nitrification nor did it affect downstream ecosystem health. However, it should be noted for future wetland design that water aeration is more efficient when NH_4^+ are lower and do not demand as much DO for nitrification. Therefore, submerged vegetation could be placed near the outlet of wetlands, after most of the NH_4^+ has been nitrified, in which low effluent DO levels are of concern in order to increase effluent DO concentrations.

Based on the water quality performance of the MPB wetland design, it was concluded that location of deepwater areas could have a significant impact of effluent TSS concentrations. DO, NH_4^+, and NO_3^- concentrations, however, appeared to be fairly insensitive to deepwater location. Therefore, the current study strongly suggests the incorporation of deepwater areas throughout a wetland design when effluent TSS concentrations are of concern. As cited by MDE (2009), deepwater areas at the inlet and outlet are crucial for the initial pre-treatment of larger TSS particles at the inlet and for the reduction of TSS resuspension at the outlet. However, deepwater areas within the main wetland body could improve TSS performance. The addition of submerged vegetation to internal wetland deepwater areas could also promote even better water quality performance due to (1) the slower velocities associated with submerged vegetated vs non-vegetated area, and (2) the increased DO production associated with the photosynthesis of submerged vegetation.

Table 6-32 Relevant PC values and relative errors E_x observed between the base and MPB stormwater wetland designs.

Performance Criteria	**Base**	**MPB**	E_x
Mean daily TSS Conc. (mg/L)	15.4	13.9	-0.099
Mean daily DO Conc. (mg/L)	10.3	10.5	0.023
Mean daily NH4 Conc. (mg/L)	0.0782	0.0782	0.001
Mean daily NO3 Conc. (mg/L)	0.258	0.245	-0.049
High-marsh PC	0.962	0.962	0.00
Low-marsh PC	0.983	0.983	0.00
High-Flow PC	0.316	0.316	0.00
Low-Flow PC	0.223	0.223	0.00
Flow Variation PC	2.90	2.90	0.00
Flood Control PC	0.382	0.382	0.00

6.14.3.2 Comparing MPB with the MP1 and MP2 designs

In order to evaluate the effect of deepwater storage change on the relocated micropool in the MPB design, the resulting PC values for the MP1 and the MP2 designs were compared with those of the MPB design using both S_X and D_X sensitivity measures (see Table 6-33). The same increasing trends in both hydrologic and water quality performance with increasing storage were seen in designs MP1 and MP2 as were seen in the FB1 and FB2 designs.

While changes were observed in all water quality PC values, only the mean daily effluent NH_4^+ concentrations were found to change significantly with a mean S_X of -0.072. This decreasing trend in effluent NH_4^+ concentrations indicated that the increased retention time associated with the increased micropool depth in design MP3 allowed for slightly more nitrification of NH_4^+. The resulting effluent NO_3^- concentrations increased marginally with increasing micropool depth, with a mean S_X of -0.022 due to the corresponding increased micropool nitrification and lack of denitrification within the micropool, which was only simulated in cells with emergent vegetation. Effluent TSS concentrations decreased marginally with increasing micropool depth with a S_X of -0.014. Therefore, similar to the trend seen in the forebay, the factor of retention time appeared to dominate slightly over the trap efficiency proportionality factor.

While all resulting S_X values indicated model performance was fairly insensitive to the changes made to the MPB design in the MP1 and MP2 designs, these designs did exhibit the same water quality trends as those observed in the designs FB1 and FB2. Final WSI values for the base, MPB, MP1, and MP2 designs were computed and summarized in Table 6-36, which further showed the model insensitivity to the changes associated with these designs with respective WSI scores of 0.640, 0.643, 0.642, and 0.644. The slight increase in TSS performance from the base to the MPB design, did suggest that deepwater areas should be incorporated throughout the wetland. The water quality trends seen in the MP1 and MP2 designs also reinforced that cell storage/depth was the dominating factor in increasing cell retention time. Therefore, a definitive total storage volume devoted to deepwater may not be the most useful method of wetland design given that its usefulness depends on a number of factors including location within the wetland and nitrogen species present in inflow.

Table 6-33 Relevant PC values, relative sensitivities, and deviation sensitive for the MPB, MP1, and MP2 stormwater wetland designs. The relationship direction indicates where the PC value increases (+) or decreases (-) with a corresponding increase in deepwater storage in the wetland.

Performance Criteria	**MPB**	**MP1**	**MP2**	S_x	$\lvert D_x \rvert$	**Relationship direction**
Mean daily TSS Conc. (mg/L)	13.9	13.9	13.8	-0.014	0.037	-
Mean daily DO Conc. (mg/L)	10.5	10.5	10.5	0.008	0.016	+
Mean daily NH4 Conc. (mg/L)	0.0782	0.0793	0.0771	-0.072	0.001	-
Mean daily NO3 Conc. (mg/L)	0.245	0.244	0.246	0.022	0.001	+
High-marsh PC	0.962	0.962	0.962	0.00	0.00	-
Low-marsh PC	0.983	0.983	0.983	0.00	0.00	+
High-Flow PC	0.316	0.316	0.316	0.00	0.00	-
Low-Flow PC	0.223	0.223	0.223	0.00	0.00	-
Flow-Variation PC	2.90	2.90	2.90	0.00	0.00	+
Flood-Control PC	0.382	0.382	0.382	0.00	0.00	+

Table 6-34 Resulting normalized metrics and final wetland sustainability indices (WSI's) for the base, DW1 and DW2 stormwater wetland designs. All WSI scores were computed assuming equal weights for all water quality and hydrologic metrics.

Performance Criteria	Metric weights	Base design metrics	MPB metrics	MP1 metrics	MP2 metrics
Mean daily TSS Conc. (mg/L)	0.1	0.328	0.357	0.356	0.357
Mean daily DO Conc. (mg/L)	0.1	1	1	1	1
Mean daily NH4 Conc. (mg/L)	0.1	0.528	0.528	0.522	0.534
Mean daily NO3 Conc. (mg/L)	0.1	1	1	1	1
High-marsh PC	0.1	0.999	0.999	0.999	0.999
Low-marsh PC	0.1	1.000	1.000	1.000	1.000
High-Flow PC	0.1	0.533	0.533	0.533	0.533
Low-Flow PC	0.1	0.396	0.396	0.396	0.396
Flow-Variation PC	0.1	0.000	0.000	0.000	0.000
Flood-Control PC	0.1	0.618	0.618	0.618	0.618
Final *WSI* score	---	0.640	0.643	0.642	0.644

6.14.4 High-marsh areas

The current section developed two designs to test the importance of high-marsh areas, which were defined by MDE (2009) to have with water depths of 0.5 ft or less. MDE (2009) also required 35% or more of the required wetland surface area SA_o allotted to these high-marsh areas. The initial design accounted for 36% of the SA_o with water depths of 0.5 ft. One test wetland was designed with 32% of the SA_o allocated to high-marsh areas and was referred to as the HM1 design. A second test design, HM2, was made with 40% of the SA_o allocated to high-marsh areas. The HM1 design was made by increasing the water depth in the high-marsh cell 14 of the base design (see Figure 6-4 and Table 6-4) from 0.5 to 1.25 ft while maintaining emergent vegetation in the cell. Conversely, the HM2 design was developed by decreasing the water depth in low-marsh cell 11 from 1.25 to 0.5 ft while, again, maintaining emergent vegetation in the cell. Therefore, the resulting HM1 and HM2 designs had respective water volumes of 5,957 and 5,749 ft^3 (0.137 and 0.132 ac-ft) as recorded in Table 6-39.

The conversion of high-marsh areas to deeper, low-marsh areas in the design HM1 increased cell storage as well as cell velocity, each of which had opposing effects on cell retention time. While greater cell storage was shown to increase cell retention time in Sections 6.14.1.1, 6.14.2.2, and 6.14.3, corresponding increased velocity was found to decrease cell retention time, allowing water to flow more quickly through the cell. The respective mean internal wetland velocities for the HM1, base, and HM2 designs were 0.000688, 0.000658, and 0.000498 ft/s. Therefore, a larger proportion of high-marsh area translated to less storage and slower internal wetland velocities. Within the model, velocity in cells with emergent vegetation was computed according to Equation 4-50, which is reproduced here:

$$n_D = \begin{cases} 33.8 & \text{for } SS < 0.328\,\text{ft} \\ 0.673 \cdot (SS/3.28)^{-1.7} & \text{for } 0.328 \le SS \le 3.28\,\text{ft} \\ 0.673 & \text{for } SS > 3.28\,\text{ft} \end{cases} \tag{6-68}$$

where n_D represents the roughness coefficient value (s/$ft^{1/3}$) for densely vegetated wetland areas, and *SS* represents the surface storage depth (ft). Therefore, the increase in depth in cell 14 in the HM1 design resulted in a faster cell velocity. Conversely, the reduction of cell 11 depth in the HM2 design resulted in a slower cell velocity.

Wetland hydrologic performance was weakly affected by the changes in high-marsh areas associated with the HM1 and HM2 designs. The high-flow and flow variation PC values increased slightly with increasing high-marsh area with respective S_X values of 0.0561 and 0.0944. While the high-flow PC represented the ratio of the proportion of pre-development to effluent flow exceeding pre-development bankfull flow, the flow variation PC value was the ratio of pre-development to effluent mean daily flow variation. Therefore, increases in both of these PC values implied a decrease in effluent values relative to the corresponding pre-development values. As a result, it was concluded that the lower internal velocities associated with more high-marsh areas in the HM2 design also produced slightly slower and less variable effluent discharge rates. Similarly, the low-flow PC value, which was the ratio of the proportion of pre-development to effluent non-zero flows, decreased with increasing high-marsh area in the wetland. This trend in the low-flow PC values indicated that while the HM2 design promoted lower internal velocities and effluent flowrates, these lower flowrates occurred over a slightly longer duration than those in the base design. This trend was also evidenced by the increase in the total number of days producing effluent flow from 262 in the base design to 263 days in the MP2 design. While this change was small, it could prove significant depending on the sensitivity of downstream ecosystems to the duration a frequency of zero-flow periods. For example, the loss of seasonal pre-development dry periods could prevent sensitive downstream species from reproducing. Therefore, the significance of changes in the low-flow and high-flow is dependent on the corresponding effects on the ability of the wetland to promote healthy downstream ecosystems.

Because increasing high-marsh areas resulted in both decreased cell storage and increased cell velocity, such changes had conflicting effects on wetland water quality performance. As a result, TSS and NH_4^+ effluent concentrations increased in both the HM1 and HM2 designs. TSS S_X values for HM1 and HM2 were -0.014 and 0.368 and NH_4^+ S_X values for HM1 and HM2 were -0.0593 and 0.215. Based on these results, the storage factor appeared to be more important as larger increases in TSS and NH_4^+ effluent concentrations were observed in the HM2, which had less storage relative to the base design. The TSS and NH_4^+ behavior exhibited in the HM1 and HM2 designs suggests that within the model, increasing cell storage is more effective in increasing cell retention time than is decreasing cell depth in order to increase the vegetated roughness coefficient. However, this trend assumes complete mixing of pollutants in each user-define wetland cell, which may not represent reality. More data are necessary to better understand how constituents such as TSS and NH_4^+ move through constructed wetlands.

The mean effluent NO_3^- concentrations also reflected the dominating effect of cell storage over cell velocity. While effluent NO_3^- concentrations increased with increasing high-marsh area, the respective S_X values for each the HM1 and HM2 designs were 0.137 and 0.419, showing that NO_3^- concentrations were more sensitive to the loss of cell storage in the HM2 design than to the increase in cell velocity in design HM1. In addition to cell storage and velocity, NO_3^- concentrations were also influenced by NH_4^+ concentrations. Due to these compounding factors, effluent NO_3^- concentrations were the most sensitive output parameter, with a mean S_X value of 0.278. Based on these results, within the model, increasing cell storage appeared to be the most effective design change in reducing NO_3^- concentrations.

Given these results, changing the high-marsh surface area within the wetland did not significantly change overall wetland performance as the final WSI values for the HM1, base, and HM2 designs were almost equal with values of 0.640, 0.640, and 0.639 (see Table 6-38). Therefore, based on the final WSI scores, high and low-marsh areas did not have significantly different impacts on wetland performance within the model. In reality, high and low-marsh areas may be required for the survival of different vegetation types, making both important elements within a healthy stormwater wetland design. The current model, however, does not simulate vegetation death and assumes all vegetation types are able to grow in all water depths. Given these model assumptions, user knowledge of vegetation needs would be crucial in order to correctly place vegetation within a given wetland design based on water depths and pollutant concentrations. Despite the insensitivity of model output to changes in the high-marsh area, the results from the HM1 and HM2 designs reinforced that cell volume was a dominating factor in wetland performance. As a result, it was further emphasized that wetlands should be designed based on the relative cell type storage volumes and the corresponding retention times rather than on the relative surface areas.

Table 6-35 Relevant PC values, relative sensitivities, and deviation sensitive for the base , HM1, and HM2 stormwater wetland designs. The relationship direction indicates where the PC value increases (+) or decreases (-) with a corresponding increase in high-marsh area in the wetland. S_X values in parenthesis were computed using the absolute values of those from HM1 and HM2 because they had both positive and negative effects on the corresponding PC value.

Performance Criteria	Base	HM1	HM2	S_x	$\lvert D_x \rvert$	Relationship direction
Mean daily TSS Conc. (mg/L)	15.4	15.4	16.0	(0.19)	0.33	+/-
Mean daily DO Conc. (mg/L)	10.3	10.3	10.3	(0.01)	0.01	+/-
Mean daily NH4 Conc. (mg/L)	0.0782	0.0787	0.0800	(0.14)	0.00	+/-
Mean daily NO3 Conc. (mg/L)	0.258	0.254	0.270	0.278	0.01	+
High-marsh PC	0.962	0.963	0.955	-0.0376	0.00402	-
Low-marsh PC	0.983	0.983	0.981	-0.00912	0.000996	-
High-Flow PC	0.316	0.315	0.314	0.0561	0.00197	+
Low-Flow PC	0.223	0.224	0.220	-0.0904	0.00224	-
Flow-Variation PC	2.90	2.88	2.95	0.0944	0.03038	+
Flood-Control PC	0.382	0.382	0.382	0.00001	0.00	+

Table 6-36 Resulting normalized metrics and final wetland sustainability indices (WSI's) for the base, HM1 and HM2 stormwater wetland designs. All WSI scores were computed assuming equal weights for all water quality and hydrologic metrics.

Performance Criteria	Metric weights	Base design metrics	HM1 metrics	HM2 metrics
Mean daily TSS Conc. (mg/L)	0.1	0.328	0.327	0.328
Mean daily DO Conc. (mg/L)	0.1	1	1	1
Mean daily NH4 Conc. (mg/L)	0.1	0.528	0.525	0.522
Mean daily NO3 Conc. (mg/L)	0.1	1	1	1
High-marsh PC	0.1	0.999	0.999	0.998
Low-marsh PC	0.1	1.000	1.000	1.000
High-Flow PC	0.1	0.533	0.530	0.529
Low-Flow PC	0.1	0.396	0.398	0.395
Flow-Variation PC	0.1	0.000	0.000	0.000
Flood-Control PC	0.1	0.618	0.618	0.618
Final *WSI* score	---	0.640	0.640	0.639

6.14.5 Low-marsh areas

MDE (2009) required that 65% or more of the SA_o allotted to high and low-marsh areas combined, which had water depths ≤ 1.5 ft. Therefore, low-marsh areas were required to compose 29% of the SA_o in the base design given that high-marsh areas composed 36% of the SA_o. The current study evaluated the importance of the proportion of the SA_o allotted low-marsh areas through the use of two test wetlands that incorporated (1) low-marsh areas composing 28% of the SA_o, and (2) low-marsh areas composing 36% of the SA_o. These designs were respectively referred to as the LM1 and LM2 designs. The LM1 design was developed by increasing the depth in the low-marsh cell 16 of the base design (see Figure 6-4 and Table 6-4) from 1.25 to 3 ft and by replacing the emergent vegetation with submerged vegetation. Conversely, the LM2 design was made by decreasing the water depth in cell 15 from 3 to 1.25 ft and by replacing the submerged vegetation with emergent vegetation.

Wetland performance in the LM1 and LM2 designs was controlled by the following three factors, which included (1) cell depth, (2) internal cell velocity, and (3) cell vegetation type. Each of these three factors directly affected cell retention time. Additionally, internal cell velocity was a function of both cell depth and cell vegetation type. Replacing low-marsh cells with deeper cells with submerged vegetation, as was done in the LM1 design, resulted in greater wetland storage. This design alteration, however, also incorporated less emergent vegetation and therefore, fewer cells in which denitrification was simulated. As a result, in addition to the influencing factors of cell storage and velocity that dominated in the HM1 and HM2 designs, the LM1 and LM2 design performance was also complicated by the factor of vegetation type.

Due to the effect of the proportion of low-marsh areas on wetland velocity, wetland hydrologic performance was weakly sensitive to the design changes made in the LM1 and LM2 designs. Respective mean internal wetland velocities for the base, LM1, and LM2 designs were 0.000658, 0.000660, and 0.000652 ft/s, showing that velocity increased slightly with increasing cell depth as well with the replacement of emergent vegetation with submerged vegetation. As observed in the HM1 and HM2 designs, the high-flow and flow variation PC values increased marginally with decreasing low-marsh area with respective S_X values of 0.055 and 0.049 (see Table 6-35). These values indicated that deeper cells promoted higher wetland velocities and discharge rates with greater variability. Additionally, the replacement of submerged for emergent vegetation further increased the cell velocity as submerged vegetation was assumed to be less dense and was simulated with lower roughness coefficients according to the following equation (see Section 4.4.1.2.1):

$$n_S = \begin{cases} 21.9 & \text{for } SS < 0.164\,\text{ft} \\ 0.673 \cdot 0.2 \cdot (SS/3.28)^{-1.7} & \text{for } 0.164 \le SS \le 3.28\,\text{ft} \\ 0.135 & \text{for } SS > 3.28\,\text{ft} \end{cases} \tag{6-69}$$

where n_S represents the roughness coefficient value (s/ft$^{1/3}$) for wetland areas with submerged vegetation.

Wetland water quality performance, especially that of effluent NO_3^- concentrations, was found to be sensitive to changes in the proportion of low-marsh areas within the wetland. Mean daily effluent TSS concentrations in the LM1 and LM2 designs showed similar but weaker trends to those seen in the HM1 and HM2 designs with respective S_X values for LM1 and LM2 of -0.014 and 0.019, which were essential zero. Both decreasing and increasing the low-marsh area in the wetland had the same effect on TSS concentrations, which suggested that the increased cell storage in the LM1 design and the decreased cell velocity in the LM2 design had similar effects on cell TSS settling. The cell storage factor did, however, dominate with respect to mean daily effluent NH_4^+ concentrations, which increased with increasing low-marsh area and subsequent increasing wetland storage with a mean S_X value of 0.119. Effluent NH_4^+ concentrations did also show some sensitivity to the velocity factor as designs LM1 and LM2 produced respective NH_4^+ S_X values of 0.041 and 0.196, which suggested that, while cell storage was the dominating factor, the decreased cell velocity associated with the shallower low-marsh cell in the LM1 design dampened its effect. Mean daily effluent DO concentrations were marginally greater in the LM1 design with a value of 10.3 mg/L versus the LM2 design, which produced a value of 10.2 mg/L. In this case, DO levels were affected by the following three factors: (1) photosynthesis via submerged vegetation, (2) cell water velocity, and (3) cell water depth. Mean effluent NO_3^- concentrations were the wetland outputs most sensitive to changes in low-marsh area with a mean S_X value of -1.05. The negative sign of the NO_3^- S_X indicated that NO_3^- concentrations decreased with increasing low-marsh area. NO_3^- concentrations were very sensitive to the loss and addition of cells with emergent vegetation because the model only simulated denitrification in these cells. Therefore, the conversion of low-marsh areas to deeper areas with submerged vegetation resulted in a proportional reduction in the retention time within the wetland devoted to denitrification of NO_3^-.

While final wetland WSI scores for the LM1, base, and LM2 designs were not significantly different (see Table 6-38), it was noted that effluent NO_3^- concentrations were very sensitive to the loss of low-marsh areas with emergent vegetation. All other water quality and hydrologic PC values were minimally sensitive to the LM1 and LM2 designs. The current study did, however, observe a number of trends in wetland performance in the LM1 and LM2 designs that further reinforced the importance of design wetlands with cell retention times in mind. Low and high-marsh retention times were found to be crucial to NO_3^- reduction due to their inclusion of emergent vegetation. Retention time in deepwater areas was also found to effect TSS and NH_4^+ concentrations. Based on these results, the current study concluded that marsh (high + low-marsh) and deepwater cells were important for water quality performance success in stormwater wetland designs. It was also found that cells with submerged vegetation also served to help improve wetland water quality performance.

Table 6-37 Relevant PC values, relative sensitivities, and deviation sensitive for the base , LM1, and LM2 stormwater wetland designs. The relationship direction indicates where the PC value increases (+) or decreases (-) with a corresponding increase in low-marsh area in the wetland.

Performance Criteria	Base	LM1	LM2	S_x	$\lvert D_x \rvert$	Relationship direction
Mean daily TSS Conc. (mg/L)	15.4	15.4	15.5	(0.02)	0.07	+/-
Mean daily DO Conc. (mg/L)	10.3	10.32	10.19	-0.074	0.06	-
Mean daily NH4 Conc. (mg/L)	0.0782	0.0778	0.0793	0.119	0.00	+
Mean daily NO3 Conc. (mg/L)	0.258	0.277	0.242	-1.053	0.02	-
High-marsh PC	0.962	0.963	0.961	-0.023	0.00	-
Low-marsh PC	0.983	0.983	0.983	-0.007	0.00	-
High-Flow PC	0.316	0.315	0.317	0.055	0.00	+
Low-Flow PC	0.223	0.224	0.222	-0.070	0.00	-
Flow-Variation PC	2.90	2.89	2.903	0.049	0.01	+
Flood-Control PC	0.382	0.382	0.382	0.000	0.00	+

Table 6-38 Resulting normalized metrics and final wetland sustainability indices (WSI's) for the base, LM1 and LM2 stormwater wetland designs. All WSI scores were computed assuming equal weights for all water quality and hydrologic metrics.

Performance Criteria	Metric weights	Base design metrics	LM1 metrics	LM2 metrics
Mean daily TSS Conc. (mg/L)	0.1	0.328	0.328	0.326
Mean daily DO Conc. (mg/L)	0.1	1	1	1
Mean daily NH4 Conc. (mg/L)	0.1	0.528	0.522	0.522
Mean daily NO3 Conc. (mg/L)	0.1	1	1	1
High-marsh PC	0.1	0.999	0.998	0.998
Low-marsh PC	0.1	1.000	1.000	1.000
High-Flow PC	0.1	0.533	0.529	0.534
Low-Flow PC	0.1	0.396	0.395	0.395
Flow-Variation PC	0.1	0.000	0.000	0.000
Flood-Control PC	0.1	0.618	0.618	0.618
Final *WSI* score	---	0.640	0.639	0.639

Table 6-39 All tested shallow wetland designs and their corresponding cell lengths (ft), number of cells, wetland surface area (ft^2), storage volume (ft^3), the areal proportion of shallow high-marsh (HM1) areas (depth of less than or equal to 0.5 ft), the areal proportion of total high-marsh (HM) areas (depths less than or equal to 1.5 ft), the volumetric proportion of deepwater zones (greater than or equal to 4 ft), and the volumetric proportion of storage in the forebay. According to MDE (2009) procedure, areal proportions were based on the required wetland surface area SA_o of 3,463 ft^2 and volumetric proportions were based on the WQ_v of 5,396 ft^3. Lightly shaded cells represent a significant change in wetland design while darkly shaded cells indicate that a given criterion was not met.

Design	Design change	SA_o % of drainage area (%)	Wetland surface area (ft^2)	Wetland storage volume (ft^3)	HM1 area % of SA_o	HMT area % of SA_o	Deepwater zone volume % of WQ_v	Forebay volume % of WQ_v
MDE specified requirements	---	≥1.5	≥ 3463	≥ 5396	≥ 35	≥ 65	≥ 25	≥ 10
Base design	---	1.5	3463	5853	36	68	25	10.3
DA1	Wetland surface area 1.25% of DA	1.25	3463	5853	36	68	25	10.3
DA2	Wetland surface area 2.5% of DA	2.5	3463	5853	36	68	25	10.3
DW1	Deepwater zones 20.5% of WQ_v	1.5	3463	5611	36	68	20.5	10.3
DW2	Deepwater zones 30.3% of WQ_v	1.5	3463	6126	36	68	30.3	10.3
FB1	Forebay 25% of WQ_v	1.5	3463	6650	36	68	39.8	25.0
FB2	Forebay 1.8% of WQ_v	1.5	3463	5396	36	68	16.6	1.8
MP1	Micropool moved to center of wetland	1.5	3463	5853	36	68	25	10.3
MP2	Deepwater areas 20.5% of WQ_v with relocated micropool	1.5	3463	5611	36	68	20.5	10.3
MP3	Deepwater area 30.3% of WQ_v with relocated micropool	1.5	3463	6126	36	68	30.3	10.3
HM1	High-marsh areas 32% of SA_o	1.5	3463	5957	32	68	30.3	10.3
HM2	High-marsh areas 40% of SA_o	1.5	3463	5749	40	68	30.3	10.3
LM1	Low-marsh areas 28% of SA_o	1.5	3463	5957	36	64	30.3	10.3
LM2	Low-marsh areas 36% of SA_o	1.5	3463	5610	36	72	30.3	10.3

6.14.6 Stormwater design criteria suggestions

Based on the trends observed in model sensitivity to MDE (2009) design criteria, the current study suggested that the retention time within each cell type (i.e., high-marsh, low-marsh, deepwater, etc.) be the focus of wetland design. It was also found that the two main factors affecting cell retention time within the model were (1) cell storage/depth and (2) cell velocity. Cell depth and velocity were directly related in vegetated cells due to the equations used in the model to define vegetation roughness coefficients (see Equations 4-50 and 4-51). Therefore, while increasing cell depth increased cell storage, which promotes a longer cell retention time, it also increased cell velocity, which promotes a shorter cell retention time. Despite this relationship, the increased storage produced by increasing cell depth appeared to dominate over the corresponding increase in cell velocity with respect cell retention time and to water quality performance. Hydrologic performance was not strongly influenced by these factors, but was rather dependent on wetland surface area and outlet structure. These hydrologic results suggested that additional wetland design structural changes would be required to produce effluent flows that better mimicked corresponding pre-development hydrology.

6.14.6.1 Retention time determination via TSS pulse experiment

In an effort to quantify the relationship between cell pollutant concentrations and cell depth, the current study performed a simple pulse experiment within one wetland cell. This simulated tracer experiment estimated cell retention time by measuring the time required for internal and effluent cell TSS concentrations to return to zero after a 1-min influent pulse with a volume of 0.84 ft^3 and a TSS concentration of 43.2 mg/L was introduced into the cell. Both influent volume and TSS concentrations were based on values used/observed in the base design. While TSS was only introduced into the cell during the first minute of simulation, the influent discharge rate was set to a constant rate of 0.0140 cfs over duration of the simulation. This rate was equal to the mean discharge rate leaving the forebay in the base stormwater wetland design. Settling was not simulated in this experiment so as to measure solely the effects of cell vegetation and depth on cell retention of pollutants. This pulse experiment was performed separately for a cell with emergent vegetation cell and for a cell with submerged vegetation. Additionally, the TSS pulse was simulated over a cell water depth range of 0.1 to 1.5 ft within the emergent vegetation cell, matching the MDE-defined water depth range for high + low-marsh ($\leq$ 1.5 ft). Similarly, the TSS pulse was simulated over a cell water depth range of 1.5 to 4 ft was in order to analyze the effect of cell depth on retention time in cells with submerged vegetation; this water depth range was not defined by MDE (2009). Deepwater areas, with water depths greater than 4 ft, were not included in this analysis given that they did not incorporate vegetation and, therefore, were simulated to have a constant roughness coefficient of 0.017 regardless of cell water depth. Obtained retention times for these two pulse simulation runs are shown in Figure 6-16.

In the case of both emergent and submerged vegetation, a linear relationship between the estimated cell retention time and depth was observed (see Figure 6-16). These linear trends illustrate that the increased cell storage rather than increasing cell velocity associated with increasing cell depth was the dominating factor that controls retention time. In addition to increasing cell retention time, greater cell depths also allowed for greater dilution of influent concentrations as is shown in Figure 6-17. While this dilution effect may be negligible if steady-state conditions are reached within the wetland as they are within the stormwater model, influent TSS concentrations are often variable in reality. Therefore, in reality, the dilution factor may play a larger role in TSS and general pollutant concentration reduction than is simulated in the model.

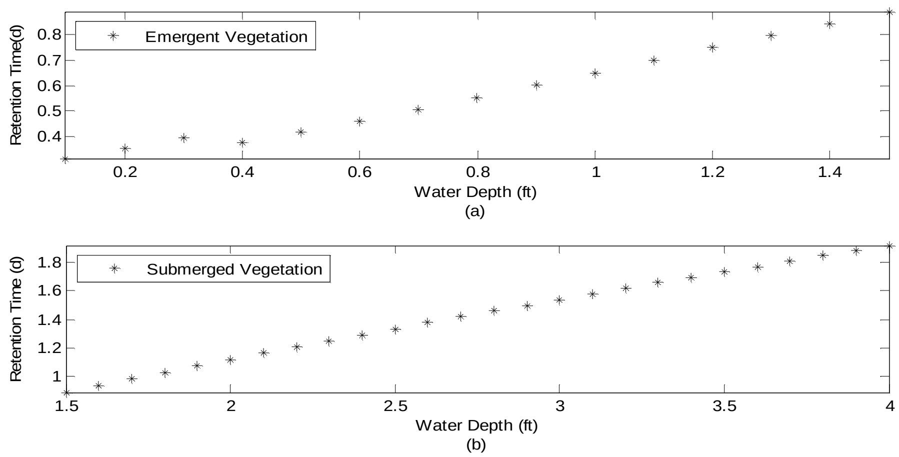

Figure 6-16 Estimated cell retention times resulting from the pulse experiments for a wetland cell with (a) emergent vegetation and with (b) submerged vegetation.

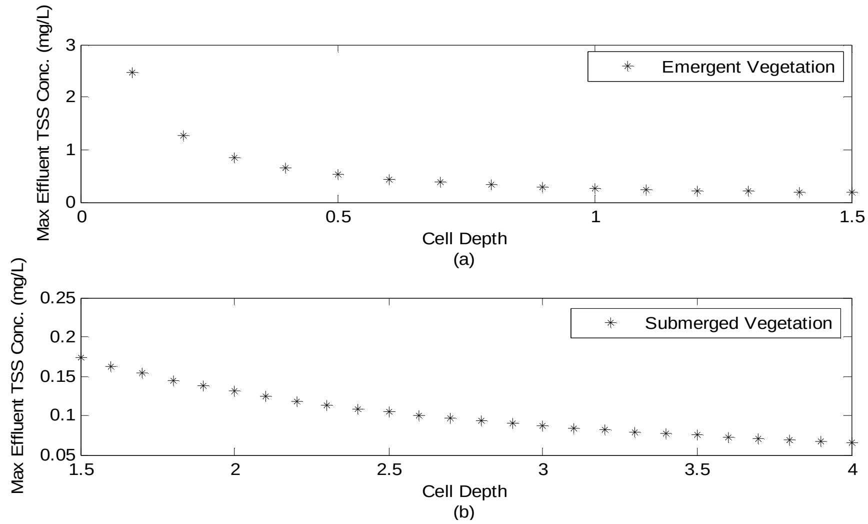

Figure 6-17 Maximum effluent TSS concentrations (mg/L) associated with varying cell water depths for (a) emergent and (b) submerged vegetation.

It was also noted that because each cell was modeled as a completely mixed flow reactor (CMFR), the 1-min influent TSS pulse produced effluent TSS concentrations that exponentially decayed over time (see Figure 6-18) rather than all at once as would a cell modeled as a plug-flow reactor (PFR). Given this cell behavior, cell retention time is difficult to define as all TSS introduced into the cell does not leave at the same time. Despite the exponentially distributed nature of effluent TSS concentrations, increasing cell depth produced more gradual effluent TSS decay, indicating that deep cells promoted slower release of TSS from the wetland cell.

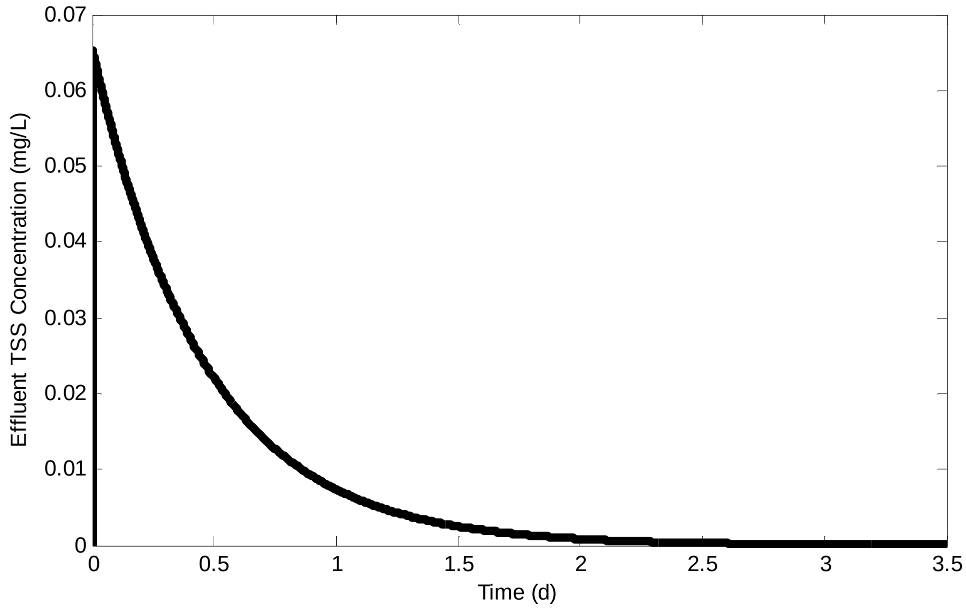

Figure 6-18 Example plot of effluent TSS concentrations (mg/L) vs. time (days) for a cell with a water depth of 4 ft with submerged vegetation.

6.14.6.2 Final water quality design suggestions

Wetland water quality performance design criteria were found to be most sensitive to (1) cell type retention time and (2) cell type location within the wetland. As quantified in Section 6.14.6.1, vegetated cell retention times were found to increase with increasing cell depth. High and low-marsh retention times were especially important for NO_3^- as the model only simulated denitrification within these cells. Additionally, Section 6.14.3.1 revealed that the location of deepwater areas within the wetland could greatly affect effluent TSS concentrations. Based on these results, the current study suggests that the water depths for each cell type (i.e., high-marsh, low-marsh, deepwater areas) should be made as deep as possible while still supporting their intended vegetation and habitat health so as to maximize wetland storage and retention time. Deepwater areas should also be incorporated throughout a wetland and not just located at the inlet and outlet. The distribution of retention times should also be optimized over all wetland cell types.

6.14.6.3 Final hydrologic design suggestions

While wetland hydrologic performance was weakly sensitive, with S_X values ranging from 0.049 to 0.944, to vegetation type and cell depth, it was most sensitive to wetland surface area and the orifice diameter of the outlet. Current stormwater wetland designs focus only on inflow volume and peak discharge reduction. The current study suggests that emphasis be placed on mimicking estimated pre-development hydrology.

The wetland did not perform well hydrologically based on the PC and metric values developed in the current study, which were designed with the goal of comparing the pre-development and wetland effluent hydrologic regimes. Therefore, while the stormwater wetland succeeded in reducing effluent peak flows as MDE (2009) intended, it did not successfully mimic pre-development hydrology as it was defined in the current study. Because natural stream health is tied to stream hydrology (Poff et al. 1997; Walsh et al. 2005; DePhillip and Moberg 2010), BMP facilities should be designed to reproduce downstream natural hydrology.

Effluent volume was targeted to be the most important factor affecting wetland hydrologic performance. The base wetland design produced, on average, about 2.61 times more effluent volume per year than the estimated pre-developed drainage area. Therefore, the current study suggested that future wetland designs promote more infiltration, perhaps by adding a wetland section with sandy soil near the outlet that is separated by a berm from the main wetland area so as to avoid drying out of the wetland. Additional infiltration would then reduce effluent volumes and reduce durations over which outflow discharges were maintained.

The current study also suggests the reevaluation of the method used to size wetland outlet orifices. Currently, MDE (2009) sizes orifices to drain the 1-yr, 24-hr storm event 24 or in 12 hours depending on the location of a wetland site. While this orifice sizing method reduces peak flows, it also promotes longer flow durations, which could prove detrimental to downstream ecosystem health. While the root of this problem lies in the fact that stormwater wetlands generally produce much more effluent volume than would an analogous pre-developed area, perhaps rethinking the orifice design could help promote a more natural wetland effluent flow regime.

Overall, the MDE (2009) design criteria evaluated in the current study guided the design of a stormwater wetland that performed well with respect to effluent water quality but poorly hydrologically. While specific, numerical design criteria such as those defined by MDE (2009) make wetland design easier, it was found that they do not necessarily result in optimal wetland designs. The current study suggests that stormwater wetlands be designed according to the characteristics and specific goals of each individual site. For a given wetland design, retention time within all relevant cell types (i.e., high-marsh, low-marsh, etc.) should be maximized in order to optimize water quality performance. Additionally, wetland surface area should also be maximized relative to the contributing drainage area in order to reduce the relative inflow volume. However, as water quality and hydrologic characteristics and requirements may change from site to site, one optimal wetland configuration cannot be defined.

6.15 INPUT PARAMETER UNCERTAINTY AND SENSITIVITY

The second portion of the sensitivity analysis evaluated the sensitivity of model performance criteria to input parameters. All model inputs were assumed to be uncorrelated for the purposes of the sensitivity analysis. In order to assess input parameter importance, upper and lower bounds for each relevant input parameter were identified based their estimated variation at the stormwater wetland site in Charles County, MD. The overall goal of this process was to evaluate the relative importance of each input parameter based on their impact on model outputs. This definition of sensitivity bounds was chosen to best show the impact of input parameter variation on model outputs within the context of a given wetland site. While a number of sensitivity bounds were based on analogous parameter variation at other wetland sites, others were based on input parameter behavior in calibration. Sensitivity bounds for wetland albedo, for example, were defined based on albedo variation trends observed at different wetland sites. Conversely, TSS particle diameter sensitivity bounds were defined based on the parameter's observed behavior in the calibration of the base stormwater wetland design. The following subsections define and explain the selection of the lower and upper bounds used in the sensitivity analysis of each input parameter.

6.15.1 Wetland albedo (a)

The wetland albedo a represents the composite reflectivity of a wetland area. This collective wetland albedo may vary greatly based on season, water depth, snow cover, vegetation height, vegetation cover, and latitude (Goodin et al. 1996; Dingman 2002). Dingman (2002) reported that water albedo is dependent on the solar angle, following an annual cycle resulting in a range of values between 0.05 and 0.60. Snow also has an a value much higher than that of water (Rouse and Bello 1983; LaFleur et al. 1987; Dingman 2002). Despite this annual cycle in water albedo, a number of studies found wetland albedos to stay fairly constant during non-snowy months (Rouse and Bello 1983; Federer et al. 1996). Based on seven study sites spanning in latitudes from Fairbanks, AK, to San Juan, Puerto Rico, Federer et al. (1996) observed that the albedo for different ecosystems (non-forest/tundra, conifer forest, broadleaf forest, savannah, cultivation, and desert) remained fairly constant for temperatures above 0°C, suggesting that snow played the largest role in changes in albedo.

6.15.1.1 Estimated albedo range for stormwater wetland design

A mean annual a of 0.158 was computed from a total of 30 data points collected over the study period of May 15, 1985, to August 15, 1985, reported by LaFleur et al. (1987). These data were collected from a sedge marsh with a mean water depth of about 0.820 ft located near the James Bay in Canada. The albedo was found to increase from 0.11 to 0.19 over the growing season as a result of increased vegetation cover. Over the course of the 1-yr study, respective minimum and maximum values of 0.10 and 0.215 were reported. Based on the computed mean of 0.158, and the minimum and maximum values of 0.10 and 0.215, variation at this site was defined in positive (e/Y^+) and negative (e/Y^-) relative error terms:

$$e/Y^- = \frac{0.10 - 0.158}{0.158} = -0.369 \tag{6-70}$$

$$e/Y^+ = \frac{0.215 - 0.158}{0.158} = 0.357 \tag{6-71}$$

Based on these relative errors, the variation of the albedo values about the mean was fairly even.

A study in the Sandhills region of Nebraska found taller vegetation contributed to lower albedo values by trapping more light than shorter vegetation (Goodin et al. 1996). The same study reported respective albedo values for high standing vegetation (i.e., vegetation height of 3.3-6.6 ft) and low standing vegetation (i.e., vegetation height of 2.5-3.3 ft) wetland areas of 0.152 and 0.178, respectively (Goodin et al. 1996). Goodin et al. (1996) also reported lake water albedo values of 0.073 at the same study site. The variation observed at this site was due more to land cover type. This variation in cover type was also an issue in the stormwater wetland design developed in the current study, which included high-marsh, low-marsh, and deepwater areas. In the stormwater wetland design, high-marsh areas included emergent vegetation rising above water level with heights of 4.9 to 13 ft, low-marsh areas only included submerged vegetation below water surface, and deepwater areas had no vegetation. Therefore, the surface area of the stormwater wetland design consisted of 68% high-marsh area (albedo of 0.152) and 32% low-marsh and deepwater areas (albedo of 0.073) combined. Based on these surface area proportions, a weighted wetland albedo mean of 0.127 was estimated based on the corresponding albedo values reported by Goodin et al. (1996). From this weighted mean positive (e/Y^{+}) and negative (e/Y^{-}) relative error terms were computed:

$$e/Y^{-} = \frac{0.073 - 0.127}{0.127} = -0.425 \qquad (6\text{-}72)$$

$$e/Y^{+} = \frac{0.178 - 0.127}{0.127} = 0.402 \qquad (6\text{-}73)$$

Based on these results, the wetland albedo may vary anywhere from 42.5% below the mean to 40.2% above the mean. These relative errors characterize the possible albedo variation due to different land covers within the wetland. Additional variation, as seen above in the values reported by LaFleur et al. (1987), exits in the albedo input parameter due to seasonal changes.

Given both the seasonal and land cover albedo variation reported in the literature, the current study used error propogation to estimate both negative (E^-) and positive (E^+) relative errors for albedo in the stormwater wetland design:

$$E^- = -\sqrt{(-42.5\%)^2 + (-36.9\%)^2} = -56.3\% \tag{6-74}$$

$$E^+ = \sqrt{(40.2\%)^2 + (35.7\%)^2} = 53.8\% \tag{6-75}$$

Based on these final wetland errors about the base albedo value of 0.159, respective lower and upper albedo values of 0.069 and 0.245. Within the current study the wetland designs incorporating the low (0.069) and high (0.245) albedo input values were referred to as the ALB1 and ALB2 designs.

6.15.1.2 Albedo sensitivity

While wetland albedo did affect wetland ET rates, it was not found to significantly impact any of the PC values relevant to the stormwater wetland design (see Table 6-40). Respective mean annual ET depths for the ALB1 (*a* of 0.069), base (*a* of 0.159), and ALB2 (*a* of 0.245) designs were 34.1, 31.0, and 30.0 in., resulting in a mean ET S_x value of -0.118. Therefore, as mean wetland albedo increases, wetland ET depths decrease. The effect of these ET changes only had a minute (≤0.30%) impact on corresponding wetland outflow depths due to the reallocation of water storage to or from ET in the wetland. This small effect suggested that the wetland inflow and rainfall fluxes were large enough to buffer the effects of the variation in ET caused by changes in the input wetland albedo.

Given these results, it was concluded that model output and corresponding design performance were not significantly impacted by changes in wetland albedo. Therefore, unless ET is of great concern to the user, albedo was not found to be a sensitive input parameter within the estimated range defined for the stormwater wetland. Due to the insensitivity of model PC values to changes in albedo, the final WSI score of 0.640 for the base design remained the same for both the ALB1 and ALB2 designs. These results indicated that the changes made to the wetland albedo in the ALB1 and ALB2 designs did not have an impact on the overall wetland design sustainability as a BMP facility. This insensitivity of design performance was rational despite the effect that albedo had on ET depths because wetland inflow was the dominate water flux controlling outflow volumes and discharge rates.

Table 6-40 Relevant PC values, relative sensitivities, and deviation sensitivities based on input albedo values of 0.159 (base design), 0.069 (ALB1), and 0.245 (ALB2). The relationship direction indicates where the PC value increases (+) or decreases (-) with a corresponding increase in albedo.

Performance Criteria	Base	ALB1	ALB2	S_x	$\lvert D_x \rvert$	Relationship direction
Mean daily TSS Conc. (mg/L)	15.4	15.4	15.4	0.00	0.02	-
Mean daily DO Conc. (mg/L)	10.3	10.3	10.3	0.00180	0.01	-
Mean daily NH4 Conc. (mg/L)	0.0782	0.0784	0.0780	0.0053	0.00	-
Mean daily NO3 Conc. (mg/L)	0.258	0.259	0.258	-0.00478	0.00	-
High-marsh PC	0.962	0.967	0.960	-0.00609	0.00328	-
Low-marsh PC	0.983	0.985	0.982	-0.00248	0.00137	-
High-Flow PC	0.316	0.314	0.317	0.00917	0.00162	+
Low-Flow PC	0.223	0.225	0.222	-0.0100	0.00125	-
Flow-Variation PC	2.90	2.89	2.89	-0.000245	0.00458	+
Flood-Control PC	0.382	0.383	0.381	-0.00352	7.51E-04	-

6.15.2 Leaf area index (LAI)

A base LAI input value of 6.5 was used in the design wetland as determined by literature values (Boyd 1987; Koch and Rawlik 1993; Federer et al. 1996; Xu et al. 2011). A number of factors can play a role in the annual trend of LAI. However, in Charles County, MD (the location of the design stormwater wetland), leaf senescence is the dominant factor. Therefore, the structure of this seasonal LAI trend defined in Equation 4-1 was assumed to sufficiently model wetland vegetation behavior. The LAI quantity during the spring and summer months, however, did contain uncertainty. Therefore, the variation in LAI in the designed stormwater wetland over these months was estimated based on literature values and its importance was evaluated in the sensitivity analysis.

6.15.2.1 Estimated LAI range for stormwater wetland design

Federer et al. (1996) assigned a LAI value of 4 for non-forest wetland/tundra areas in Fairbanks, AK. Another study reported LAI values from 14.1 to 17 (mean of 14.9 and sample size of 12) for the common rush (*Juncus effuses*), a common emergent wetland plant, in Auburn, Alabama (Boyd 1987). This study collected LAI values for common rush from constructed tanks over the growing season (i.e., May through October) of the calendar year 1985 (Boyd 1987). The resulting mean e/Y^-, and e/Y^+ values were computed over all three cells accordingly:

$$e/Y^- = \frac{14-14.9}{14} = -0.054 \qquad (6\text{-}76)$$

$$e/Y^+ = \frac{17-14.9}{14.9} = 0.141 \qquad (6\text{-}77)$$

In this case, variation was greater in the positive direction (+ 14.1% of the mean), while the variation in the negative direction was relatively small with a value of 5.4% below the mean.

Xu et al. (2011) also monitored three study beds within a reed-dominated (species included *Typha latifolia* or cattails) wetland located in North China, which had respective mean LAI values of 2.7, 3.6, and 5.6 based on monthly measurements taken during the growing season (May through September) for the years 2008 and 2009. All monthly LAI values and corresponding mean, e/Y^-, and e/Y^+ values for each of the three study beds are compiled in Table 6-41. From these data, mean e/Y^-, and e/Y^+ of -0.711 and +0.538 were computed:

$$e/Y^- = \frac{-0.743 - 0.662 - 0.730}{3} = -0.711 \qquad (6\text{-}78)$$

$$e/Y^+ = \frac{0.581 + 0.465 + 0.568}{3} = 0.538 \qquad (6\text{-}79)$$

Table 6-41 Monthly reported LAI values for three reed-dominated study beds and corresponding mean, e/Y^-, and e/Y^+ values (Xu et al. 2011).

	Bed I	**Bed II**	**Bed III**
May-08	0.9	1.4	1.7
Jun-08	2.8	3.8	5.2
Jul-08	4.3	4.9	8.2
Aug-08	3.5	4.5	7.3
Sep-08	2.3	3.5	5.5
May-09	0.7	1.2	1.5
Jun-09	2	2.9	4.3
Jul-09	4.1	5.2	8.7
Aug-09	3.7	4.6	7.5
Sep-09	2.9	3.5	5.6
Mean	2.7	3.6	5.6
e/Y^-	-0.743	-0.662	-0.730
e/Y^+	0.581	0.465	0.568

Koch and Rawlik (1993) also reported LAI respective values of 3.94 ± 0.78 and 5.78 ± 0.46 for two 0.25 m^2 plot of *Typha domingenisis* in the Everglades wetland system in Florida. Assuming normal distributions, corresponding minimum and maximum values were estimated for both test plots by subtracting and adding 2σ to each of the plot means in order to estimate the extreme values ± 95.5% about the assumed normal distribution means of 3.94 and 5.78. This range of ±2σ was assumed to represent reasonable maximum and minimum estimates for a given sample. Two standard deviations are commonly used in developing confidence intervals. Resulting minimum and maximum LAI values were 2.38 and 5.50 for the first plot ($2\sigma_1 = 1.56$), and 4.86 and 6.70 for the second plot ($2\sigma_2 = 0.92$). Because the minimum and maximums value were symmetrical about the mean, the resulting relative errors (e/Y_1 and e/Y_2) were also symmetrical for both plots:

$$e/Y_1 = \frac{\pm 1.56}{3.94} = \pm 0.396 \qquad (6\text{-}80)$$

$$e/Y_2 = \frac{\pm 0.92}{5.78} = \pm 0.159 \qquad (6\text{-}81)$$

These computed e/Y_1 and e/Y_2 values were averaged to achieve final mean e/Y value of ±0.278 for both plots.

These reported values represent a wide range of climates and vegetation species; however, all studies consider emergent wetland species, which are used in the model. It was estimated that the design stormwater wetland, which was designed for Charles County, MD, would experience growing season conditions closest to those seen in the study done by Koch and Rawlik (1993), which took place in the

Everglades in Florida. Therefore, a final e/Y value of ± 0.416 was chosen to represent variation at the design stormwater wetland site. Resulting low and high LAI input bounds of 3.8 and 9.2 about the base value of 6.5 were computed from this e/Y and used in the sensitivity analysis of the input parameter LAI. In the following section, the names LAI1 and LAI2 were assigned to the wetland designs with LAI inputs of 3.8 and 9.2.

6.15.2.2 LAI Sensitivity

While changes in input LAI values did significantly impact wetland ET rates, the resulting stormwater wetland performance was not significantly changed as evidenced by the resulting PC values and their corresponding sensitivities (see Table 6-42). As a result of the insensitivity of the model to changes in LAI, the base, LAI1, and LAI2 designs produced the same final WSI score of 0.640. Respective mean annual ET depths resulting from the LAI1, base, and LAI2 designs were 26.1, 31.0, and 40.1 in. These resulting ET depths suggested that wetland ET was sensitive to LAI with a S_x value of -0.221 and increased with increasing LAI, which is expected given that larger LAI values imply larger leaf surface areas available for ET. Very slight changes of 0.073 to 0.13 in. were observed in mean annual wetland outflow depths but were not significant as the base design outflow depth was 15.5 in. Because wetland inflow dominated the wetland water balance, the changes in ET due to different LAI inputs did not have a large impact on wetland outflow.

While the changes made to input LAI values in the LAI1 and LAI2 designs did not significantly affect the wetland performance, they did have a slight impact on the hydrology of the wetland. Decreased ET rates associated with the LAI1 design also resulted in larger wetland water depths as evidenced by the respective high and low-marsh S_x values of 0.0279 and 0.0114. Increased water depths in the LAI1 design also resulted in increased internal wetland velocities and effluent flowrates, and less frequent periods of zero-flows. Conversely, the increased ET in the LAI2 design promoted decreased internal wetland velocities, decreased effluent flowrates, and more frequent zero-flow periods. The respective base, LAI1, and LAI2 low-flow PC values of 0.219, 0.223, and 0.228 reflected this increasing trend in zero-flow periods with increasing LAI values. Interestingly, the high-flow PC values showed an opposite trend with LAI1, base, and LAI2 values of 0.321, 0.316, and 0.310. This trend in high-flow PC values suggested that as LAI values and corresponding ET rates increased, the total duration over which outflow discharge rates exceeded pre-development bankfull flow also increased. While these results seem counterintuitive, they reflect the reduced head over the outlet orifice in the LAI2 design due to decreased storage in the wetland. This reduced head promoted slightly longer outflow durations. Therefore, the decreased head in the LAI2 design resulted in longer durations of flows exceeding the estimated pre-development bankfull discharge rate of 0.366 cfs.

The small changes in wetland outflow did also slightly (S_x values ≤ 0.0205) change wetland water quality performance. Higher LAI input values produced slightly poorer effluent wetland water quality. Within the model, pollutant loads were not removed with evapotranspired water. Therefore, ET had a concentrating effect on pollutant levels in the wetland. This concentrating behavior was assumed to be indicative of real world conditions, suggesting that too much wetland ET may increase pollutant concentrations even though loads remain constant. Therefore, while changes in LAI did not significantly affect the stormwater wetland performance, it is worth noting that the changes in ET observed in the LAI1 and LAI2 design have direct effects on both wetland hydrology and water quality.

Table 6-42 Relevant PC values, relative sensitivities, and deviation sensitivities of the base, LAI1 and LAI2 designs. The relationship direction indicates where the PC value increases (+) or decreases (-) with a corresponding increase in LAI.

Performance Criteria	Base	LAI1	LAI2	S_x	$\lvert D_x \rvert$	Relationship direction
Mean daily TSS Conc. (mg/L)	15.4	15.3	15.4	0.0109	0.0699	+
Mean daily DO Conc. (mg/L)	10.3	10.3	10.3	0.00591	0.0252	+
Mean daily NH4 Conc. (mg/L)	0.0782	0.0776	0.0789	0.0202	0.000657	+
Mean daily NO3 Conc. (mg/L)	0.258	0.256	0.260	0.0205	0.00220	+
High-marsh PC	0.962	0.954	0.977	0.0279	0.01115	+
Low-marsh PC	0.983	0.980	0.989	0.0114	0.00464	+
High-Flow PC	0.316	0.321	0.310	0.0423	0.00556	
Low-Flow PC	0.223	0.219	0.228	0.0464	0.00430	+
Flow Variation PC	2.90	2.90	2.88	-0.0108	0.0130	-
Flood Control PC	0.382	0.380	0.385	0.0162	0.00257	+

6.15.3 Shelter factor (f_s)

6.15.3.1 Estimated shelter factor range for stormwater wetland design

Shelter factor is a fractional measure with values between 0.5 and 1 of the degree to which vegetation shades itself, with a value of 0.5 indicating that 50% of the vegetation is unshaded while a value of 1 indicates that 100% of the vegetation is unshaded. Wetland vegetation f_s values were not found in the literature. However, the f_s was assumed to vary based on emergent vegetation density and was defined to have a required range of 0.5 to 1 (Dingman 2002). More dense vegetation should have a lower f_s (closer to 0.5) as the shading effect of more dense vegetation is greater than more sparsely populated vegetation. Conversely, a f_s value of 1 implies that shading effects do not occur. The base f_s input value for the design stormwater wetland was 0.75 as a mean value. Given the lack of data on f_s, the current study estimated a sensitivity bound of 0.525 to 0.975, which was based on an estimated symmetric relative error of ±30%. This range of f_s was assumed to be reasonable as vegetation density may vary between years or even within a season due to vegetation growth and death, wetland maintenance, and changes in the distributions of vegetation species within the wetland. The wetland designs incorporating the low (0.525) and high (0.975) f_s were referred to as designs FS1 and FS2.

6.15.3.2 Shelter factor sensitivity

The FS1 and FS2 designs produced similar results as those observed in the LAI1 and LAI2 designs. Input FS1, base, and FS2 designs resulted in respective simulated mean annual ET depths of 26.7, 31.0, and 38.5 in. These ET values also produced a S_x value of -0.361, suggesting that ET was inversely related to f_s. Therefore, as f_s decreased and a greater proportion of vegetation within the wetland was shaded, resulting wetland ET depths also decreased. While ET depths were affected by changes in f_s values, overall wetland performance was not sensitive to these changes as evidenced by the resulting FS1 and FS2 PC values and sensitivities shown in Table 6-43. Additionally, the resulting WSI score for the base, FS1, and FS2 designs was 0.640, revealing that the associated changes in f_s had not impact on the overall sustainability of the wetland design as a BMP facility.

Despite the relative insensitivity to PC values to changes in f_s, similar hydrologic and water quality trends as those observed in the LAI1 and LAI2 designs were observed in the FS1 and FS2 designs. The increased ET rates associated with the larger f_s value in the FS2 design promoted shallower water depths, slower internal velocities, and lower discharge rates and extended duration of low flows. Conversely, the decreased ET rates in the FS1 design promoted larger water depths, faster internal velocities, and higher discharge rates and shorter duration of low flows. Additionally, as observed in the LAI1 and LAI2 designs, effluent water quality improved slightly (S_x values ≤ 0.0242) with decreasing ET rates associated with smaller f_s values due to the concentrating effect of increased ET on wetland pollutant levels.

Table 6-43 Relevant PC values, relative sensitivities, and deviation sensitivities based on input f_s values of 0.75 (base design), 0.525 (FS1), and 0.975 (FS2). The relationship direction indicates where the PC value increases (+) or decreases (-) with a corresponding increase in f_s.

Performance Criteria	Base	FS1	FS2	S_x	$\lvert D_x \rvert$	Relationship direction
Mean daily TSS Conc. (mg/L)	15.4	15.3	15.4	0.0135	0.0625	+
Mean daily DO Conc. (mg/L)	10.3	10.3	10.3	0.00715	0.0220	+
Mean daily NH4 Conc. (mg/L)	0.0782	0.0776	0.0788	0.0242	0.000567	+
Mean daily NO3 Conc. (mg/L)	0.258	0.256	0.260	0.0233	0.00180	+
High-marsh PC	0.962	0.955	0.974	0.0323	0.00932	+
Low-marsh PC	0.983	0.980	0.988	0.0132	0.00388	+
High-Flow PC	0.316	0.320	0.311	0.0473	0.00449	-
Low-Flow PC	0.223	0.220	0.227	0.0520	0.00348	+
Flow Variation PC	2.90	2.90	2.88	-0.0127	0.0110	-
Flood Control PC	0.382	0.380	0.384	0.0188	0.00215	+

6.15.4 Maximum leaf conductance (C^*_{leaf})

6.15.4.1 Estimated C^*_{leaf} range in the stormwater wetland

The maximum leaf conductance C^*_{leaf}, which is also referred to as maximum stomatal conductance, represents the maximum rate (mm/s) at which the leaves of a given plant will transfer water into the surrounding atmosphere. This maximum rate occurs with the leaf pores or stomata are completely opened. Different vegetation species can have different C^*_{leaf} values due to different leaf areas, stomatal densities within each leaf, and stomatal opening size. While a number of studies in the literature report leaf conductance values, not many studies report maximum leaf conductance values. Federer et al. (1996) reported a C^*_{leaf} of 6.6 mm/s for tundra/non-forest wetland ecosystems in Fairbanks, AK. This value was the only explicitly defined value of C^*_{leaf} found in the literature. All other literature values used represent the maximum leaf conductance values reported from studies done on seasonal leaf conductances.

Koch and Rawlik (1993) reported stomatal conductances for *Typha domingensis* plant species of 10.5 ± 0.9 mm/s (n = 10) in a study plot the Everglades. In a separate study, *Carex aquatilis*-dominated wetlands in Fairbanks, AK, produced leaf conductance values that ranged from 1.7 to 7 mm/s and corresponding mean values between 3 to 5 mm/s during summer months with air temperatures ranging from 14 to 18.9°C (Morrissey et al. 1993). Tank studies with *Typha latifolia* species in Phoenix, Arizona, reported leaf conductances of 0 to 12 mm/s.

A C^*_{leaf} of 12.3 mm/s was estimated from the data provided by Koch and Rawlik (1993) assuming conductance values were normally distributed and that the maximum value was three standard deviations greater than the reported mean, where $\pm 2\sigma$ of the mean represents 95.5% of the assumed normal distribution. Therefore, the maximum values estimated from the literature were 12.3, 7, 12, and 6.6 mm/s. The lower reported C^*_{leaf} values of 6.6 and 7 mm/s were observed in the colder climate of Alaska tundra/sedge marshes, while higher values of 12.3 and 12 mm/s were reported in the hotter climates of Arizona and Florida. In order to estimate C^*_{leaf} at a given site, the cold climate values of 6.6 and 7 mm/s were assumed to represent a collective cold site with a mean of 6.8 mm/s while the warm climate values were assumed to represent a collective warm site with a mean of 12.2 mm/s. These estimates resulted in symmetrical cold and warm climate relative errors of ± 1.2% and ± 4.7%.

The actual C^*_{leaf} for the Charles County, MD, which was site of designed stormwater wetland area was thought to be in between the hot and cold extremes represented in the literature with an input value of 9.7 mm/s for the base C^*_{leaf} value. The associated variation for this wetland was estimated to equal the mean of the cold and warm relative errors, resulting in a value of ± 3.0% and low and high C^*_{leaf} bounds of 9.41 and 10.0 mm/s. Wetland designs CM1 and CM2 were used to simulate wetland designs with respective C^*_{leaf} values of 9.41 and 10.0 mm/s.

6.15.4.2 Maximum leaf conductance sensitivity

ET rates were sensitive to the changes made to C^*_{leaf} input values in the CM1 and CM2 designs as evidenced by the resulting ET relative sensitivity S_x of 0.654. However, due to the small C^*_{leaf} range of 9.41 to 10.0 mm/s used for the stormwater design in Section 6.15.4.1, this sensitivity was not reflected in model output values. The resulting mean annual ET depths that correspond to the CM1, base, and CM2 designs were 30.5, 31.0 and 31.7 in. This increasing trend is rational as higher C^*_{leaf} values imply a greater capacity of the wetland leaves to promote ET. Despite, the strong sensitivity of ET rates to C^*_{leaf}, the small changes in C^*_{leaf} in the CM1 and CM2 designs did not have significant impacts on wetland performance. Resulting CM1 and CM2 PC values and WSI scores were not notably differ from those of the base design (see Table 6-44). Therefore, due to the small range of C^*_{leaf} estimated for the stormwater wetland, changes made to C^*_{leaf} in the CM1 and CM2 did not have a significant impact on wetland hydrology nor water quality.

Table 6-44 Relevant PC values, relative sensitivities, and deviation sensitivities based on input C^*_{leaf} values of 9.7 (base design), 9.41(CM1), and 10.0 mm/s (CM2). The relationship direction indicates where the PC value increases (+) or decreases (-) with a corresponding increase in C^*_{leaf}.

Performance Criteria	Base	CM1	CM2	S_x	$\lvert D_x \rvert$	Relationship direction
Mean daily TSS Conc. (mg/L)	15.4	15.4	15.4	0.0189	0.00883	-
Mean daily DO Conc. (mg/L)	10.3	10.3	10.3	0.0171	0.00535	+
Mean daily NH4 Conc. (mg/L)	0.0782	0.0781	0.0782	0.0185	0.0000441	+
Mean daily NO3 Conc. (mg/L)	0.258	0.258	0.258	0.0166	0.000131	+
High-marsh PC	0.962	0.961	0.963	0.0332	0.000972	+
Low-marsh PC	0.983	0.983	0.983	0.0135	0.000406	+
High-Flow PC	0.316	0.317	0.316	0.0634	0.000609	-
Low-Flow PC	0.223	0.222	0.223	0.0634	0.000429	+
Flow Variation PC	2.90	2.90	2.90	-0.00264	0.000230	-
Flood Control PC	0.382	0.381	0.382	0.0194	2.26E-04	+

6.15.5 Emergent vegetation height above water (z_v)

6.15.5.1 Estimated emergent vegetation range in stormwater wetland

In order to estimate the variation in emergent vegetation height above the water, the emergent species *Typha* spp. (common name, cattails) was chosen to represent the emergent vegetation in the design wetland. Cattail species are successful in and commonly used in constructed wetlands (Kadlec and Knight 1996; EPA 2000) and require water depths of 0.1 to 0.75 m (0.30 to 2.5 ft) and can stand from 1.5 to 4 m (4.9 to 13 ft) tall (Kadlec and Knight 1996; Peron 2002; USDA 2006). Given that water depths with emergent vegetation in the example wetland design range from 0.5 to 1 ft, a z_v range of 3.9 to 12.5 ft (1.18 to 3.81 m) was estimated. However, because the model limited input z_v values to be less than or equal to the corresponding input z_m in order to avoid irrational outputs, the z_v upper limit was restricted to 2 m rather than 3.18 m. Therefore, the sensitivity range used to evaluate the importance of z_v was 1.18 to 2 m about the base value of 1.65 m. The wetland designs incorporating input z_v values of 1.18 and 2 m were respectively referred to at designs ZV1 and ZV2.

6.15.5.2 Emergent vegetation height sensitivity

The changes made to z_v in the ZV1 and ZV2 slightly impacted mean annual ET depths with a mean S_x of 0.00124. This insignificant increase in ET with increasing z_v reflects the larger leaf area available for ET when emergent vegetation is taller. Increasing z_v also decreased effluent DO concentrations slightly as evidenced by the corresponding mean S_x of -0.0414. This decreasing trend in DO was due to the increased water temperatures produced by higher conductive-convective heat flux Q_H values (see Equation 4-39) that resulted from larger z_v values. In order to better illustrate this trend, the resulting mean internal water temperatures were computed for the ZV1, base, and ZV2 designs and were found to respectively equal 12.4, 12.7, and 13.0°C. Despite these effects, significant changes in model outputs were not observed in the ZV1 and ZV2 designs, which indicated that model performance was not sensitive to changes in z_v.

Table 6-45 Relevant PC values, relative sensitivities, and deviation sensitivities based on input z_v values of 1.65 m (base design), 1.18 m (ZV1), and 3.81 m (ZV2). The relationship direction indicates where the PC value increases (+) or decreases (-) with a corresponding increase in z_v.

Performance Criteria	Base	ZV1	ZV2	S_x	$\lvert D_x \rvert$	Relationship direction
Mean daily TSS Conc. (mg/L)	15.4	15.4	15.4	0.000244	0.00114	+
Mean daily DO Conc. (mg/L)	10.3	10.5	10.1	-0.0414	0.204	-
Mean daily NH4 Conc. (mg/L)	0.0782	0.0789	0.0774	-0.0195	0.000722	-
Mean daily NO3 Conc. (mg/L)	0.258	0.259	0.257	-0.00658	0.000816	-
High-marsh PC	0.962	0.962	0.962	7.69×10^{-5}	3.56×10^{-5}	+
Low-marsh PC	0.983	0.983	0.983	3.13×10^{-5}	1.48×10^{-5}	+
High-Flow PC	0.316	0.316	0.316	0.000629	6.96×10^{-5}	-
Low-Flow PC	0.223	0.223	0.223	0.000629	4.90×10^{-5}	+
Flow Variation PC	2.90	2.90	2.90	-0.0013	0.00140	-
Flood Control PC	0.382	0.382	0.382	5.98×10^{-5}	1.05×10^{-5}	+

6.15.6 Influent TSS particle diameter (D)

Influent TSS particle diameter (*D*) variation within any given site depends on a number of factors including wetland influent water type (municipal wastewater, agricultural wastewater, urban stormwater, etc.), upstream soil types and land use, season, pretreatment processes, rainfall intensity/duration, droughts and floods, and construction within the drainage area (Pathak et al. 2004; Rinker Materials 2004; DeGroot 2008). Influent TSS particle diameter and its associated variation are, therefore, very difficult to characterize as a result the large number of factors influencing their values. The initial estimated range of *D* values used in calibration of the base stormwater wetland was 1.0×10^{-8} to 6.5×10^{-5}m as estimated by urban runoff values reported by USEPA (1983). A base *D* value of 1.2×10^{-6} m was estimated via calibration in Section 6.8.

6.15.6.1 Estimated TSS Particle diameter range in stormwater wetland

Because reports of stormwater runoff TSS particle size data were lacking in the literature, the sensitivity of *D* in the initial stormwater wetland calibration process was used to estimate high and low bounds for the sensitivity analyses. An input *D* value of 9.5×10^{-6} m resulted in a mean daily effluent concentration 3 mg/L, which was the irreducible TSS background concentration in the wetland, suggesting that nearly all influent TSS particles settled out. An input *D* value of 1.0×10^{-6} m resulted in a daily effluent TSS concentration of 19.1 mg/L, which was slightly more than the target effluent concentration of 15.2 mg/L. Given these model responses to changes in *D*, it was apparent that *D* was a very important input parameter and that even small changes in it could affect wetland effluent values greatly. Given this model sensitivity to *D*, high and low bounds of 5.5×10^{-6} and 7.5×10^{-7}m were defined with the goal of showing output sensitivity to *D* as it was estimated to vary at the design stormwater wetland site. Wetland designs incorporating the low (7.5×10^{-7}m) and high (5.5×10^{-6}) *D* values were respectively defined as designs D1 and D2.

6.15.6.2 TSS particle diameter sensitivity

As shown in Table 6-46, the changes made to the input TSS particle diameter significantly impacted mean daily effluent TSS concentrations with a S_x value of 0.894, but did not change any other wetland output parameters. Increasing the particle diameter of influent TSS in the D2 design simulated faster TSS settling velocities within the model, which resulted in a higher wetland trap efficiency and a lower mean daily effluent TSS concentration of 3.4 mg/L as compared with the base value of 15.4 mg/L. Similarly, the reduced TSS particle diameter in the D1 design decreased TSS settling velocities and decreased the wetland trap efficiency as evidenced by the high D1 mean daily effluent TSS concentration of 24.5 mg/L. Because the TSS particle diameter was only related to TSS settling velocities, its alteration did not affect any other wetland outputs. The changes in effluent TSS concentrations resulted in respective D1 and D2 WSI scores of 0.630 and 0.707. Therefore, despite the fact that *D* only affected TSS concentrations, it did impact the overall wetland sustainability (see Table 6-47), which indicates that within the BMP-weighting scheme defined in Section 6.9.1.8, effluent TSS concentrations are very important to wetland performance. This sensitivity of WSI to effluent TSS concentrations is, however, subject to change depending on the goals of the model user.

Based on these results, effluent TSS concentrations were found to be very sensitive to the input *D* value, which suggests that the user should take care to estimate it as accurately as possible. Unfortunately, *D* is also very difficult to define given that TSS particle size depends on a number of factors and can change from storm to storm depending on rainfall intensity, antecedent dry time, drainage area activity, etc. While more data are necessary to better characterize *D*, the current study suggested that expected wetland TSS trap efficiency and calibration be used to best estimate *D* in a base design before evaluating the effects of different design changes.

Table 6-46 Relevant PC values, relative sensitivities, and deviation sensitivities based on input *D* values of 1.2×10^{-6} m (base design), 7.5×10^{-7} m (D1), and 5.5×10^{-6} m (D2). The relationship direction indicates where the PC value increases (+) or decreases (-) with a corresponding increase in *D*.

Performance Criteria	Base	D1	D2	S_x	$\lvert D_x \rvert$	Relationship direction
Mean daily TSS Conc. (mg/L)	15.4	24.5	3.4	0.894	10.50	-
Mean daily DO Conc. (mg/L)	10.3	10.3	10.3	0.00	0.00	-
Mean daily NH4 Conc. (mg/L)	0.0782	0.0782	0.0782	0.00	0.00	-
Mean daily NO3 Conc. (mg/L)	0.258	0.258	0.258	0.00	0.00	-
High-marsh PC	0.962	0.962	0.962	0.00	0.00	-
Low-marsh PC	0.983	0.983	0.983	0.00	0.00	-
High-Flow PC	0.316	0.316	0.316	0.00	0.00	+
Low-Flow PC	0.223	0.223	0.223	0.00	0.00	-
Flow-Variation PC	2.90	2.90	2.90	0.00	0.00	+
Flood-Control PC	0.382	0.382	0.382	0.00	0.00	-

Table 6-47 Resulting normalized metrics and final wetland sustainability indices (WSI's) for the base, D1 and D2 stormwater wetland designs. All WSI scores were computed assuming equal weights for all water quality and hydrologic metrics.

Performance Criteria	Metric weights	Base design metrics	D1 metrics	D2 metrics
Mean daily TSS Conc. (mg/L)	0.1	0.328	0.225	1.00
Mean daily DO Conc. (mg/L)	0.1	1.00	1.00	1.00
Mean daily NH4 Conc. (mg/L)	0.1	0.528	0.528	0.528
Mean daily NO3 Conc. (mg/L)	0.1	1.00	1.00	1.00
High-marsh PC	0.1	0.999	0.999	0.999
Low-marsh PC	0.1	1.00	1.00	1.00
High-Flow PC	0.1	0.533	0.533	0.533
Low-Flow PC	0.1	0.396	0.396	0.396
Flow Variation-PC	0.1	0.00	0.00	0.00
Flood Control-PC	0.1	0.618	0.618	0.618
Final *WSI* score	---	0.640	0.630	0.707

6.15.7 Maximum photosynthesis rate (PMAX)

6.15.7.1 Estimated PMAX range in stormwater wetland

As estimated in Section 4.4.2.3.1, $PMAX$ in the base stormwater wetland design was set to equal 910 mg/m^2-hr, which was the estimated mean literature value for annual wetland oxygen production of 1710 g-O_2/m^2-yr. Values of the annual photosynthesis varied greatly within the literature (Mitsch and Gosselink 1993; USEPA 2000; Tian et al. 2010). $PMAX$ variation at the design site within the current study was estimated based on values reported by Mitsch and Gosselink (1993) from a study done by Bernard and Solsky (1977) for a sedge meadow with *Larex lacrustris* in New York. The reported range of net primary productivity for this site was 1,078 to 1,741 g total biomass/m^2-yr. As stated in Section 4.4.2.3.1, USEPA (2000) estimated that about 1 g of O_2 is produced for each gram of biomass produced. Therefore, the corresponding photosynthesis rate range would also be 1,078 to 1,741 g O_2/m^2-yr with a mean of 1,410 g O_2/m^2-yr and symmetric relative error of ±23.5%. Based on this estimated relative error, the resulting high and low $PMAX$ bounds were set equal to 1124 and 696 g O_2/m^2-yr. Resulting wetland designs with respective $PMAX$ inputs of 696 and 1124 g O_2/m^2-yr were referred to as designs PMX1 and PMX2.

6.15.7.2 PMAX sensitivity

Increasing $PMAX$ slightly increased mean daily effluent DO concentrations as shown by the small mean DO S_x value of 0.0162. All other wetland outputs were negligibly affected by the PMX1 and PMX2 designs. The DO levels were not significantly impacted by changes in $PMAX$ because even the DO levels produced by the low *PMAX* in the PMX1 design promoted DO levels near saturation. Additionally, surface aeration accounted for a significant amount of wetland DO in the shallower wetland cells. The model restricted DO levels to be less than or equal to saturated levels. Therefore, because the stormwater design promoted these maximum saturated levels, DO levels remained constantly at the water saturation point. Despite this insensitivity, *PMAX* may play a larger role in wetland designs in which surface aeration is not a large contributor to DO as well as those that incorporate fewer areas with photosynthesizing submerged vegetation.

Table 6-48 Relevant PC values, relative sensitivities, and deviation sensitivities of the base, PMX1, and PMX2 designs. The relationship direction indicates where the PC value increases (+) or decreases (-) with a corresponding increase in *PMAX*.

Performance Criteria	Base	PMX1	PMX2	S_x	$\lvert D_x \rvert$	Relationship direction
Mean daily TSS Conc. (mg/L)	15.4	15.4	15.4	0.00	0.00	-
Mean daily DO Conc. (mg/L)	10.3	10.2	10.3	0.0162	0.0391	+
Mean daily NH4 Conc. (mg/L)	0.0782	0.0782	0.0782	0.00	0.00	-
Mean daily NO3 Conc. (mg/L)	0.258	0.258	0.258	0.00	0.00	-
High-marsh PC	0.962	0.962	0.962	0.00	0.00	-
Low-marsh PC	0.983	0.983	0.983	0.00	0.00	-
High-Flow PC	0.316	0.316	0.316	0.00	0.00	+
Low-Flow PC	0.223	0.223	0.223	0.00	0.00	-
Flow-Variation PC	2.90	2.90	2.90	0.00	0.00	+
Flood-Control PC	0.382	0.382	0.382	0.00	0.00	-

6.15.8 Influent TSS concentration (TSS_{in})

6.15.8.1 Influent TSS concentration range in stormwater wetland

The base design TSS_{in} value was 43.2 mg/L and was based on the average influent TSS event mean concentration (EMC) for a total of 10 stormwater wetlands in the mid-Atlantic region reported by Leisenring et al. (2012). In order to estimate the variation in this influent concentration at the design wetland site, the variation in influent TSS concentrations was characterized for three BMP sites treating stormwater from residential drainage areas (www.bmpdatabase.org). All three site, their median, associated number of samples, 25th percentile, 75th percentile, and their computed e/Y^-, and e/Y^+ values are shown in Table 6-49. Computed e/Y^-, and e/Y^+ values were determined for each site by using the 25th percentile EMC as the minimum and the 75th percentile EMC as the maximum influent EMC. The resulting e/Y^-, and e/Y^+ values for each site were averaged to determine final positive and negative relative errors of +327% and -45.1%. Therefore, the corresponding TSS_{in} bounds were estimated to equal 23.7 and 184 mg/L. Sensitivity analysis designs TSS1 and TSS2 were defined as those incorporating influent TSS_{in} concentrations of 23.7 and 184 mg/L.

The www.bmpdatabase.org data represent EMC's, TSS_{in} values are input to the model on a 1-min increment. While significant error is associated with this application of EMC values on a 1-min increment, sufficient data was not available to estimate variation at a 1-min time step. Therefore, the EMC values reported by www.bmpdatabase.org were assumed to be the best estimate of water quality (TSS, NH_4^-, and NO_3^+) influent concentration variation. This same assumption was made with respect to the base influent water quality concentrations in Section 6.8.

Table 6-49 Residential stormwater runoff TSS statistics for three different BMP sites as reported by the BMP database (www.bmpdatabase.org). EMC is event mean concentration.

	Influent TSS characteristics					
	# Samples	Median EMC (mg/L)	25th Percentile EMC (mg/L)	75th Percentile EMC (mg/L)	e/Y^-	e/Y^+
May's Chapel Wetland Basin (Baltimore, MD)	27	50.5	33.3	89.5	-0.342	0.772
Queen Anne's Pond (Centerville, MD)	39	25.1	14.7	52.5	-0.413	1.09
Club Run Bioretention Cell (Chantilly, VA)	10	16.8	6.74	151	-0.600	7.94
Mean	---	30.8	18.2	97.5	-0.451	3.27

6.15.8.2 Influent TSS concentration sensitivity

Mean daily effluent TSS concentrations were observed to increase with increasing TSS_{in} as demonstrated by the resulting mean TSS S_x value of 0.970. These changes did not, however, affect any other model outputs (see Table 6-50) because the TSS portion of the model did not affect the computation of other portions of the model. The daily mean effluent TSS concentrations for the TSS1, base, and TSS2 designs were respectively 8.7, 15.4, and 64.5 mg/L. These large discrepancies in effluent TSS concentrations suggested that wetland TSS performance was directly related to influent TSS concentrations within the model. Final WSI scores for the TSS1, base, and TSS2 designs were respectively 0.659, 0.640, and 0.618 (see Table 6-52), which indicated that TSS_{in} was an important input parameter with respect to the overall wetland performance as a BMP facility.

Table 6-50 Relevant PC values, relative sensitivities, and deviation sensitivities based on input TSS_{in} values of 43.2 mg/L (base design), 23.7 mg/L (TSS1), and 184 mg/L (TSS2). The relationship direction indicates where the PC value increases (+) or decreases (-) with a corresponding increase in TSS_{in}.

Performance Criteria	Base	TSS1	TSS2	S_x	$\|D_x\|$	Relationship direction
Mean daily TSS Conc. (mg/L)	15.4	8.7	64.5	0.970	27.9	+
Mean daily DO Conc. (mg/L)	10.3	10.3	10.3	0.00	0.00	-
Mean daily NH4 Conc. (mg/L)	0.0782	0.0782	0.0782	0.00	0.00	-
Mean daily NO3 Conc. (mg/L)	0.258	0.258	0.258	0.00	0.00	-
High-marsh PC	0.962	0.962	0.962	0.00	0.00	-
Low-marsh PC	0.983	0.983	0.983	0.00	0.00	-
High-Flow PC	0.316	0.316	0.316	0.00	0.00	
Low-Flow PC	0.223	0.223	0.223	0.00	0.00	-
Flow-Variation PC	2.90	2.90	2.90	0.00	0.00	+
Flood-Control PC	0.382	0.382	0.382	0.00	0.00	-

Table 6-51 Resulting normalized metrics and final wetland sustainability indices (WSI's) for the base, TSS1 and TSS2 stormwater wetland designs. All WSI scores were computed assuming equal weights for all water quality and hydrologic metrics.

Performance Criteria	Metric weights	Base design metrics	TSS1 metrics	TSS2 metrics
Mean daily TSS Conc. (mg/L)	0.1	0.328	0.519	0.103
Mean daily DO Conc. (mg/L)	0.1	1.00	1.00	1.00
Mean daily NH4 Conc. (mg/L)	0.1	0.528	0.528	0.528
Mean daily NO3 Conc. (mg/L)	0.1	1.00	1.00	1.00
High-marsh PC	0.1	0.999	0.999	0.999
Low-marsh PC	0.1	1.00	1.00	1.00
High-Flow PC	0.1	0.533	0.533	0.533
Low-Flow PC	0.1	0.396	0.396	0.396
Flow-Variation PC	0.1	0.00	0.00	0.00
Flood-Control PC	0.1	0.618	0.618	0.618
Final *WSI* score	---	0.640	0.659	0.618

6.15.9 Influent NH_4^+ concentration ($NH4_{in}$)

6.15.9.1 Influent NH_4^+ concentration range in stormwater wetland

Mean daily effluent NH_4^+ concentrations increased with increasing $NH4_{in}$. The base design $NH4_{in}$ value was 0.13 mg/L and was based on the average influent NH_4^+ EMC for a total of 10 stormwater wetlands in the mid-Atlantic region reported by Leisenring et al. (2012). In order to estimate a reasonable range for $NH4_{in}$ in the sensitivity analysis, data for influent NH_4^+ event mean concentrations was used from three residential BMP sites (see Table 6-52). The resulting mean e/Y^- and e/Y^+ applied to the example wetland design were -0.29 and +1.11 with corresponding $NH4_{in}$ concentrations of 0.0908 and 0.274 mg/L. These lower and upper $NH4_{in}$ values were incorporated into the designs AM1 and AM2 for sensitivity analysis.

Table 6-52 Residential stormwater runoff NH_4^+ statistics for three different BMP sites as reported by the BMP database (www.bmpdatabase.org). EMC is event mean concentration.

	Influent NH_4^+ characteristics					
	# Samples	Median EMC (mg/L)	25th Percentile EMC (mg/L)	75th Percentile EMC (mg/L)	e/Y^-	e/Y^+
May's Chapel Wetland Basin (Baltimore, MD)	23	0.14	0.12	0.28	-0.14	1.00
Queen Anne's Pond (Centerville, MD)	39	0.11	0.08	0.27	-0.27	1.45
Club Run Bioretention Cell (Chantilly, VA)	10	0.43	0.23	0.8	-0.47	0.86
Mean	---	0.23	0.14	0.45	-0.29	1.11

6.15.9.2 Influent NH_4^+ concentration sensitivity

While the mean daily effluent NH_4^+ concentration was most affected by the changes made to $NH4_{in}$ in the AM1 and AM2 designs (mean S_x of 1.10), mean daily effluent DO and NO_3^- concentrations were also affected with respective mean S_x values of 0.00348 and 0.117 (see Table 6-53). Mean daily effluent NH_4^+ concentrations for the AM1, base, and AM2 were found to equal 0.0552, 0.0782, and 0.1647. This increasing trend indicated that effluent NH_4^+ concentrations were directly related to corresponding influent NH_4^+ concentrations. Increased $NH4_{in}$ values promoted larger internal NH_4^+ concentrations, which increased the nitrification driving force or the difference between the NH_4^+ concentration for a given cell and the user-defined background NH_4^+ concentration. As a result of this increase in the nitrification driving force, greater nitrification occurred within the AM2 design than in the base design. This increased driving force, however, was not sufficient to counteract the increased $NH4_{in}$ in the AM2 design as evidenced by the elevated effluent NH_4^+ concentrations in the AM2 design.

Because nitrification rates increased slightly with increasing $NH4_{in}$ values, both DO demand and NO_3^- production also increased with $NH4_{in}$. DO levels decreased minutely, producing a mean S_x of 0.00348, with the increased $NH4_{in}$ in the AM2 design due to the corresponding increase in nitrification oxygen demand. However, because the wetland efficiently generated DO, achieving near saturation DO levels, this increased oxygen demand did not have a large effect on the wetland DO concentrations. Mean daily effluent NO_3^- concentrations were also observed to increase significantly (mean S_x of 0.117) with increasing $NH4_{in}$ values due to the corresponding increased nitrification within the wetland.

Based on the results from the AM1 and AM2 wetland designs, the current study concluded that the $NH4_{in}$ input parameter was very important with respect to effluent NH_4^+ concentrations and also had minor impacts on effluent DO and NO_3^- concentrations. As shown in Table 6-53, final AM1, base, and AM2 design WSI scores were computed to be 0.654, 0.640, and 0.610. The variation in the final WSI scores was due solely to the changes in mean daily effluent NH_4^+ concentrations as the changes in effluent DO and NO_3^- concentrations were not sufficient to change the corresponding metric values. Therefore, within the context of the stormwater design developed in the current study, $NH4_{in}$ primarily affected effluent NH_4^+ concentrations. However, in a wetland design with lower DO levels and/or less denitrification, $NH4_{in}$ could have a greater impact on effluent DO and NO_3^- concentrations. The significance of the differences observed in the WSI scores for the base, AM1, and AM2 designs would depend on the sensitivity of downstream ecosystems to such changes. While many species may not be sensitive to such changes in NH_4^+ concentrations, others may require a very narrow range of NH_4^+ concentrations.

Table 6-53 Relevant PC values, relative sensitivities, and deviation sensitivities based on input $NH4_{in}$ values of 0.13 mg/L (base design), 0.0908 mg/L (AM1), and 0.274 mg/L (AM2). The relationship direction indicates where the PC value increases (+) or decreases (-) with a corresponding increase in $NH4_{in}$.

Performance Criteria	Base	AM1	AM2	S_x	$\lvert D_x \rvert$	Relationship direction
Mean daily TSS Conc. (mg/L)	15.4	15.4	15.4	0.00	0.00	-
Mean daily DO Conc. (mg/L)	10.3	10.3	10.2	0.00348	0.0234	-
Mean daily NH4 Conc. (mg/L)	0.0782	0.0552	0.1647	1.10	0.0548	+
Mean daily NO3 Conc. (mg/L)	0.258	0.250	0.289	0.117	0.0193	+
High-marsh PC	0.962	0.962	0.962	0.00	0.00	-
Low-marsh PC	0.983	0.983	0.983	0.00	0.00	-
High-Flow PC	0.316	0.316	0.316	0.00	0.00	+
Low-Flow PC	0.223	0.223	0.223	0.00	0.00	-
Flow-Variation PC	2.90	2.90	2.90	0.00	0.00	+
Flood-Control PC	0.382	0.382	0.382	0.00	0.00	-

Table 6-54 Resulting normalized metrics and final wetland sustainability indices (WSI's) for the base, AM1 and AM2 stormwater wetland designs. All WSI scores were computed assuming equal weights for all water quality and hydrologic metrics.

Performance Criteria	Metric weights	Base design metrics	AM1 metrics	AM2 metrics
Mean daily TSS Conc. (mg/L)	0.1	0.328	0.328	0.328
Mean daily DO Conc. (mg/L)	0.1	1.00	1.00	1.00
Mean daily NH4 Conc. (mg/L)	0.1	0.528	0.664	0.223
Mean daily NO3 Conc. (mg/L)	0.1	1.00	1.00	1.00
High-marsh PC	0.1	0.999	0.999	0.999
Low-marsh PC	0.1	1.00	1.00	1.00
High-Flow PC	0.1	0.533	0.533	0.533
Low-Flow PC	0.1	0.396	0.396	0.396
Flow-Variation PC	0.1	0.00	0.00	0.00
Flood-Control PC	0.1	0.618	0.618	0.618
Final *WSI* score	---	0.640	0.654	0.610

6.15.10 Influent NO_3^- concentration ($NO3_{in}$)

6.15.10.1 Influent NO_3^- concentration range in stormwater wetland

The base influent NO_3^- concentration ($NO3_{in}$) was equal to 0.50 mg/L based on Leisenring et al. (2012) reported values. Variation in $NO3_{in}$ was estimated from the reported influent NO_3^- event mean concentration median, and the 25th and 75th percentile concentrations of three BMP sites in the mid-Atlantic region (see Table 6-55). Mean e/Y^- and e/Y^+ values of -0.382 and 0.652 were computed from these sites, resulting in a $NO3_{in}$ range of 0.316 and 0.705 mg/L for the example stormwater wetland designed in the current study. Stormwater wetland designs developed with respective $NO3_{in}$ inputs of 0.316 and 0.705 mg/L were referred to as NIT1 and NIT2 in the following section.

Table 6-55 Residential stormwater runoff NO_3^- statistics for three different BMP sites as reported by the BMP database (www.bmpdatabase.org). EMC is event mean concentration.

	Influent NO_3^- characteristics					
	# Samples	Median EMC (mg/L)	25th Percentile EMC (mg/L)	75th Percentile EMC (mg/L)	e/Y^-	e/Y^+
May's Chapel Wetland Basin (Baltimore, MD)	22	1.12	0.65	1.35	-0.42	0.21
Queen Anne's Pond (Centerville, MD)	39	0.32	0.22	0.53	-0.31	0.66
Club Run Bioretention Cell (Chantilly, VA)	39	25.1	14.7	52.5	-0.41	1.09
Mean		8.85	5.20	18.1	-0.38	0.65

6.15.10.2 *Influent NO_3^- concentration sensitivity*

Mean daily effluent NO_3^- concentrations were observed to increase with increased $NO3_{in}$ input values as evidence by a corresponding mean S_x value of 0.835. Because NO_3^- concentrations did not affect any other pollutant concentration within the model, changes in TSS, DO, and NH_4^+ were not observed in the NIT1 and NIT2 designs. Additionally, while effluent NO_3^- concentrations were very sensitive to changes in $NO3_{in}$, the base, NIT1, and NIT2 designs resulted in the same WSI score of 0.640 because each design produced a mean daily effluent NO_3^- concentration less than 0.36 mg/L, which represented the upper limit of NO_3^- concentrations estimated to support healthy downstream ecosystems (see Section 3.4.7.1). Therefore, the overall sustainability of the stormwater wetland, as it was defined in the current study, was not affected by the changes in $NO3_{in}$ in the NIT1 and NIT2 designs. Despite the WSI score insensitivity to changes in $NO3_{in}$, it should be noted that the WSI scores of wetland designs producing effluent NO_3^- concentrations greater than 0.36 mg/L would be sensitive to $NO3_{in}$ values.

Table 6-56 Relevant PC values, relative sensitivities, and deviation sensitivities based on input $NO3_{in}$ values of 0.50 mg/L (base), 0.316 mg/L (NIT1), and 0.704 mg/L (NIT2). The relationship direction indicates where the PC value increases (+) or decreases (-) with a corresponding increase in $NO3_{in}$.

Performance Criteria	Base	NIT1	NIT2	S_x	$\lvert D_x \rvert$	Relationship direction
Mean daily TSS Conc. (mg/L)	15.4	15.4	15.4	1.13 x10^{-7}	6.96 x10^{-7}	-
Mean daily DO Conc. (mg/L)	10.3	10.3	10.3	0.00348	0.0234	-
Mean daily NH4 Conc. (mg/L)	0.0782	0.0782	0.0782	1.10	0.0548	-
Mean daily NO3 Conc. (mg/L)	0.258	0.173	0.353	0.117	0.0193	+
High-marsh PC	0.962	0.962	0.962	-3.62 x10^{-7}	2.19 x10^{-7}	-
Low-marsh PC	0.983	0.983	0.983	-1.48 x10^{-7}	9.17 x10^{-7}	-
High-Flow PC	0.316	0.316	0.316	1.32 x10^{-6}	1.67 x10^{-7}	+
Low-Flow PC	0.223	0.223	0.223	-8.39 x10^{-7}	1.18 x10^{-7}	-
Flow-Variation PC	2.90	2.90	2.90	1.04 x10^{-6}	1.89 x10^{-7}	+
Flood-Control PC	0.382	0.382	0.382	-2.16 x10^{-7}	5.21 x10^{-7}	-

6.15.11 Influent DO concentration (DO_{in})

6.15.11.1 Influent DO concentration range in stormwater wetland

A base DO_{in} concentration of 7.5 mg/L was used in the example stormwater wetland designed in the current study. Influent DO concentrations can vary based on temperature (colder water can sustain more oxygen), nutrient (NH_4^+ and NO_3^- in the current study) concentrations, as well as other contaminant concentrations. Very few studies reported DO runoff levels, as it is often difficult and time-consuming to analyze for. However, the USEPA (1998) reported that urban stormwater runoff throughout the US was found to have DO concentrations of greater than or equal to 5 mg/L and cited that urban runoff generally did not cause downstream DO sags. Therefore, a sensitivity analysis range of 5 to 15 mg/L was estimated for the example design wetland. This range accounted for the minimum concentration of 5 mg/L cited by the USEPA (1998) up to the maximum DO saturation concentration of about 15 mg/L during colder months. The current study referred to stormwater designs with DO_{in} input values of 5 and 15 mg/L as the DOX1 and DOX designs.

6.15.11.2 Influent DO concentration sensitivity

The changes made to DO_{in} in the designs DOX1 and DOX2 did not significantly affect the stormwater wetland performance (see Table 6-60). Mean effluent DO concentrations were the only outputs sensitive to changes in DO_{in} with a mean S_x value of 0.0961, which indicated that internal wetland and effluent DO concentrations were directly related to DO_{in} values. However, because the base stormwater design already produced near-saturated DO levels via surface aeration and photosynthesis within the wetland, changes to influent water DO levels did not strongly impact wetland DO concentrations. As a result of the insensitivity of the stormwater wetland to changes in DO_{in} values the final WSI scores for the base, DOX1, and DOX2 designs were all equal to 0.640. While the stormwater wetland design was resilient to changes in DO_{in} due to strong DO production mechanisms within the wetland, designs with weaker surface aeration and photosynthesis would be more sensitive to such changes in DO_{in}.

Table 6-57 Relevant PC values, relative sensitivities, and deviation sensitivities based on input DO_{in} values of 7.5 mg/L (base), 5 mg/L (DOX1), and 15 mg/L (DOX2). The relationship direction indicates where the PC value increases (+) or decreases (-) with a corresponding increase in DO_{in}.

Performance Criteria	Base	DOX1	DOX2	S_x	$\lvert D_x \rvert$	Relationship direction
Mean daily TSS Conc. (mg/L)	15.4	15.4	15.4	$9.05\text{x}10^{-8}$	$6.96\text{ x}10^{-7}$	-
Mean daily DO Conc. (mg/L)	10.3	9.8	10.7	0.0961	0.482	+
Mean daily NH4 Conc. (mg/L)	0.0782	0.0782	0.0782	$2.23\text{x}10^{-7}$	$8.72\text{ x}10^{-9}$	-
Mean daily NO3 Conc. (mg/L)	0.258	0.258	0.258	$-1.34\text{x}10^{-7}$	$3.46\text{ x}10^{-8}$	-
High-marsh PC	0.962	0.962	0.962	$-2.28\text{x}10^{-7}$	$2.19\text{ x}10^{-7}$	-
Low-marsh PC	0.983	0.983	0.983	$-9.33\text{x}10^{-8}$	$9.17\text{ x}10^{-8}$	-
High-Flow PC	0.316	0.316	0.316	$1.06\text{x}10^{-6}$	$1.67\text{ x}10^{-7}$	+
Low-Flow PC	0.223	0.223	0.223	$-5.29\text{x}10^{-7}$	$1.18\text{ x}10^{-7}$	-
Flow-Variation PC	2.90	2.90	2.90	$6.53\text{x}10^{-7}$	$1.89\text{ x}10^{-6}$	+
Flood-Control PC	0.382	0.382	0.382	$-1.36\text{x}10^{-7}$	$5.21\text{ x}10^{-8}$	-

6.15.12 Nitrification rate constant (K_{NIT})

6.15.12.1 Nitrification rate constant range in stormwater wetland

Because values of nitrification rates (K_{NIT}) at the 1-hr scale were not available in the literature, variation about the base value of 0.004 hr^{-1} was estimated based on its behavior in calibration. Reducing K_{NIT} from 0.01 to 0.008 hr^{-1} resulted in the same daily mean effluent NH_4^+ concentration of 0.05 mg/L suggesting that K_{NIT} required larger changes in order to have a greater effect on model output. Therefore, because effluent NO_3^- concentrations were not sensitive to small changes (±20%) in K_{NIT}, the sensitivity bounds about K_{NIT} were made large enough to observe output NO_3^- sensitivity. Based on this K_{NIT} behavior, a sensitivity range of 0.0004 to 0.04 hr^{-1} was chosen to evaluate K_{NIT} importance in the model. The corresponding designs used to test the sensitivity of the model to K_{NIT} values of 0.0004 and 0.04 hr^{-1} were referred to the KN1 and KN2 designs.

6.15.12.2 Nitrification rate constant sensitivity

The changes made to K_{NIT} in the KN1 and KN2 designs significantly impacted effluent NH_4^+ concentrations and slightly affected DO and NO_3^- effluent concentrations. These trends in mean daily effluent NH_4^+, NO_3^-, and DO concentrations in the KN1 and KN2 designs were similar to those observed in the AM1 and AM2 designs. Therefore, as K_{NIT} increased, wetland nitrification rates also increased, which decreased internal wetland and effluent NH_4^+ concentrations. These increased nitrification rates observed in the KN2 design also required more oxygen and produced more NO_3^- within the wetland. As a result of these model mechanisms, increasing the K_{NIT} value increased mean daily effluent NH_4^+ concentrations (mean S_x of -0.345), decreased mean daily effluent DO concentrations (mean S_x value of -0.00169), and increased mean daily effluent NO_3^- concentrations (mean S_x values of 0.0564).

The sensitivity of effluent NH_4^+ concentrations to changes in K_{NIT} input values were reflected in the final respective WSI scores of the KN1, base, and KN2 designs of 0.622, 0.640, and 0.687 (see Table 6-59). These resulting WSI scores suggested that the stormwater wetland design sustainability was fairly sensitive to the K_{NIT}. More knowledge about the sensitivity of downstream species to NH_4^+ is necessary to determine whether these differences in WSI scores are significant. Again, because the stormwater wetland design promoted internal DO levels near saturation, DO levels were not significantly impacted by changes in K_{NIT}. However, if wetland DO levels were closer to 2 mg/L, changes in K_{NIT} would be restricted as DO could also act as a limiting factor in nitrification NH_4^+ removal and NO_3^- production.

Table 6-58 Relevant PC values, relative sensitivities, and deviation sensitivities based on input K_{NIT} values of 0.004 hr^{-1} (base), 0.0004 hr^{-1} (KN1), and 0.04 hr^{-1} (KN2). The relationship direction indicates where the PC value increases (+) or decreases (-) with a corresponding increase in K_{NIT}.

Performance Criteria	Base	KN1	KN2	S_x	$\|D_x\|$	Relationship direction
Mean daily TSS Conc. (mg/L)	15.4	15.4	15.4	2.77×10^{-8}	6.96×10^{-7}	-
Mean daily DO Conc. (mg/L)	10.3	10.3	10.2	-0.00169	0.0404	-
Mean daily NH4 Conc. (mg/L)	0.0782	0.121	0.0161	-0.345	0.0522	-
Mean daily NO3 Conc. (mg/L)	0.258	0.235	0.289	0.0564	0.0270	+
High-marsh PC	0.962	0.962	0.962	-1.14×10^{-7}	2.19×10^{-7}	-
Low-marsh PC	0.983	0.983	0.983	-4.66×10^{-8}	9.17×10^{-8}	-
High-Flow PC	0.316	0.316	0.316	3.23×10^{-7}	1.67×10^{-7}	+
Low-Flow PC	0.223	0.223	0.223	-2.65×10^{-7}	1.18×10^{-7}	-
Flow-Variation PC	2.90	2.90	2.90	3.27×10^{-7}	1.89×10^{-6}	+
Flood-Control PC	0.382	0.382	0.382	-6.82×10^{-8}	5.21×10^{-8}	-

Table 6-59 Resulting normalized metrics and final wetland sustainability indices (WSI's) for the base, KN1 and KN2 stormwater wetland designs. All WSI scores were computed assuming equal weights for all water quality and hydrologic metrics.

Performance Criteria	Metric weights	Base design metrics	KN1 metrics	KN2 metrics
Mean daily TSS Conc. (mg/L)	0.1	0.328	0.328	0.328
Mean daily DO Conc. (mg/L)	0.1	1.00	1.00	1.00
Mean daily NH4 Conc. (mg/L)	0.1	0.528	0.346	1.000
Mean daily NO3 Conc. (mg/L)	0.1	1.00	1.00	1.00
High-marsh PC	0.1	0.999	0.999	0.999
Low-marsh PC	0.1	1.00	1.00	1.00
High-Flow PC	0.1	0.533	0.533	0.533
Low-Flow PC	0.1	0.396	0.396	0.396
Flow-Variation PC	0.1	0.00	0.00	0.00
Flood-Control PC	0.1	0.618	0.618	0.618
Final *WSI* score	---	0.640	0.622	0.687

6.15.13 Denitrification rate constant (K_{DNT})

6.15.13.1 Denitrification rate constant range in stormwater wetland

Literature values for hourly K_{DNT} were also not available in the literature, requiring sensitivity analysis bounds based on K_{DNT} calibration behavior. K_{DNT} was more sensitive than K_{NIT}, with effluent NO_3^- mean daily concentrations of 0.35 and 0.26 mg/L resulting from respective K_{DNT} inputs of 0.0208 and 0.05 hr^{-1}. Based on this sensitivity, an estimated range of 0.0275 and 0.0825 hr^{-1} (±50% of the mean) was chosen for the sensitivity analysis. The stormwater wetland designs incorporating K_{DNT} input values of 0.0275 and 0.0825 hr^{-1} were referred to as KD1 and KD2.

6.15.13.2 Denitrification rate constant sensitivity

The changes made to K_{DNT} in the KD1 and KD2 designs significantly affected the mean daily effluent NO_3^- concentration with a corresponding mean S_x of -0.384. As evidenced by the negative resulting S_x, NO_3^- concentrations decreased with increasing K_{DNT} values. Therefore, the increased denitrification rates promoted by higher K_{DNT} values resulted in greater removal of NO_3^- from the wetland via denitrification. Additionally, other wetland outputs were not affected by changes in K_{DNT} as denitrification only controlled NO_3^- levels within the model. While effluent NO_3^- concentrations were fairly sensitive to input K_{DNT} concentrations, the resulting WSI score did not change for the KD1 and KD2 designs, as all resulting mean daily NO_3^- concentrations were below 0.36 mg/L, which was the estimated upper limit indicative of a healthy downstream ecosystem. Therefore, while the sustainability of the stormwater design was not affected by changes in K_{DNT}, designs with greater NO_3^- concentrations may exhibit greater sensitivity.

Table 6-60 Relevant PC values, relative sensitivities, and deviation sensitivities based on input K_{DNT} values of 0.055 hr^{-1} (base), 0.0275 hr^{-1} (KD1), and 0.0825 hr^{-1} (KD2). The relationship direction indicates where the PC value increases (+) or decreases (-) with a corresponding increase in K_{DNT}.

Performance Criteria	Base	KD1	KD2	S_x	$\|D_x\|$	Relationship direction
Mean daily TSS Conc. (mg/L)	15.4	15.4	15.4	9.05×10^{-8}	6.96×10^{-7}	-
Mean daily DO Conc. (mg/L)	10.3	10.3	10.3	4.88×10^{-8}	2.50×10^{-7}	-
Mean daily NH4 Conc. (mg/L)	0.0782	0.0782	0.0782	2.23×10^{-7}	8.72×10^{-9}	-
Mean daily NO3 Conc. (mg/L)	0.258	0.320	0.221	-0.384	0.495	-
High-marsh PC	0.962	0.962	0.962	0.00	2.19×10^{-7}	-
Low-marsh PC	0.983	0.983	0.983	0.00	9.17×10^{-8}	-
High-Flow PC	0.316	0.316	0.316	1.06×10^{-6}	1.67×10^{-7}	+
Low-Flow PC	0.223	0.223	0.223	0.00	1.18×10^{-7}	-
Flow-Variation PC	2.90	2.90	2.90	0.00	1.89×10^{-6}	+
Flood-Control PC	0.382	0.382	0.382	0.00	5.21×10^{-8}	-

6.15.14 Model input sensitivity

Pollutant influent concentrations, TSS particle diameter, and the nitrification rate constant were found to be the most important parameters with respect to the stormwater wetland design used in the current study. While the TSS particle diameter and influent concentrations are potentially measurable input parameters, the nitrification rate constant is a calibration input. Table 6-61 summarizes the base, high, and low values assigned to all user input and calibration parameters assumed to have significant variation within the stormwater wetland design. As shown in Table 6-62, the TSS particle diameter D and the nitrification rate constant K_{NIT} inputs had the largest impact on final wetland WSI scores. Based on the sensitivities and WSI scores associated with the changes made to each input parameter, the water quality input parameters, excluding those directly related to DO and NO_3^- concentrations, appeared to be most sensitive to variation and user calibration.

Wetland DO levels were not sensitive to changes in relevant input parameters due to the near-saturation DO concentrations maintained in the stormwater wetland design. Because the design wetland efficiently produced oxygen via photosynthesis and surface aeration, changes in DO_{in}, $PMAX$, $NH4_{in}$, and K_{NIT} did not significantly impact wetland effluent DO concentrations. However, as previously mentioned, DO levels in a wetland design with fewer areas with submerged vegetation and less surface aeration could be greatly impacted by these parameters. Therefore, while DO concentrations were not sensitive in the stormwater design, the effects of DO_{in}, $PMAX$, $NH4_{in}$, and K_{NIT} on DO levels should not be ignored as they should become more relevant in wetland designs with lower DO concentrations.

It was also noted that despite effluent NO_3^- strong sensitivity to changes in the input parameters K_{DNT} and $NO3_{in}$, the final WSI scores associated with changes to these parameters did not differ from that of the base design. The reason for this insensitivity of the final WSI score was due to the fact that mean daily effluent NO_3^- concentrations resulting from all K_{DNT} and $NO3_{in}$ designs were below 0.39 mg/L, which was the estimated upper limit for NO_3^- concentrations associated with healthy downstream ecosystems. Therefore, while the stormwater design developed in the current study did not reach this threshold, if a design promoted NO_3^- effluent concentrations greater than 0.39 mg/L, K_{DNT} and $NO3_{in}$ would have a greater impact on the final wetland WSI score.

While model outputs and overall performance was sensitive to water quality-related input parameters, ET-related input parameters did not have the same impact. This discrepancy is likely due to the fact that effluent pollutant concentrations, which were directly related to the corresponding input water quality parameters, were used to quantify wetland performance in the form of both PC values and metrics. Conversely, ET depths, which were a function of the ET input parameters, were not used in the direct computation of wetland PC values. Therefore, while changes in the ET parameters did affect change in ET depths, such changes did not significantly impact wetland performance. Additionally, while changes in ET rates did have a slight impact on wetland hydrology, runoff inflow and rainfall proved to be the dominating factors in both the wetland water balance and hydrology. The concentrating effects of ET on pollutant concentrations also appear to be minimal within the context of the stormwater wetland design. Despite these results, the ET input parameters may be used to increase or decrease ET depths within a given design in order to obtain the appropriate wetland water balance.

Based on the input sensitivity results, the current study suggested that the model user collect as much relevant data as possible in order to best calibrate a given wetland design. Wetland performance was found to be especially sensitivity to changes to water quality inputs. Unfortunately, limited data are available with respect to wetland water quality parameters, especially the nitrification and denitrification rate constants at an hourly time interval. Given the lack of data within the literature, the current model should be used to assess the effects of changes in both design and input parameters on a given wetland design rather than as a predictive tool.

Table 6-61 All resulting base, low and high values for each input parameters evaluated in the sensitivity analysis.

Input parameter	Base value	Low	High
Wetland Albedo	0.159	0.069	0.245
LAI	6.5	3.8	9.2
Shelter factor	0.75	0.525	0.975
Max leaf conductance	9.7	9.41	10.0
Emergent vegetation height above water (m)	1.65	1.18	2.00
TSS Particle diameter (m)	1.2×10^{-6}	7.5×10^{-7}	5.5×10^{-6}
PMAX (mg-O2/m2-hr)	910	696.15	1124
Influent TSS (mg/L)	43.2	23.7	184
Influent NH4 (mg/L)	0.13	0.0918	0.274
Influent NO3 (mg/L)	0.5	0.316	0.705
Influent DO (mg/L)	10	5	15
Nitrification rate constant (hr^{-1})	0.004	0.0004	0.04
Denitrification rate constant (hr^{-1})	0.055	0.0275	0.0825

Table 6-62 Final wetland WSI scores for base, low and high values for each input parameter evaluated in the sensitivity analysis.

Input Category	Input parameter	WSI score		
		Base value	Low	High
ET calibration parameters	Wetland Albedo	0.640	0.640	0.640
	LAI	0.640	0.640	0.640
	Shelter factor	0.640	0.640	0.640
	Max leaf conductance	0.640	0.640	0.640
	Emergent vegetation height above water (m)	0.640	0.640	0.640
Water Quality input parameters	TSS Particle diameter (m)	0.640	0.630	0.707
	Influent TSS (mg/L)	0.640	0.659	0.618
	Influent NH4 (mg/L)	0.640	0.654	0.610
	Influent NO3 (mg/L)	0.640	0.640	0.640
	Influent DO (mg/L)	0.640	0.640	0.640
Water Quality calibration parameters	PMAX (mg-O2/m2-hr)	0.640	0.640	0.640
	Nitrification rate constant (hr^{-1})	0.640	0.622	0.687
	Denitrification rate constant (hr^{-1})	0.640	0.640	0.640

Chapter 7: USEPA Municipal Wastewater Wetland

7.1 DESIGN EXAMPLE AND PROCEDURE

The current section discusses in detail the procedures and methods followed to design a constructed wetland for the treatment of primary effluent in a municipal wastewater treatment plant (WWTP). Therefore, the designed treatment wetland would serve as secondary treatment within a given WWTP. In order to design this treatment wetland, a design example for a free water surface treatment wetland outlined in the EPA manual of *Constructed Wetlands Treatment of Municipal Wastewaters* (USEPA 2000) was followed. This design example was modified in the current study to fit scaled down primary effluent water quantity and quality data obtained from a local WWTP. According to USEPA specifications, the wetland was required to be designed to meet BOD and TSS 30-day mean effluent requirements of 30 mg/L.

In order to design a rationally sized treatment wetland, the seasonal WWTP hourly primary effluent flow curves derived in Section 4.4.1.5.3 from flow data from a local WWTP had to be scaled down. The mean of the actual WWTP flow was 25.1 MGD, which, according to USEPA (2000), would require a treatment wetland area between 100 and 625 ac depending on the primary effluent pollutant content. USEPA (2000) reported that 90% of all wastewater treatment wetlands are less than 250 acres, while the majority of wastewater treatment wetlands have an area of 25 acres or less (USEPA 2000). Therefore, the WWTP hourly curves derived in Equations 4-60, 4-61, and 4-62 were scaled down to have an overall daily mean flow Q_{avg} of 5.16 MGD

(19,608 m^3/d), which was close to the Q_{avg} of 5 MGD used in the actual USEPA (2000) example. The example shown in USEPA (2000) also estimated the corresponding maximum monthly flow Q_{max} for a given facility to be two times Q_{avg} for design purposes. Based on this assumption, the design Q_{max} was computed to equal 10.32 MGD (39,216 m^3/d).

Corresponding primary effluent TSS, BOD, and NH_4^+ daily grab samples for the calendar year of 2012 were also obtained from the same local WWTP from which flow data was obtained and are summarized in Table 7-1. While water quality can vary greatly within any given day within WWTP, grab samples were assumed to serve as reasonable estimates of daily averages within the context of the current study. Additionally, influent NO_3^- levels were assumed to equal zero and DO levels were assumed to equal 2 mg/L as indicative of anaerobic water leaving primary treatment. The local WWTP data did not exhibit any trends relating flowrate and water quality levels. Therefore, water quality concentrations were assumed to be constant and independent of flowrate.

Table 7-1 Shows WWTP primary effluent estimated daily water quality concentrations as based on daily grab samples obtained from a local WWTP. TKN was estimated by the current study based on the ratio of NH_4^+ to TKN influent concentrations reported by USEPA (2000) and summarized in Table 2-11.

Constituent	**Mean Concentration (mg/L)**	**Standard Deviation (mg/L)**
TSS	59.5	18.0
BOD	93.9	22.2
NH_4^+	22.3	3.0
NO_3^-	0	---
TKN*	47.1	---

Given all of the required inputs, the associated areal loading rates required for wetland effluent concentrations of 30 mg/L for BOD and TSS were input to Equation 2-5 to determine total wetland area based on BOD and TSS treatment, as well as Q_{avg} and Q_{max} (USEPA 2000):

$$r_{AL} = 0.001 \cdot \frac{Q \cdot C}{A_w} \quad (7\text{-}1)$$

where Q represents the influent flowrate (m^3/d) to the wetland, C is the influent concentration of the constituent of concern (i.e., BOD or TSS), A_w is the wetland area (ha), and r_{AL} is the areal loading rate (kg/ha-d) for constituent C. The term 0.001 serves as a conversion factor. The largest resulting area A_w was chosen as an initial wetland area estimate. Wetland areas based on BOD requirements were calculated first by rearranging Equation 2-5 (USEPA 2000):

$$A_{w(avg)} = \frac{(19{,}608\ \mathrm{m^3/d})(93.9\ \mathrm{mg/L})}{60\ \mathrm{kg/ha-d}} \times \frac{1000\ \mathrm{L}}{\mathrm{m^3}} \times \frac{\mathrm{kg}}{10^6\ \mathrm{mg}} = 31\ \mathrm{ha} \quad (7\text{-}2)$$

$$A_{w(\max)} = \frac{(39{,}216\ \mathrm{m^3/d})(93.9\ \mathrm{mg/L})}{60\ \mathrm{kg/ha-d}} \times \frac{1000\ \mathrm{L}}{\mathrm{m^3}} \times \frac{\mathrm{kg}}{10^6\ \mathrm{mg}} = 61\ \mathrm{ha} \quad (7\text{-}3)$$

where $A_{w(avg)}$ represents the required wetland area (ha) to produce a BOD effluent concentration of 30 mg/L at average flow and $A_{w(\max)}$ is the required wetland area (ha) for BOD treatment at maximum flow. Next, the same procedure was followed to determine resulting wetland areas for TSS treatment (USEPA 2000):

$$A_{w(avg)} = \frac{(19{,}608\ \mathrm{m^3/d})(59.5\ \mathrm{mg/L})}{50\ \mathrm{kg/ha-d}} \times \frac{1000\ \mathrm{L}}{\mathrm{m^3}} \times \frac{\mathrm{kg}}{10^6\ \mathrm{mg}} = 23\ \mathrm{ha} \quad (7\text{-}4)$$

$$A_{w(\max)} = \frac{(39{,}216\ \mathrm{m^3/d})(59.5\ \mathrm{mg/L})}{50\ \mathrm{kg/ha-d}} \times \frac{1000\ \mathrm{L}}{\mathrm{m^3}} \times \frac{\mathrm{kg}}{10^6\ \mathrm{mg}} = 47\ \mathrm{ha} \quad (7\text{-}5)$$

From this step, an initial estimate of the total wetland area of 61 ha was made by the current study. The same sizing procedure could also be used to size vegetated and open water wetland sections separately. However, due to a lack of data, it is currently more accurate to size wetlands as a whole (USEPA 2000). Given the high primary effluent BOD concentrations entering the wetland, the resulting wetland area would have to be 61 ha (151 acres) in order to achieve an aerial loading rate of 60 kg/ha-d. This large wetland area is not realistic in many cases as sufficient land may not be available. Within the context of the current study, this large wetland area was used only to evaluate the behavior and performance of a treatment wetland sized according to USEPA (2000) guidelines.

Once an initial wetland area A_w of 151 ac was determined, the resulting wetland hydraulic retention time was estimated based on assumed minimum water depths of 0.6 m (2 ft) in zones 1 and 3, and of 1.2 m (4 ft) in zone 2 (USEPA 2000). Additionally, USEPA (2000) assumed a wetland porosity ε of 0.75 for zones 1 and 3, and a ε of 1 for zone 2. Weighted mean values of 0.8 m (2.62 ft) and 0.8 for the overall wetland depth and ε were then estimated by USEPA (2000) to determine the wetland hydraulic retention time t_{HR} given an A_w of 61 ha as defined by Equation 2-6 (USEPA 2000):

$$t_{HR} = 10{,}000 \cdot \frac{A_w \cdot h_w \cdot \varepsilon}{Q} \tag{7-6}$$

where h_w represents the mean wetland depth (m), ε is a measure of porosity of the flow path through the wetland with respect to vegetation (dimensionless), and 10,000 is a conversion factor from ha to m^2. The resulting t_{HR} values for Q_{avg} and Q_{max} were then calculated with Equation 2-6 (USEPA 2000):

$$t_{HR(avg)} = \frac{(61\,\text{ha})(0.8\,\text{m})(0.8)}{19{,}608\,\text{m}^3/\text{d}} \times \frac{10{,}000\,\text{m}^2}{\text{ha}} = 19.9\,\text{days} \tag{7-7}$$

$$t_{HR(\max)} = \frac{(61\,\text{ha})(0.8\,\text{m})(0.8)}{39{,}216\,\text{m}^3/\text{d}} \times \frac{10{,}000\,\text{m}^2}{\text{ha}} = 9.96\,\text{days} \tag{7-8}$$

If it is assumed that the area of each zone is equal, each zone would have an individual t_{HR} of 6.6 days during days with mean flow and of 3.3 days during days with high flow, which is sufficient for complete treatment. Ideally, the minimum t_{HR} for each zone should be 2 days (occurring at Q_{max}). Therefore, the wetland is properly sized based on hydraulic retention time. If individual t_{HR} values of 2 days at Q_{max} had not been achieved for each wetland zone, USEPA (2000) suggested making $t_{HR(avg)}$ for each zone 4 days. This longer hydraulic retention time may allow for unwanted algal growth in zones 1 and 3, but was assumed fine within this procedure (USEPA 2000). From this assumption, a larger total wetland area could be calculated based on hydraulic retention time requirements.

Given a final wetland area, the individual zone areas could be estimated according to their water depths *h*, porosity ε, and hydraulic retentions times t_{HR} at Q_{max} (3.3 days for each zone). The area of zone 2 was calculated based on Q_{max} accordingly (USEPA 2000):

$$A_{2(t)} = \frac{(3.3\,\text{d})(39{,}216\,\text{m}^3/\text{d})}{(1.2\,\text{m})(1)} \times \frac{1\,\text{ha}}{10{,}000\,\text{m}^2} = 10.8\,\text{ha} \tag{7-9}$$

where $A_{2(t)}$ is the resulting surface area of zone 2 based on a design hydraulic retention time of 3.3 days at Q_{max}. Areas for zones 1 and 3 could then be calculated accordingly (USEPA 2000):

$$A_{1,3(t)} = \frac{61\,\text{ha} - 10.8\,\text{ha}}{2} = 25.1\,\text{ha} \tag{7-10}$$

where $A_{1,3(t)}$ represents the individual surface areas for zones 1 and 3. Therefore the final surface areas will equal 25.1 ha, 10.8 ha, and 25.1 ha respectively for zones 1, 2, and 3. This zone area calculation was the last quantitative step given in the EPA wetland sizing procedure.

7.2 FINAL MUNICIPAL WASTEWATER WETLAND DESIGN

From this point on, the qualitative discussion of treatment wetland design by USEPA (2000) was followed to configure the wetland in more detail. The EPA suggested a length-to-width ratio greater than 3:1 for the entire wetland. It was also suggested that the wetland be divided into two trains with parallel flow, with each train treating half of the total flow (USEPA 2000). Therefore, the areas calculated by USEPA (2000) in Section 7.1 for each zone were divided by 2 in the current study to create two parallel wetland trains and a length to width ratio of 3:1 was applied to each of the resulting six zones (two of each zone). The halved areas for zones 1, 2, and 3 were 12.6, 5.4, and 12.6 ha, respectively.

All zones were assumed to have a rectangular shape. While USEPA (2000) stated that treatment wetlands have been constructed in a number of shapes including rectangles, ovals, kidney shapes, and crescent shapes, no shape was found to perform dominantly. Therefore, a rectangular shape was chosen for computational ease. The resulting zone 2 dimensions for one wetland train were then calculated to have a 3:1 L:W ratio accordingly:

$$A_{2(t)} = 3W_2^2 \tag{7-11}$$

$$W_2 = \sqrt{\frac{A_{2(t)}}{3}} = \sqrt{\frac{5.4\,\mathrm{ha}}{3} \times \frac{10{,}000\,\mathrm{m}^2}{\mathrm{ha}}} = 134\,\mathrm{m}\,(440\,\mathrm{ft}) \tag{7-12}$$

$$L_2 = 3 \cdot W_2 = 3 \cdot 134\,\mathrm{m} = 402\,\mathrm{m}\,(1319\,\mathrm{ft}) \tag{7-13}$$

where W_2 and L_2 are respectively the width and length of zone 2. Similarly, the dimensions for zones 1 and 3 were determined with an initial L:W ratio of 1:3:

$$W_{1,3} = \sqrt{\frac{12.6\,\mathrm{ha}}{3} \times \frac{10{,}000\,\mathrm{m}^2}{\mathrm{ha}}} = 205\,\mathrm{m}\,(672\,\mathrm{ft}) \tag{7-14}$$

$$L_{1,3} = 3 \cdot W_{1,3} = 615\,\mathrm{m}\,(2017\,\mathrm{ft}) \tag{7-15}$$

where $W_{1,3}$ and $L_{1,3}$ are the width and length of zones 1 and 3.

USEPA (2000) also suggested the installation of an initial settling zone for this example due to the high TSS influent concentration to the wetland. As defined by USEPA (2000), an inlet settling zone should add 1 day to the total wetland hydraulic retention time, have a depth of 1 m (3 ft), and be devoid of vegetation (ε equal to 1). Therefore, the inlet settling zone area was estimated based on Q_{max} using Equation 2-6 accordingly:

$$A_{IN(t)} = \frac{(1\,\mathrm{d})(39{,}216\,\mathrm{m}^3/\mathrm{d})}{(1\,\mathrm{m})(1)} \times \frac{1\,\mathrm{ha}}{10{,}000\,\mathrm{m}^2} = 3.9\,\mathrm{ha}\,(9.7\,\mathrm{ac}) \tag{7-16}$$

where $A_{IN(t)}$ represents the surface area of the inlet settling zone. This area was also divided in half and introduced as the first zone in each of the two parallel trains. This resulted in two adjacent inlet cells with areas of 1.95 ha (4.8 ac). The inlet settling zone should be constructed across the inlet of the wetland (the width of zone 1). Therefore the width of the inlet zone W_{IN} set equal to W_2 (440 ft) resulting in an inlet zone length L_{IN} of 475 ft. The total wetland hydraulic retention time would then be 10.96 days during Q_{max} and 20.9 days during Q_{avg}.

From the initial required estimates of each zone width and lengths, dimensions were modified to establish a constant width for the entire wetland. Each zone, expect for the inlet settling zone, was also restricted by a required L:W ratio greater than 3:1. A common width of 140 m (458 ft) was chosen as the wetland width. The resulting final zone dimensions and corresponding areas are given in Table 7-2. The final area of one train including the inlet zone was 43.6 ha (108 ac). Therefore, the total wetland area (comprised of two parallel trains) was 87.1 ha (216 ac). Again, due to the high BOD concentrations entering the wetland, the resulting required area would be unrealistic for most sites, but could be reasonable in areas with large amounts of land. The purpose of this design was to show treatment wetland behavior based on USEPA (2000) design as well as real influent WWTP inputs.

Table 7-2 Shows the final wetland zone dimensions and L:W ratios for one train of the two-train wetland system.

Zone	**Width (ft)**	**Length (ft)**	**Final Area (ha)**	**L:W**
Inlet Settling	458	458	1.95	1:1
1	458	3206	13.6	7:1
2	458	1374	14.4	3:1
3	458	3206	13.6	7:1

Nitrogen was not considered in the USEPA (2000) design procedure due to the lack of data and knowledge of nitrogen behavior within these complex systems.

Limited data were, however, used to estimate wetland effluent TKN concentrations based on the TKN aerial loading rate of the final wetland design (see Figure 7-1). From Table 7-1, an average TKN concentration of 47.1 mg/L was estimated to enter the wetland from primary treatment. A resulting TKN areal loading rate was calculated accordingly (USEPA 2000):

$$r_{TKN} = \frac{(19{,}608\ \mathrm{m^3/d}) \cdot (47.1\ \mathrm{mg/L})}{87.1\ \mathrm{ha}} \times \frac{1000\ \mathrm{L}}{\mathrm{m^3}} \times \frac{1\ \mathrm{kg}}{10^6\ \mathrm{mg}} = 10.6\ \mathrm{kg/ha-d} \qquad (7\text{-}17)$$

where r_{TKN} is the resulting estimated TKN areal loading rate at Q_{avg} (kg/ha-d). While data were lacking, a predicted mean TKN effluent of 20 mg/L was estimated based on the limited treatment wetland TKN data reported by USEPA (2000) in Figure 7-1. The treatment wetland designed in the current study included both vegetated (zones 1 and 3) and open space (zone 2) areas and, therefore, corresponded to the three data points in Figure 7-1 labeled to have significant open space. Given the limited data as well as the possible differences between sample wetland designs, significant error was associated with the effluent TKN concentration of 20 mg/L estimated from Figure 7-1.

In order to estimate wetland effluent TSS concentrations, the actual TSS areal loading rate r_{TSS} for the final treatment wetland design was computed based on the influent TSS concentration and the wetland area:

$$r_{TSS} = \frac{(19{,}608\ \mathrm{m^3/d}) \cdot (59.5\ \mathrm{mg/L})}{87.1\ \mathrm{ha}} \times \frac{1000\ \mathrm{L}}{\mathrm{m^3}} \times \frac{1\ \mathrm{kg}}{10^6\ \mathrm{mg}} = 6134\ \mathrm{kg/ha-d} \qquad (7\text{-}18)$$

based on this r_{TSS} value, an effluent TSS concentration of 10 mg/L was estimated from Figure 7-2 (USEPA 2000). Again, due to limited data, differences in wetland designs, and wetland complexity, significant error was associated with this TSS effluent concentration. USEPA (2000) did, however, cite that wetlands with r_{TSS} values less than 30 kg/ha-d had been shown to reliably produce TSS effluent below 20 mg/L. Therefore, a reasonable range for effluent TSS concentrations was assumed to be between 10 and 20 mg/L.

The final municipal treatment wetland design was input into the model in order to assess its performance. Only one train of the two-train wetland system was modeled in order to simplify computations (one train has an area of 108 ac). Therefore, at the end of a simulation, all flow values could be multiplied by two to obtain the total effluent volume and flows for the complete wetland with a total area of 216 acres. Effluent concentrations were assumed to be the same for each parallel train in the wetland. Therefore, in order to evaluate the treatment wetland design, it was only necessary to simulate flow through one train.

A cell size of 140 x 140 m (458 x 458 ft) was chosen to route flow through the wetland in order to best characterize flow and zone areas. The resulting number of cells used to simulate each zone is given in Table 7-3. The final cell design for one train of the design municipal wastewater wetland input into the model is shown in Figure 7-3 and summarized in Table 7-4.

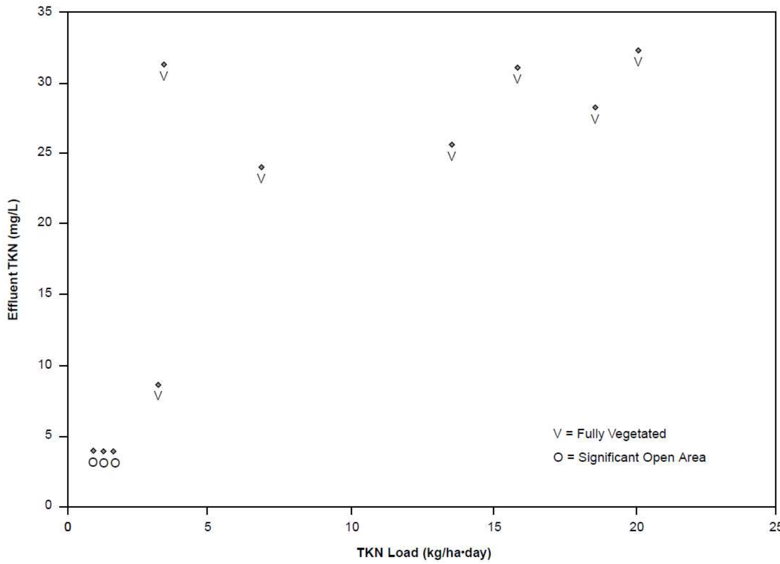

Figure 7-1 Plot of TKN effluent concentrations (mg/L) versus TKN aerial loading rate (kg/ha-d) based on a total of 10 wastewater treatment wetlands (USEPA 2000).

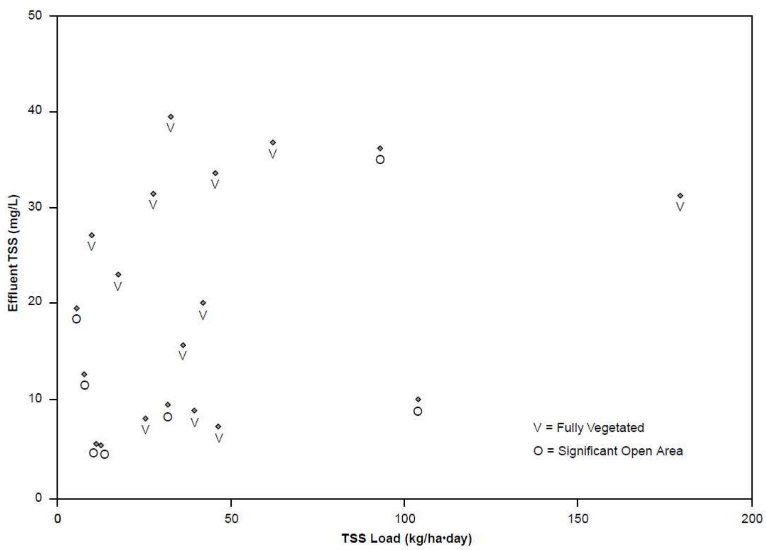

Figure 7-2 plot of TSS effluent concentrations (mg/L) TSS aerial loading rate (kg/ha-d) based on a total of 19 wastewater treatment wetlands (USEPA 2000).

Table 7-3 Shows the number of cells used to simulate each of the four zones within one train of the EPA-design wastewater treatment wetland.

Zone	**Number of cells in zone**	**Zone depth (ft)**
Inlet Settling	1	3
1	7	2
2	3	4
3	7	2

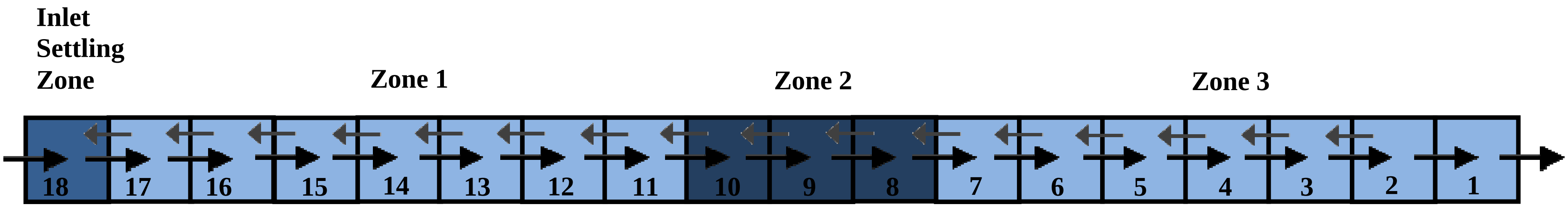

Figure 7-3 Shows the USEPA Municipal wastewater wetland design for one train of the wetland entered into the model. Each number cell has dimensions of 458 x 458 ft. Black arrows show FID flowpath while grey arrows show FID2 flowpath.

Table 7-4 Municipal treatment wetland cell specifications for FID1, FID2, vegetation type (VEG), initial design depth in ft SS, and cell elevation above a datum EL in ft. Vegetation descriptor values of 0, 1, 2 indicate respectively that a given cell has no vegetation, emergent vegetation, and submerged vegetation.

Cell	FID	FID2	SS (ft)	VEG	EL (ft)	BERM (ft)
1	0	1	3	0	1	0
2	1	3	2	1	2	0
3	2	4	2	1	2	0
4	3	5	2	1	2	0
5	4	6	2	1	2	0
6	5	7	2	1	2	0
7	6	8	2	1	2	0
8	7	9	2	1	2	0
9	8	10	4	2	0	0
10	9	11	4	2	0	0
11	10	12	4	2	0	0
12	11	13	2	1	2	0
13	12	14	2	1	2	0
14	13	15	2	1	2	0
15	14	16	2	1	2	0
16	15	17	2	1	2	0
17	16	18	2	1	2	0
18	17	18	2	1	2	0

7.3 TREATMENT WETLAND CALIBRATION

Before the wastewater treatment wetland designed in Sections 7.1 and 7.2 could be evaluated, it had to be calibrated. Mean daily effluent TSS and TKN concentrations were estimated to respectively equal about 10 and 20 mg/L. A conversion factor of 1.59 was used to estimate effluent NH_4^+ concentrations from the derived TKN effluent concentration based on relative mean effluent TKN (19 mg/L) and NH_4^+ (12 mg/L) concentrations reported by USEPA (2000) for 22 wastewater treatment wetlands (see Table 2-11). The resulting wetland mean daily effluent NH_4^+ concentration was then estimated to equal 12.5 mg/L. Data for treatment wetland effluent NO_3^- concentrations was not found in the literature as it generally comprised a very small portion of the effluent nitrogen. As shown in Table 7-1, a mean NO_3^- concentration of 0 mg/L entered the design treatment wetland. Wastewater treatment wetlands are generally anaerobic and promote little production of NO_3^- via nitrification of influent NH_4^+. Additionally, any NO_3^- produced should also be reduced to $N_{2(g)}$ via denitrification due to the anaerobic conditions in these wetlands. Given this wetland behavior, it was assumed that effluent NO_3^- would be very low compared with effluent NH_4^+ concentrations and simulation proved this to be the case with a final mean daily effluent NO_3^- concentration of 0.11 mg/L.

While a large amount of error is associated with these estimated effluent concentrations, they were only used within the current study to illustrate the wetland behavior and respective performance under different circumstances rather than for specific wetland design. More water quality data would be necessary in order to properly calibrate the model for design purposes. Given the scope of the current study and the estimated effluent TSS and NH_4^+ concentrations, the treatment wetland designed in Section 7.2 was calibrated with respect to water quality performance. All user inputs before and after calibration are summarized in Table 7-5, and all calibration steps and relevant outputs were reported in Table 7-6.

During the calibration the effluent NH_4^+ concentrations reached a lower limit of 14.7 mg/L, which was due to the low dissolved oxygen concentrations (around 2 mg/L) within the wetland. Without oxygen to support the transformation of NH_4^+ to NO_3^- via nitrification, effluent NH_4^+ concentrations were not affected by increases in the nitrification reaction rate K_{NIT} (see Table 7-6). The nitrification that did occur within the wetland occurred mainly in the wetland cells 8, 9, and 10, which contained submerged vegetation that produced dissolved oxygen via photosynthesis. Water velocities were too slow in the remainder of the wetland to promote significant dissolved oxygen levels via surface aeration. As shown in Figure 7-4, cell 8 produced higher daily DO levels than the inlet (cell 18) and outlet (cell 1) cells of the wetland. Increased DO levels during the summer and fall months reflect the lower simulated influent discharge rates to the treatment wetland (see Figure 4-19).

Based on these DO trends within the wetland, nitrification within a treatment wetland could be increased by increasing the retention time in areas with submerged vegetation. The location of areas with submerged vegetation also plays a role in overall nitrogen reduction as the NO_3^- generated from nitrification in the aerobic areas with submerged vegetation (i.e., zone 2) can be reduced to $N_{2(g)}$ via denitrification in the shallower areas with emergent vegetation (i.e., zones 1 and 3). Therefore, increasing the area of deep aerobic zones at the beginning of the wetland may impact total effluent nitrogen levels greater than if they were added to the middle or end of the wetland.

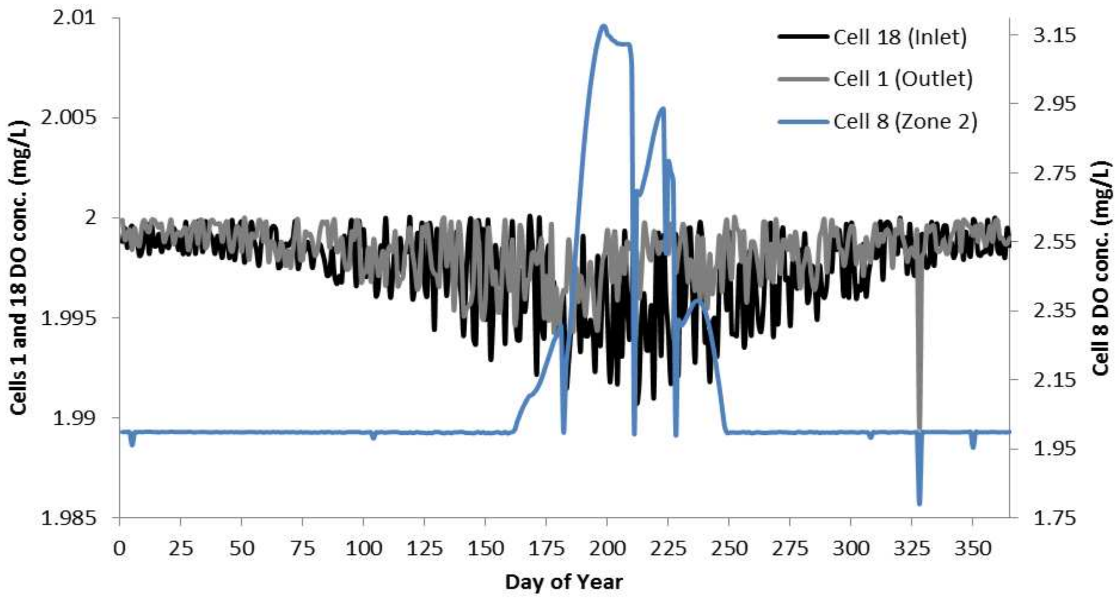

Figure 7-4 Daily DO concentrations (mg/L) in the treatment wetland for cells 18 (black), 1 (grey), and 8 (blue) over the last year of the 25-yr simulation period.

Table 7-5 All user inputs for the municipal wastewater treatment wetland design, their assigned initial values, and final values after calibration.

User input	Initial value	Final value
Number of years of simulation	25	25
Cell length (ft)	458	458
Contributing drainage area (ac)	---	---
Number of cells in wetland design	18	18
FID vector	See Table 7-4	---
Vegetation specification for each cell (no vegetation = 0, emergent = 1, submerged = 1)	See Table 7-4	---
Initial water depth in each cell	See Table 7-4	---
Bottom elevation in each cell	See Table 7-4	---
Berm height at exit of each cell	no berms	---
Orifice or Weir (Orifice = 1, Weir = 2)	2	2
Weir length (ft)	458	458
Orifice area (ft^2)	---	---
Weir invert height H_I (ft)	4	4
Hydraulic conductivity K_V (ft/d)	0	0
Shelter factor f_s	0.75	0.75
Wetland albedo a	0.159	0.159
Leaf area index LAI	6.5	6.5
Maximum leaf conductance C^*_{leaf} (mm/s)	9.7	9.7
Emergent vegetation height z_v (m)	1.65	1.65
Wind speed measurement height z_m (m)	2	2
Maximum photosynthesis rate $PMAX$ (mg-O_2/m^2-hr)	910	910
TSS particle diameter D (m)	6.5×10^{-6}	8.1×10^{-7}
Initial water temperature $T_{w(o)}$ (°C)	15.5	15.5
Nitrification reaction rate K_{NIT} (hr^{-1})	0.004	0.004
Denitrification reaction rate K_{DNT} (hr^{-1})	0.055	0.055
TSS wetland background concentration TSS_o (mg/L)	3	3
NH_4^+ wetland background concentration $NH4_o$ (mg/L)	0	0
NO_3^- wetland background concentration $NO3_o$ (mg/L)	0	0
DO initial concentration in wetland DO_o (mg/L)	2	2
Influent DO concentration DO_{in} (mg/L)	2	2
Influent TSS concentration TSS_{in} (mg/L)	59.5	59.5
Influent NH_4^+ concentration $NH4_{in}$ (mg/L)	22.3	22.3
Influent NH_3^- concentration $NO3_{in}$ (mg/L)	0	0
Wetland perimeter (ft)	17,404	17,404
Number of wetland habitat types	2	2
Number of habitat islands	0	0
Goal high-marsh design depth (ft)	2	2
Goal low-marsh design depth (ft)	4	4

Table 7-6 Municipal wastewater treatment wetland calibration trials and corresponding results.

Trial	Change made	Change rationale	Mean annual rainfall (in.)	Mean annual ET (in.)	Daily mean effluent TSS conc. (mg/L)	Daily mean effluent DO conc. (mg/L)	Daily mean effluent NH_4^+ conc. (mg/L)	Daily mean effluent NO_3^- conc. (mg/L)
1	---	---	43.4	31.0	3.00	2.01	14.7	0.11
2	TSS particle diameter decreased from $6.5x10^{-6}$ m to $6.5 x10^{-7}$ m	Increase effluent TSS concentrations	43.4	31.0	18.8	2.01	14.7	0.11
3	TSS particle diameter increased from $6.5x10^{-7}$ m to $9.5 x10^{-7}$ m	Decrease effluent TSS concentrations	43.4	31.0	5.69	2.01	14.7	0.11
4	TSS particle diameter increased from $9.5x10^{-7}$ m to $8.5 x10^{-7}$ m	Increase effluent TSS concentrations	43.4	31.0	8.82	2.01	14.7	0.11
5	TSS particle diameter increased from $8.5x10^{-7}$ m to $8.15 x10^{-7}$ m	Increase effluent TSS concentrations	43.4	31.0	10.2	2.01	14.7	0.11
5	K_{NIT} increased from 0.004 to 0.04 hr^{-1}	Decrease effluent NH_4^+ concentrations	43.4	31.0	10.2	1.99	14.7	0.11
6	K_{NIT} decreased from 0.04 to 0.4 hr^{-1}	Decrease effluent NH_4^+ concentrations	43.4	31.0	10.2	1.87	14.7	0.11

7.4 TREATMENT WETLAND PERFOMANCE EVALUATION

Once calibration was complete for the municipal wastewater treatment wetland, the corresponding performance criteria and metrics were computed. The flood control (see Section 3.4.2) and downstream hydrologic regime (see Section 3.4.3) performance criteria were not relevant the municipal wastewater wetland because it was fed by wastewater flow rather than runoff from a drainage area. Therefore, a total of 11 performance criteria (see Table 7-7) were used to evaluate the performance of the treatment wetland designed in the current section. Of these 11 performance criteria, three evaluated water quality performance, two evaluated wildlife habitat within the wetland, one evaluated groundwater recharge and baseflow maintenance, two evaluated the wetland water levels, and three evaluated the aesthetic appeal of the wetland.

7.4.1.1 Treatment wetland performance criteria and metrics

While all PC values evaluating wetland performance with respect to wildlife habitat, wetland water balance, groundwater recharge and baseflow maintenance, and aesthetics were computed for the treatment wetland in the same manner as they were for the stormwater wetland in Section 7.4, the treatment wetland designed in the current section was evaluated with different water quality PC values and corresponding metrics (see Section 3.4.7.2). The water quality PC and metric values used to evaluate the treatment wetland were designed based on USEPA secondary treatment effluent water quality requirements and goals for municipal wastewater (USEPA 1985; USEPA 2000). A total of three municipal wastewater PC values were developed in Section 3.4.7.2, which included (1) the average 30-day mean wetland effluent TSS concentration (mg/L) TSS_{30}, (2) the average 7-day mean wetland effluent TSS concentration (mg/L) TSS_7, and (3) the average 30-day mean wetland effluent TKN concentration (mg/L) TKN_{30}. The treatment wetland returned respective TSS_{30}, TSS_7, and TKN_{30} concentrations of 10.2, 10.2, and 23.4 mg/L over the 25-year simulation period. Because influent TSS concentrations were assumed constant over the simulation period, a significant difference was not observed between TSS_{30} and TSS_7.

Once all three municipal wastewater PC values were computed, the corresponding metrics $M_{T(30)}$, $M_{T(7)}$, and M_{TKN} were defined based on Equations 3-36, 3-37, and 3-38:

$$M_{T(30)} = 1 \tag{7-19}$$

$$M_{T(7)} = 1 \tag{7-20}$$

$$\begin{aligned} M_{TKN} &= 1.27 - 0.0270 \cdot TKN_{30} \\ &= 1.27 - 0.0270 \cdot (23.4 \text{ mg/L}) \\ &= 0.639 \end{aligned} \tag{7-21}$$

based on the resulting $M_{T(30)}$ and $M_{T(7)}$ values, it was concluded that the treatment wetland designed in the current study met both 30-day and 7-day mean concentration requirements of 30 and 45 mg/L. Additionally, the effluent TKN_{30} concentration of 23.4 mg/L was greater than double that of the ambitious USEPA (2000) goal of 10 mg/L as reflected by a M_{TKN} of 0.639.

Table 7-7 All computed performance criteria (PC) values for the municipal wastewater treatment wetland designed in the current section.

Performance criterion	Base design
30-day mean TSS conc. (mg/L)	10.2
7-day mean TSS conc. (mg/L)	10.2
30-day mean TKN conc. (mg/L)	23.4
Vegetative cover PC	0.78
Habitat Island PC	0.000
High-marsh PC	0.966
Low-marsh PC	0.979
GW Recharge PC	0
Wetland Perimeter PC (ft)	2.53
Wetland Diversity PC	2
Wetland Area PC (acres)	86.7

7.4.1.2 Final WSI computations

Once all metrics were computed for the treatment wetland, two weighting schemes were used to evaluate the overall Wetland Sustainability Index (WSI) of the design. The first scheme equally weighted all 11 computed metrics while the second focused on those metrics pertaining to wetland water quality and water levels. The second weighting scheme was referred to as the Wastewater Treatment Plant (WWTP) weighting scheme, which only included the three water quality metrics and the two wetland water level metrics. The WWTP weighting scheme, therefore, was most concerned with wetland water quality performance as well as the maintenance of appropriate water levels. All resulting metrics, weights, and final WSI scores are summarized in Table 7-8.

The final WSI scores for the equal and WWTP weighting schemes were respectively 0.686 and 0.928. These scores show that while the treatment wetland met WWTP requirements very well, it did not perform as well at providing wildlife habitat and aesthetic appeal. Improved wildlife habitat and wetland aesthetics could have been improved by adding waterfowl habitat islands, adding more emergent vegetated areas, increasing wetland perimeter irregularity, and by incorporating different wetland types into the design. A number of these changes, however, may negatively affect the treatment wetland water quality performance or even pose a threat to wildlife health. Therefore, any changes involving wildlife habitat should be made cautiously and with wildlife nutrient and pollutant generation and tolerances in mind.

Despite these results, the treatment wetland performed worse with respect to TKN remove with a raw metric value of 0.639. This low TKN score reflects the high 30-day effluent TKN average concentration of 23.4 mg/L with respect to the optimistic goal of 10 mg/L. All other water quality metrics were equal to 1.0, which suggested that, in general, the treatment wetland performed reasonably well despite the high TKN effluent concentrations that are indicative of treatment wetlands due to their anaerobic nature (USEPA 2000). Therefore, the treatment wetland design was successful with respect to its intended WWTP design purpose as evidenced by the high resulting WWTP-weight WIS score of 0.928.

Table 7-8 Computed metrics, weights and final Wetland Sustainability Indices (WSI's) resulting from two different metric weighting schemes.

Performance criterion	Raw metrics	Weights	
		Equally-weighted	**WWTP-weighted**
30-day TSS conc. (mg/L)	1.00	0.0909	0.2
7-day TSS conc. (mg/L)	1.00	0.0909	0.2
30-day TKN conc. (mg/L)	0.639	0.0909	0.2
Vegetative cover PC	0.001	0.0909	0
Habitat Island PC	0	0.0909	0
High-marsh PC	1.00	0.0909	0.2
Low-marsh PC	1.00	0.0909	0.2
GW Recharge PC	0.00	0.0909	0
Wetland Perimeter PC (ft)	0.505	0.0909	0
Wetland Diversity PC	0.400	0.0909	0
Wetland Area PC (acres)	1.00	0.0909	0
Final *WSI* score	---	0.595	0.928

7.4.2 Effects of water conservation on performance

The sensitivity of the treatment wetland performance to water conservation was assessed in the current section. The effect of water conservation on the treatment wetland performance was estimated by reducing influent discharge rates by 29.9%. This percentage was based on the estimated residential per capita reduction in water use in gallons per day due to water conservation practices such as the use of more efficient appliances (i.e., toilets, washers, and faucets) and the repair of leaky pipes (Viessman et al. 2009). While this value of 29.9% percent only represents water conservation in residential households, the current study applied it to the whole area being served by the municipal wastewater treatment wetland under the assumption that these water conservation methods were not already in use. The purpose of this reduction in influent discharge was to determine if water conservation could significantly improve treatment wetland performance. This water conservation wetland design was referred to as design EPA1. All influent water quality concentrations for the design EPA1 were set equal to those of the base treatment wetland design, which was referred to as design EPA. Influent concentrations were not changed under the assumption that the increased population would generate the same amount of waste on a per capita basis. For the purpose of comparing the EPA and EPA1 designs, the resulting WWTP PC values, and corresponding S_X and D_X were computed and compiled in Table 7-9.

As expected, effluent TSS and TKN concentrations decreased as a result of water conservation in the service area. Resulting TSS_{30} and TSS_7 concentrations produced by the EPA1 design were both 5.13 mg/L as compared to 10.2 mg/L for the EPA design. Similarly, the TKN_{30} concentration decreased from 23.4 mg/L in the EPA design down to 19.5 mg/L in the EPA1 design. These decreases in TSS_{30}, TSS_7, and TKN_{30} were also found to be significant with respective corresponding S_x values of 1.66, 1.66, and 0.553. The S_x corresponding to TKN_{30} was less than half that of the TSS concentrations, which reflected the limited nitrification in the wetland due to its largely anaerobic conditions. Based on this significantly improved water quality performance observed in the EPA1 design, the decreased influent discharge rates and volumes allowed for slower velocities through the wetland and, therefore, greater overall wetland retention time.

Wetland water depths also decreased in the EPA1 design due to the reduced inflow volume to the wetland as evidenced by the increased high-marsh and low-marsh PC values in the EPA1 design. The effects of water conservation on wetland water depths, however, were not as strong as those on effluent pollutant concentrations as evidenced by the respective high-marsh and low-marsh S_x values of -0.154 and -0.0898. Therefore, water conservation appeared to significantly improve effluent TSS_{30}, TSS_7, TKN_{30} concentrations and slightly decrease wetland water depths. These results suggest that water conservation may serve to improve the efficiency and overall performance of wastewater treatment wetlands and most likely all WWTPs regardless of the facilities used.

Table 7-9 Relevant PC values, relative sensitivities, and deviation sensitive for the EPA and EPA1 wetland designs. The relationship direction indicates if the PC value decreases (+) or increases (-) with the decreased influent discharge rates associated with an increase in water conservation within the WWTP service area.

Performance Criteria	EPA	EPA1	S_x	D_x	Relationship direction
30-day TSS conc. (mg/L)	10.2	5.13	1.66	-5.05	+
7-day TSS conc. (mg/L)	10.2	5.13	1.66	-5.05	+
30-day TKN conc. (mg/L)	23.4	19.5	0.553	-3.86	+
High-marsh PC	0.966	1.010	-0.154	4.45E-02	-
Low-marsh PC	0.979	1.005	-0.0898	2.62E-02	-

7.4.3 Effects of population growth on performance

A second simulation was made to demonstrate the effect of possible increased population in the treatment wetland service area. According to the Maryland Department of Planning (MDP), the Maryland population is estimated to grow by 23.6% from the last census results taken in 2010 to 2040 (MDP 2014). Therefore, the current study wanted to validate that the design treatment wetland would be able to perform sufficiently given such an increase in population and corresponding water use. For simplicity, population growth was assumed to be linearly related to area water consumption and subsequent wetland influent discharge rates. While the relationship between population and water is very complex and based on a number of factors including land use, household sizes, etc., the current study assumed a simple linear relationship was sufficient to show the general effect of population growth. This treatment wetland design, which treated 23.6% more primary-treated wastewater, was referred to as design EPA2. All resulting PC values and sensitivities are shown in Table 7-10.

Population growth in the service area resulted in poorer treatment wetland water quality performance and deeper water depths in the EPA2 design due to increased internal wetland velocities and subsequent decreased wetland retention time. The resulting TSS_{30}, TSS_{7}, TKN_{30} S_x values were 1.59, 1.59, and 0.335, which reflected in increased TSS_{30}, TSS_{7}, TKN_{30} concentrations of 14.0, 14.0, and 25.2 mg/L. Similarly, the high-marsh and low-marsh depths increased with the increased service area population in the EPA2 design with respective S_x values of -0.124 and -0.0748. Based on these results, an increase in population appears to have a significant effect on the treatment wetland performance, which suggested that such factors should be accounted for in any treatment wetland design.

Table 7-10 Relevant PC values, relative sensitivities, and deviation sensitive for the EPA and EPA2 wetland designs. The relationship direction indicates where the PC value increases (+) or decreases (-) with a corresponding an increase in service area population.

Performance Criteria	**EPA**	**EPA2**	S_x	D_x	**Relationship direction**
30-day TSS conc. (mg/L)	10.19	14.0	1.59	3.84	+
7-day TSS conc. (mg/L)	10.18	14.0	1.59	3.84	+
30-day TKN conc. (mg/L)	23.4	25.2	0.335	1.85	+
High-marsh PC	0.966	0.937	-0.124	-0.0282	-
Low-marsh PC	0.979	0.961	-0.0748	-0.0173	-

7.4.4 Effects of population growth and water conservation on performance

Finally, the water conservation applied in design EPA1 was applied to design EPA2 in order to assess the importance of water conservation as populations increase. The goal of this EPA3 design was to determine if water conservation could help to counteract the negative effects of population growth on treatment wetland performance observed in the EPA2 design. Final relevant PC values and corresponding sensitivities for the EPA3 design are summarized in Table 7-11.

In the case of the EPA3 design, the positive effects of water conservation slightly dominated over the effects of population growth, which was expected as water conservation was estimated to account for 29.9% of influent water while population growth was estimated to increase inflow by 23.6%. Therefore, the EPA3 design produced slightly lower TSS_{30}, TSS_{7}, TKN_{30} concentrations than the EPA design as evidenced by their corresponding S_x values of 0.572, 0.572, and 0.149. Similarly, water depths in the EPA3 design were slightly shallower than those in the EPA design with high-marsh and low-marsh PC S_x values of -0.0474 and -0.0282. Therefore, while the water conservation and population figures are rough estimates, they show that water conservation may be useful tool in extending treatment wetland and WWTP usefulness despite increased population.

Table 7-11 Relevant PC values, relative sensitivities, and deviation sensitive for the EPA and EPA3 wetland designs. The relationship direction indicates where the PC value decreases (+) or increases (-) with the decreased influent discharge rates associated with increases in service area population and increase in water conservation.

Performance Criteria	EPA	EPA3	S_x	D_x	Relationship direction
30-day TSS conc. (mg/L)	10.19	9.4	0.572	-0.779	+
7-day TSS conc. (mg/L)	10.18	9.40	0.572	-0.779	+
30-day TKN conc. (mg/L)	23.4	22.91	0.149	-0.464	+
High-marsh PC	0.966	0.972	-0.0474	0.0061	-
Low-marsh PC	0.979	0.982	-0.0282	0.0037	-

7.4.5 EPA, EPA1, EPA2, and EPA3 design WSI scores

The experiments on the EPA treatment wetland design revealed that both water conservation and service area population growth may have a significant effect on treatment wetland performance. Treatment wetlands are designed to be more natural and less controlled methods of secondary treatment, which allows for greater energy savings, but also allows for less operator adjustment as service area needs change and grow. Therefore, such factors as population growth and water conservation may have a greater impact on treatment wetland performance than traditional, more controlled wastewater facilities. Given this possible vulnerability of treatment wetlands, it is necessary to understand ways that they react to service area water use changes. The current section aimed to evaluate such effects through the development of the EPA1, EPA2, and EPA3 designs. While both service area population and water conservation were found to affect effluent TSS and TKN concentrations significantly, these changes did not significantly change the final design WSI scores, which were respectively 0.928, 0.948, 0.918, and 0.930 for the EPA, EPA1, EPA2, and EPA3 designs (see Table 7-12). Based on these resulting

WSI scores, treatment wetlands designed with sufficiently large areas may be fairly resilient to population growth.

Table 7-12 Resulting normalized metrics and final wetland sustainability indices (WSI's) for the EPA, EPA1, EPA2, and EPA3 treatment wetland designs. All WSI scores were computed assuming equal weights for all water quality and hydrologic metrics.

Performance Criteria	**Metric weights**	**EPA**	**EPA1**	**EPA2**	**EAP3**
30-day TSS conc. (mg/L)	0.20	1.00	1.00	1.00	1.00
7-day TSS conc. (mg/L)	0.20	1.00	1.00	1.00	1.00
30-day TKN conc. (mg/L)	0.20	0.639	0.743	0.589	0.652
High-marsh PC	0.20	1.00	0.996	0.999	1.00
Low-marsh PC	0.20	1.00	0.999	1.000	1.00
Final *WSI* score	---	0.928	0.948	0.918	0.930

While it was shown that the model could be used to evaluate wastewater treatment wetland performance, it should be noted that additional water quality constituents and processes may be required to more accurately simulate their water quality performance. Currently, the model does not simulate BOD or organic nitrogen, both of which are important constituents in wastewater. High BOD concentrations restrict nitrification of NH_4^+ and deplete dissolved oxygen levels. Organic nitrogen can also be converted into NH_4^+ via the process of ammonification. While the exclusion of these processes can be compensated in part by the calibration of K_{NIT}, it would be beneficial to include both BOD and organic nitrogen processes in future versions of the model given that they are both environmentally important constituents.

Chapter 8: NRCS Agricultural Wastewater Wetland Design

8.1 DESIGN EXAMPLE

NRCS (2002) provided procedures for the design of agricultural wastewater treatment wetlands using both the presumptive and field test design methods discussed in Sections 2.2.3.2 and 2.2.3.3. The current study followed the NRCS (2002) example, which used the field test method to design a wetland intended to treat a confined swine finishing operation with 11,500 pigs, each weighing an average of 135-lbs. Within this example, the influent wastewater to the wetland was pretreated with an anaerobic waste treatment lagoon, which reduced TN concentrations by 80%. The wetland effluent was also intended for irrigation of cropland located at the same site. Therefore, NRCS (2002) designed the treatment wetland to produce effluent total nitrogen (TN) concentrations sufficient for the fertilization of the receiving cropland. All site specifications, as defined within NRCS (2002), are given in Table 8-1.

Table 8-1 Site specifications given for the wetland design example in NRCS (2002).

Parameter	Value
Annual volume of wastewater discharged to the wetland from the treatment lagoon	1,852,800ft^3/yr
Cropland available for wastewater application	80 acres
Crop requirement for TN	150 lb-TN/ac/yr
TN application losses from sprinkler irrigation	25%
Losses of TN through storage leaching during non-operational months	5%
Storage for effluent from wetland (results in an additional 10% TN loss)	45-day storage pond
Wetland porosity	0.90
Pretreatment effluent TN concentration	412 mg/L
Wetland water depth	8 in.
Average wetland temperature during operational months (April-October)	22.5°C
t_{CW} (April-October)	210 days

An agricultural swine wastewater treatment wetland was designed with the example using the field test method outlined by NRCS (2002), which is also presented and discussed in 2.2.3.3. The resulting procedure for the example wetland defined by NRCS (2002) is shown below:

1. Estimate the average daily Q_d (ft^3/d and gal/d) and annual Q_a (ft^3/yr) pre-treatment effluent volumes (NRCS 2002):

$$\begin{aligned} Q_a &= 1{,}852{,}800\,\frac{\text{ft}^3}{\text{yr}} = 1{,}852{,}800\,\frac{\text{ft}^3}{\text{yr}} \times \left(\frac{1\text{ yr}}{365\text{ d}}\right) = 5{,}076\,\frac{\text{ft}^3}{\text{d}} \\ &= 5{,}076\,\frac{\text{ft}^3}{\text{d}} \times \left(\frac{7.48\text{ gal}}{\text{ft}^3}\right) = 37{,}970\,\frac{\text{gal}}{\text{d}} \end{aligned} \tag{8-1}$$

2. Estimate the average daily TN_d (lb-TN/d) and annual TN_a (lb-TN/yr) pre-treatment effluent total nitrogen TN loads (NRCS 2002):

$$\begin{aligned} TN_d &= \left(\frac{3.79\text{ L}}{\text{gal}}\right)\left(\frac{2.2E-6\text{ lb}}{\text{mg}}\right) Q_d \cdot TN_i \\ &= \left(\frac{3.79\text{ L}}{\text{gal}}\right)\left(\frac{2.2E-6\text{ lb}}{\text{mg}}\right) \times 37{,}970\,\frac{\text{gal}}{\text{d}} \times 412\,\frac{\text{mg}}{\text{L}} \\ &= 130.4\text{ lb}-\text{TN/d} \end{aligned} \tag{8-2}$$

$$\begin{aligned} TN_a &= 130.4\,\frac{\text{lb}-\text{TN}}{\text{yr}} \times \left(\frac{365\text{ d}}{\text{yr}}\right) \\ &= 47{,}609\text{ lb}-\text{TN/yr} \end{aligned} \tag{8-3}$$

3. Determine the cropland area (acres) required to utilize pre-treatment effluent total TN loads (NRCS 2002):

$$A_R = \frac{TN_a}{\left(\dfrac{\text{Crop TN Requirment}}{\text{Fraction of TN after losses}}\right)} = \frac{47{,}609\text{ lb}-\text{TN/yr}}{\left(\dfrac{150\text{ lb - TN/yr/ac}}{(0.75)(0.95)}\right)} = 226\text{ ac} \tag{8-4}$$

$$226\text{ ac} < 80\text{ ac} \quad \therefore \text{Constructed wetland required}$$

4. Estimate the daily total N_i (lb-TN/d) required for the available cropland (NRCS 2002):

$$\begin{aligned} N_i &= \frac{\text{Available cropland} \times \text{Crop TN requirement}}{365\ \text{d/yr} \times (\text{fraction of TN after losses})} \\ &= \frac{80\ \text{ac} \times 150\ \text{lb-TN/yr}}{365\ \text{d/yr} \times 0.75 \times 0.05} = 51.3\ \text{lb} - \text{TN/d} \end{aligned} \tag{8-5}$$

5. Calculate the average daily total TN effluent concentration C_e (mg/L) required from the wetland in order to provide sufficient TN to the receiving cropland (NRCS 2002):

$$\begin{aligned} C_e &= \frac{N_i}{Q_d} \times \left(\frac{1\ \text{gal}}{3.79\ \text{L}}\right)\left(\frac{\text{mg}}{2.2E-6\ \text{lb}}\right) \\ &= \frac{51.3\ \text{lb} - \text{TN/d}}{37{,}970\ \text{gal/d}} \times \left(\frac{1\ \text{gal}}{3.79\ \text{L}}\right)\left(\frac{\text{mg}}{2.2E-6\ \text{lb}}\right) = 162\ \text{mg/L} \end{aligned} \tag{8-6}$$

6. Calculate the nitrogen reaction rate k_T (m/yr) based on the annual average operating temperature T (°C) of the wetland (NRCS 2002):

$$k_T = k_{20}\theta^{T-20} = (14\ \text{m/yr}) \times (1.06)^{22.5-20} = 16.2\ \text{m/yr} \tag{8-7}$$

7. Calculate the resulting wetland surface area SA (ac) based on the total number of days of operation t_{CW} over a given year (NRCS 2002):

$$\begin{aligned} \text{S}A &= -(0.305)\frac{1\ \text{ac}}{43{,}560\ \text{ft}^2}\left(\frac{Q_a}{k_T}\right)\ln\left[\frac{(C_e - C^*)}{(TN_i - C^*)}\right]\frac{365\ \text{d}}{t_{CW}} \\ &= -(0.305)\frac{1\ \text{ac}}{43{,}560\ \text{ft}^2}\left(\frac{1{,}852{,}800\ \text{ft}^3}{16.2}\right)\ln\left[\frac{(162\ \text{mg/L} - 10\ \text{mg/L})}{(412\ \text{mg/L} - 10\ \text{mg/L})}\right]\frac{365\ \text{d}}{210\ \text{d}} \\ &= 1.35\ \text{ac}\ \ (58{,}806\ \text{ft}^2) \end{aligned} \tag{8-8}$$

8. Calculate the estimated wetland hydraulic detention time in days (NRCS 2002):

$$t_d = (SA)\times D\times\frac{n}{Q_d} = 58{,}806\ \mathrm{ft}^2\times 0.67\ \mathrm{ft}\times\frac{0.9}{5{,}076\ \mathrm{ft}^2/\mathrm{d}} = 6.95\ \mathrm{d} \tag{8-9}$$

$$6.95\mathrm{d} > \text{minimum required } t_d \text{ of } 6 \text{ days}, \therefore \text{design is ok}$$

9. Calculate required winter storage volume (ft^3) (NRCS 2002):

$$\text{Winter storage volume} = (365\ \mathrm{d} - 210\ \mathrm{d})\times 5{,}067\ \mathrm{ft}^3/\mathrm{d} = 785{,}385\ \mathrm{ft}^3 \tag{8-10}$$

8.2 FINAL AGRICULTRUAL WASTEWATER WETLAND DESIGN

The current section interpreted all qualitative NRCS (2002) wetland design guidelines in order to develop a final agricultural wastewater treatment wetland design. NRCS (2002) suggested an overall wetland L:W ratio range of 1:1 to 4:1. Therefore, the current study used the maximum L:W ratio of 4:1. The wetland bottom was made flat as to avoid water depth problems cited to occur with long, sloped wetlands (NRCS 2002). The resulting wetland dimensions were then computed accordingly:

$$\begin{aligned} SA &= L\times W = (4W)\times W = 4W^2 \\ W &= \sqrt{\frac{SA}{4}} = \sqrt{\frac{58{,}806\ \mathrm{ft}^2}{4}} = 121.2\ \mathrm{ft} \\ L &= 485\ \mathrm{ft} \end{aligned} \tag{8-11}$$

where L and W represent the total wetland length and width (ft). As suggested by NRCS (2002), the wetland design was divided into two parallel cell trains. A deep zone was also incorporated at the outlet of the wetland to ensure proper suction of the outlet drain despite its flat bottom (NRCS 2002). Specifications were not given for this deep water zone, neither was it clear if this zone should be incorporated into the design wetland surface area or if it should be added on to the original area. The current study assumed that this deep water zone was included as part of the wetland area and was estimated to have initial dimensions of 121.2 ft by 60.6 ft, comprising about 12.5% of the total wetland area. In order to maintain wetland depths of 8 in., the outlet invert elevation was placed at the design water level. NRCS (2002) discussed the use of a number of outlet structure designs including a slotted pipe across the width of the wetland, a flashboard dam, and a swiveling elbow pipe. The current study simulated a slotted pipe across the width of the wetland by using an outlet weir with a length equal to that of the wetland train width. A bottom liner was also input into the model in order to avoid infiltration of the wastewater into groundwater via an input bottom media hydraulic conductivity of 0 ft/d.

In order to input the resulting NRCS agricultural wastewater treatment wetland design into the model, flow through only one cell train was simulated, as was computed for the EPA wastewater treatment wetland. Influent flows were divided by two before input into the cell train within the model. Similarly, all output loads and volumes were multiplied by two in order to obtain to output from the entire wetland, which consists of two parallel trains.

From the design procedure outlined in the previous section, one wetland train had dimensions of 485 ft by 60.6 ft and was divided into eight cells in series, each with dimensions of 60.6 ft by 60.6 ft. The final design wetland area was then 58,782 ft^2. Each wetland train had a L:W ratio of 8:1 while the entire wetland maintained the original design L:W ratio of 4:1. This high L:W of 8:1 in each of the wetland trains was acceptable as NRCS (2002) cited that wetlands with train L:W ratios as high as 20:1 have been used in practice successfully. All cells except the outlet cell had water depths of 8 in. (0.667 ft.) and emergent vegetation. The outlet cell was assumed to be a deepwater zone with a design depth of 4 ft and without vegetation. A forebay was not included in this wetland design as pretreatment was assumed to occur in upstream facilities. Figure 8-1 shows the resulting NRCS swine wastewater wetland design input to the model. Additionally, all cell specifications are shown in Table 8-2.

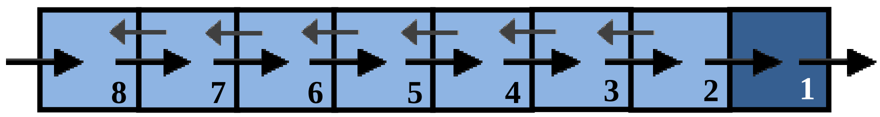

Figure 8-1 Final NRCS swine wastewater wetland design for one train input into the model. Each cell has dimensions of 60.6 x 60.6 ft. Black arrows show FID flowpath while grey arrows show FID2 flowpath.

Table 8-2 Agricultural wastewater treatment wetland cell specifications for FID1, FID2, vegetation type (VEG), initial design depth in ft SS, and cell elevation above a datum EL in ft. Vegetation descriptor values of 0, 1, 2 indicate respectively that a given cell has no vegetation, emergent vegetation, and submerged vegetation.

Cell	FID	FID2	SS (ft)	VEG	EL (ft)	BERM (ft)
1	0	1	4	0	0	0
2	1	3	0.667	1	3.33	0
3	2	4	0.667	1	3.33	0
4	3	5	0.667	1	3.33	0
5	4	6	0.667	1	3.33	0
6	5	7	0.667	1	3.33	0
7	6	8	0.667	1	3.33	0
8	7	8	0.667	1	3.33	0

8.3 AGRICULTRUAL WETLAND CALIBRATION

Before the model simulation of the agricultural wastewater treatment wetland was evaluated, it was calibrated based both on the NRCS (2002) example influent and effluent TN concentrations of 412 and 162 mg/L (see Section 8.1) and water quality data from 19 swine wastewater treatment wetlands reported by Knight et al. (2000) (see Table 2-13). From the water quality data reported by Knight et al. (2000), the current study estimated influent and effluent TSS concentrations of 128 and 62 mg/L. Additionally, influent and effluent NH_4^+:TN ratios of 0.90 and 0.89 were computed using the NH_4^+ and TN data and multiplied by the NRCS (2002) example TN influent and effluent concentrations of 412 and 162 mg/L in order to estimate model influent and effluent NH_4^+ concentrations of 370 and 144 mg/L. While large errors were associated with estimated relationship between NH_4^+ and TN concentrations, the current study only aimed to evaluate how the wetland performed with respect to changes made to the design rather than predict wetland outputs. All initial and final input values for the base agricultural wetland design are shown in Table 8-3. Results from each calibration trial are also summarized in Table 8-4.

While the calibration succeeded in simulating the appropriate wetland TSS removal with a mean daily effluent TSS concentration of 60.9 mg/L, it was not successful in simulating the NRCS (2002) design effluent TN concentration of 162 mg/L. Even with a high input K_{NIT} value of 0.4 hr^{-1}, the model simulated a mean daily effluent NH_4^+ concentration of 293 mg/L, which corresponded to a mean daily effluent TN concentration of 325 mg/L. This limited NH_4^+ removal via nitrification within the wetland was due to the low DO levels associated with the agricultural wetland design. Within the model, nitrification was only simulated in a cell if the DO concentration was greater or equal to 2 mg/L because it was an aerobic process (USEPA 2000). The daily mean effluent DO concentrations in the final calibrated design was 2.01 mg/L, which indicated that DO was the limiting factor in wetland nitrification.

A number of design attributes contributed to the low DO levels simulated within the agricultural wetland. Influent water to the wetland, which was from an anaerobic treatment lagoon, was defined in the model to have a constant DO concentration of 2 mg/L (see Table 8-3). Additionally, the agricultural wetland design did not simulate the production of DO via photosynthesis given that submerged vegetation was not included in the design. Despite the shallow water depths within the wetland, surface aeration also did not play a large factor in contributing to wetland DO levels due to the very slow velocity of water through the wetland and the constant inflow of near-anoxic wastewater.

Table 8-3 All user inputs for the agricultural wastewater treatment wetland design, their assigned initial values, and final values after calibration.

User input	Initial value	Final value
Number of years of simulation	25	25
Cell length (ft)	60.6	60.6
Contributing drainage area (ac)	---	---
Number of cells in wetland design	8	8
FID vector	See Table 8-2	---
Vegetation specification for each cell (no vegetation = 0, emergent = 1, submerged = 1)	See Table 8-2	---
Initial water depth in each cell	See Table 8-2	---
Bottom elevation in each cell	See Table 8-2	---
Berm height at exit of each cell	no berms	---
Orifice or Weir (Orifice = 1, Weir = 2)	2	2
Weir length (ft)	65	65
Orifice area (ft^2)	---	---
Weir invert height H_I (ft)	4	4
Hydraulic conductivity K_V (ft/d)	0	0
Shelter factor f_s	0.75	0.75
Wetland albedo a	0.159	0.159
Leaf area index LAI	6.5	6.5
Maximum leaf conductance C^*_{leaf} (mm/s)	9.7	9.7
Emergent vegetation height z_v (m)	1.65	1.65
Wind speed measurement height z_m (m)	2	2
Maximum photosynthesis rate $PMAX$ (mg-O_2/m^2-hr)	910	910
TSS particle diameter D (m)	6.5×10^{-6}	
Initial water temperature $T_{w(o)}$ (°C)	15.5	15.5
Nitrification reaction rate K_{NIT} (hr^{-1})	0.004	0.004
Denitrification reaction rate K_{DNT} (hr^{-1})	0.055	0.055
TSS wetland background concentration TSS_o (mg/L)	3	3
NH_4^+ wetland background concentration $NH4_o$ (mg/L)	0	0
NO_3^- wetland background concentration $NO3_o$ (mg/L)	0	0
DO initial concentration in wetland DO_o (mg/L)	2	2
Influent DO concentration DO_{in} (mg/L)	2	2
Influent TSS concentration TSS_{in} (mg/L)	128	128
Influent NH_4^+ concentration $NH4_{in}$ (mg/L)	370	370
Influent NH_3^- concentration $NO3_{in}$ (mg/L)	0	0
Wetland perimeter (ft)	1,300	1,300
Number of wetland habitat types	2	2
Number of habitat islands	0	0
Goal high-marsh design depth (ft)	0.667	0.667
Goal low-marsh design depth (ft)	4	4

Table 8-4 Agricultural wastewater treatment wetland calibration trials and corresponding results.

Trial	Change made	Change rationale	Mean annual rainfall (in.)	Mean annual ET (in.)	Daily mean effluent TSS conc. (mg/L)	Daily mean effluent DO conc. (mg/L)	Daily mean effluent NH_4^+ conc. (mg/L)	Daily mean effluent NO_3^- conc. (mg/L)
1	---	---	43.4	31.0	3.00	2.01	294	0.24
2	TSS particle diameter decreased from 6.5×10^{-6} m to 6.5×10^{-7} m	Increase effluent TSS concentrations	43.4	31.0	23.6	2.01	294	0.24
3	TSS particle diameter decreased from 6.5×10^{-7} m to 1.0×10^{-7} m	Increase effluent TSS concentrations	43.4	31.0	99.1	2.01	294	0.24
4	TSS particle diameter increased from 1.0×10^{-7} m to 3.5×10^{-7} m	Increase effluent TSS concentrations	43.4	31.0	60.9	2.01	294	0.24
5	K_{NIT} increased from 0.004 to 0.04 hr^{-1}	Decrease effluent NH_4^+ concentrations	43.4	31.0	60.9	1.68	299	0.25
6	K_{NIT} increased from 0.04 to 0.4 hr^{-1}	Decrease effluent NH_4^+ concentrations	43.4	31.0	60.9	1.20	293	0.26
7	K_{NIT} decreased from 0.4 back to 0.004 hr^{-1}	Use original K_{NIT}	43.4	31.0	60.9	2.01	294	0.24

8.4 AGRICULTRUAL WETLAND PERFORMANCE EVALUATION

Once the agricultural wetland design was calibrated, all relevant PC values and metrics were computed in order to evaluate its performance with respect to sustainability. Similar to the municipal wastewater treatment wetland designed in Chapter 7, the agricultural wetland did not treat water from a drainage area, but rather from an anaerobic treatment lagoon. Therefore, the downstream hydrologic regime and flood control metrics were not relevant to the overall performance of the agricultural wetland. Furthermore, because the effluent from the agricultural wetland was to be used for crop irrigation, effluent volume reduction was not beneficial but rather detrimental to its intended use. Given these agricultural wetland design characteristics and intended uses, only water quality, wetland water balance, wildlife habitat, and aesthetic PC values and the corresponding metrics were used to evaluate its overall WSI score. An agricultural wastewater metric weighting scheme was used to evaluate the wetland's performance within the context of its sole goal of producing appropriate effluent TN concentrations for crop irrigation. This WSI weighting scheme evenly weighted the metric that corresponded to the mean daily effluent TN concentration and those that corresponded to the high and low-marsh water depths. All resulting PC values for the agricultural wetland design are shown in Table 8-5 and final metrics and WSI scores for both weighting schemes are compiled in Table 8-6.

While all of the other metrics relevant to the agricultural wetland design performance were computed in the same manner as in Section 6.8, the water quality metric was computed differently. The agricultural wetland was designed according to the NRCS (2002) field test to produce effluent TN concentrations of 162 mg/L.

Therefore, the current study developed a TN metric M_{TN} for the agricultural wetland in Section 3.4.7.3 and was computed for the design example using Equation 3-39:

$$M_{TN} = 0 \tag{8-12}$$

Because the mean daily effluent TN concentration of 329 mg/L was greater than 324 mg/L, the resulting M_{TN} was zero which indicated that the simulated agricultural wetland design failed to produce effluent TN levels appropriate for crop irrigation and fertilization. This failure of the design was likely due to the exclusion of NH_3 volatilization and NH_4^+ nitrification at the root sites within the model.

Table 8-5 All computed performance criteria (PC) values for the agricultural wastewater treatment wetland designed in the current section.

Performance criterion	**Base design**
Mean daily effluent TN conc. (mg/L)	325
Vegetative cover PC	0.88
Habitat Island PC	0
High-marsh PC	0.994
Low-marsh PC	1.00
Wetland Perimeter PC (ft)	1.99
Wetland Diversity PC	2.00
Wetland Area PC (acres)	0.78

The final computed WSI scores for the equally weighted and agricultural wastewater weighted metrics were computed to be 0.507 and 0.667. Due to the anoxic conditions within the agricultural wetland design, it performed poorly with respect to effluent TN concentrations, producing a M_{TN} of zero. The wetland design did, however, successfully maintain design water levels as evidenced by the high and low-marsh metric values of 1.00. The large proportion of emergent vegetation in the design also promoted sufficient marsh wren habitat resulting in a vegetative cover metric of 0.979. Conversely, the wetland area and shape did promote a high aesthetic metric. Based on these results the agricultural wetland design produced effluent TN concentrations that were much higher than the goal of 162 mg/L. It was thought that this poor design performance was due to the exclusion of nitrogen removal mechanisms in the model given that a number of sources have reported the relative success of similar agricultural wetlands in reducing nitrogen concentrations in wastewater (Cronk 1996; NRCS 2002; Poach et al. 2004).

Table 8-6 Computed metrics, weights and final Wetland Sustainability Indices (WSI's) resulting from two different metric weighting schemes.

Performance criterion	**Raw metrics**	**Weights**	
		Equally-weighted	**Agricultural WW-weighted**
Mean daily effluent TN conc. (mg/L)	0.000	0.125	0.333
Vegetative cover PC	0.979	0.125	0
Habitat Island PC	0	0.125	0
High-marsh PC	1.00	0.125	0.333
Low-marsh PC	1.00	0.125	0.333
Wetland Perimeter PC (ft)	0.399	0.125	0
Wetland Diversity PC	0.600	0.125	0
Wetland Area PC (acres)	0.0776	0.125	0
Final *WSI* score	---	0.507	0.667

Given these model results two additional NH_3/ NH_4^+ removal mechanisms were incorporated into the model in order to evaluate their effect on effluent NH_4^+ concentrations in the agricultural wastewater wetland design. These mechanisms were (1) nitrification that occurred at the roots of emergent vegetation and (2) NH_3 volatilization. While a number of sources considered these two mechanisms negligible (Kadlec and Knight 1996; USEPA 2000), NRCS (2002) considered them to be the main mechanisms of agricultural wetland NH_3/ NH_4^+ removal due to the high relative organic-N and NH_3/ NH_4^+ entering the wetland. These two removal mechanisms are also often excluded from wetland models due to the lack of data in the literature surrounding their behavior (USEPA 2000). The following sections evaluate the effects of these two removal mechanisms and discuss how different mechanisms may dominate in different wetland designs.

8.5 EFFECT OF PLANT OXYGEN TRANSFER IN RHIZOSPHERE

Emergent vegetation have adapted to survive in anaerobic conditions such as those exhibited in the agricultural wastewater wetland designed in the current study by developing air pathways that transfer oxygen from the atmosphere to their roots. This oxygen is used by the plants for respiration and is referred to as plant oxygen transfer. A number of studies have cited that this transfer of oxygen to plant roots produces aerobic conditions in the surrounding soil, which is referred to the rhizosphere (Kadlec and Knight 1996; Brix 1997; USEPA 2000; NRCS 2002; Bastviken 2006). While their significance is subject to controversy (USEPA 2000), the aerobic zones produced by plant oxygen transfer have been hypothesized to promote areas of nitrification in the soil of free water surface treatment wetlands (Kadlec and Knight 1996; Brix 1997; USEPA 2000; NRCS 2002).

Plant oxygen transfer rates reported in the literature were used to estimate a mean wetland plant oxygen transfer rate of 0.264 mg-O_2/ft^2-min. Due to the difficulties associated with measuring plant oxygen transfer as well as corresponding nitrification, few studies exist in the literature that deal with the characterization of nitrification within the rhizosphere. Despite the limited data, Brix (1997) reported a literature plant oxygen transfer range of 0.02 to 12 g O_2/m^2-d (0.00129 – 0.775 mg O_2/ft^2-min) for *Phragmites sp.*, which are a common emergent vegetation species used in constructed wetlands. Kadlec and Knight (1996) also reported a literature range of 0.02 to 4.3 g O_2/m^2-d (0.00129 – 0.278 mg O_2/ft^2-min) for emergent species planted in soil media.

In order to evaluate the effectiveness of plant oxygen transfer as a NH_4^+ removal mechanism in the agricultural wastewater wetland design, a plant oxygen transfer module was added to the model. The resulting modified wetland design was referred to as the AG1 design. In the AG1 design, a constant plant oxygen transfer rate of 0.264 mg-O_2/ft^2-min was applied to the soils in cells with emergent vegetation (i.e., VEG = 1). Additionally, nitrification was simulated in the soil of these emergent vegetation cells if the load of DO produced by plant oxygen transfer met the corresponding oxygen demand of the nitrification transformation.

Flow through the AG1 design was simulated over 25 years producing a mean daily effluent NH_4^+ concentration of 136 mg/L, which corresponded to an estimated mean daily effluent TN concentration of 153 mg/L. These results greatly improved the apparent TN removal of the agricultural wastewater wetland design, which produced an initial mean daily effluent TN concentration of 239 mg/L, with a relative error E_X of -0.534. Calibration of the input plant oxygen transfer rate to a value of 0.120 mg/L further improved the apparent design performance, producing mean daily effluent TN concentrations of 163 mg/L and a corresponding TN PC value of 1.00. Given these results, it was concluded that based on literature values, plant oxygen transfer could promote significant nitrification within a wetland design including significant areas with emergent vegetation. That said, more research related to plant oxygen transfer and its relationship with nitrification in the soil is required in order to more realistically and accurately model such mechanisms.

8.6 EFFECTS OF NH_3 VOLATILIZATION

The current section developed an agricultural wetland design that incorporated NH_3 volatilization as a nitrogen removal mechanism. This wetland design was referred to as the AG2 design. As discussed in Section 0, at a circumneutral pH, only 0.6% of ammonia species are in the form of NH_3 (Kadlec and Knight 1996). Therefore, under normal wetland operating conditions, volatilization of NH_3 is generally assumed to be negligible (Kadlec and Knight 1996; USEPA 2000; Bastviken 2006). NRCS (2002), however, cited NH_3 volatilization has a major mechanism of ammonia removal in agricultural wastewater treatment wetlands due to the large TN concentrations typical of these wetland influents. Kadlec and Knight (1996) even cited that NH_3 volatilization was negligible unless ammonia concentrations in a wetland were greater than 20 mg/L. Given that the agricultural wetland design receives inflow with an estimated NH_4^+ concentration of 412 mg/L, NH_3 volatilization may be of greater relevance than in other wetland designs.

Given that the agricultural wetland promoted conditions in which NH_3 volatilization could be a significant ammonia removal mechanism, a model agricultural wetland design in which NH_3 volatilization was developed and simulated. In order to estimate NH_3 volatilization within the model, literature NH_3 volatilization rates were compiled to estimate an average wetland NH_3 volatilization rate of 0.0239 mg-NH_3^-N/ft^2-min. While wetland NH_3 volatilization data are limited, Poach et al.

(2004) reported volatilization rates ranging from 0 to 15 mg-NH_3^-N/m^2-hr (0 to 0.0581 mg-NH_3^-N/ft^2-min) for the marsh portions of a marsh-pond-marsh wetland used for the treatment of pre-treated swine wastewater. The estimated average wetland NH_3 volatilization rate of 0.0239 mg-NH_3^-N/ft^2-min was applied to each cell within the model and was assumed to be constant over the duration of the 25-yr simulation period. The resulting AG2 design mean daily effluent TN concentration was equal to 297 mg/L, which was 9.62% lower than the initial value of 329 mg/L. Increasing the NH_3 volatilization to the literature maximum of 0.0581 mg-NH_3^-N/ft^2-min, resulted in a mean daily effluent TN concentration of 262 mg/L, which was 20.4% less than the base value of 297 mg/L. Based on these results, while NH_3 volatilization did not reduce effluent TN concentrations down to the goal value of 162 mg/L, it did have a significant impact on TN concentrations. Therefore, while it did not have as large of an impact on ammonia concentrations as the rhizosphere nitrification, NH_3 volatilization may be an important nitrogen removal mechanism in wetlands that warrants more attention when modelling wetlands treating water with high (> 20 mg/L) NH_3/NH_4^+ concentrations.

8.7 COMBINED EFFECTS OF PLANT OXYGEN TRANSFER AND NH_3 VOLATILIZATION

A final agricultural wastewater wetland design, referred to as the AG3 design, was developed that incorporated both the rhiszosphere nitrification and the NH_3 volatilization mechanisms included respectively in the AG1 and AG2 designs. Additionally, the nitrification mechanism inputs were calibrated in this final design (see Table 8-7) in order to compute new agricultural wetland design PC values and corresponding metrics, which are compiled respectively in Tables 8-8 and 8-9 and Table 8-9. The resulting agricultural wastewater weighting scheme returned a WSI score of 1.00 as compared with that of the base agricultural wetland of 0.667. This significant improvement in WSI scores was due to the respective base and AG3 mean daily effluent TN concentrations of 329 and 164 mg/L. Therefore, within the context of the agricultural wastewater wetland, plant oxygen transfer and NH_3 volatilization proved to be significant contributors to wetland ammonia removal.

Table 8-7 Agricultural wastewater treatment wetland calibration trials and corresponding results for the AG3 design incorporating both NH_3 volatilization and rhizosphere nitrification.

Trial	Change made	Change rationale	Daily mean effluent NH_4^+ conc. (mg/L)	Daily mean effluent NO_3^- conc. (mg/L)	Estimated daily mean effluent TN conc. (mg/L)
1	---	---	121	1.18	136
2	Decrease plant oxygen transfer rate from 0.264 to 0.120 mg-O_2/ft^2-min	Increase effluent NH_4^+ and TN conc.	124	1.19	139
3	Decrease plant oxygen transfer rate from 0.120 to 0.0500 mg-O_2/ft^2-min	Increase effluent NH_4^+ and TN conc.	188	0.94	211
4	Increase plant oxygen transfer rate from 0.050 to 0.110mg-O_2/ft^2-min	Decrease effluent NH_4^+ and TN conc.	126	1.21	147
5	Decrease plant oxygen transfer rate from 0.110 to 0.090 mg-O_2/ft^2-min	Increase effluent NH_4^+ and TN conc.	137	1.24	154
6	Decrease plant oxygen transfer rate from 0.110 to 0.080 mg-O_2/ft^2-min	Increase effluent NH_4^+ and TN conc.	146	1.22	164

Table 8-8 All computed performance criteria (PC) values for the AG3 design.

Performance criterion	Base design
Mean daily effluent TN conc. (mg/L)	164
Vegetative cover PC	0.875
Habitat Island PC	0.00
High-marsh PC	0.994
Low-marsh PC	1.00
Wetland Perimeter PC (ft)	0.40
Wetland Diversity PC	3
Wetland Area PC (acres)	0.8

Table 8-9 Computed metrics, weights and final Wetland Sustainability Indices (WSI's) resulting from two different metric weighting schemes for the AG3 design.

Performance criterion	Raw metrics	Weights	
		Equally-weighted	**Agricultural WW-weighted**
Mean daily effluent TN conc. (mg/L)	1.00	0.125	0.333
Vegetative cover PC	0.979	0.125	0
Habitat Island PC	0	0.125	0
High-marsh PC	1.00	0.125	0.333
Low-marsh PC	1.00	0.125	0.333
Wetland Perimeter PC (ft)	0.0798	0.125	0
Wetland Diversity PC	0.120	0.125	0
Wetland Area PC (acres)	0.00776	0.125	0
Final *WSI* score	---	0.523	1.00

The overall purpose of the addition and evaluation of these two nitrogen removal mechanisms was to show that different mechanisms may dominate in different wetland designs. Additionally, mechanisms that many sources consider negligible in one case may be crucial in another. Due to the current lack of data and understanding of nitrogen behavior in constructed wetlands it is difficult to discern which mechanisms (1) are most important and (2) are documented and understood enough to reliably model. It is also worth noting that while both plant oxygen transfer and NH_3 volatilization may occur in all wetland designs, they only dominate ammonia removal in designs with large influent ammonia concentrations and low DO concentrations. Therefore, while the exclusion of these mechanisms could be compensated for in the stormwater and municipal wastewater wetland designs (see Chapters 6 and 7) through calibration of the water column nitrification rate constant K_{NIT}, calibration of K_{NIT} did not have the same effect on NH_4^+ concentrations in agricultural wetland where DO levels were insufficient to promote significant nitrification. Additionally, as shown by the current section, the model can be modified to include or exclude water quality mechanisms as more data become available to allow for more accurate model representation of corresponding wetland behavior.

It has been demonstrated that both plant oxygen transfer and NH_3 volatilization may contribute to nitrogen removal in wetlands with large influent ammonia concentrations and with anoxic water conditions. However, due to the limited knowledge of how plant oxygen transfer and NH_3 volatilization directly affect ammonia levels, it is difficult to accurately model them. Given the uncertainty surrounding these mechanisms, the current study suggests that more research be focused on the effectiveness of plant oxygen transfer and NH_3 volatilization in removing ammonia in wetland

Chapter 9: NRCS Habitat Wetland Design

9.1 DESGIN EXAMPLE

The final wetland design evaluated in the current study was a wildlife habitat wetland, which was designed according to specifications made by NRCS (2009). The guidelines for habitat wetland design defined by NRCS (2009) and throughout the literature were more qualitative and more general than those specified for wetlands designed for water treatment and management. A wetland area of 10 acres was chosen for the example design in order to meet the NRCS minimum required wetland area for habitat island incorporation. Given a wetland area of 10 acres, 4 habitat islands could be included in the design based on the placement ratio of 1 habitat islands/2.5 acres of wetland specified by NRCS (2009). According to NRCS (2009), a wetland area of 10 acres would also support 50 basking areas for reptiles such as snakes and turtles. These islands and basking areas were assumed to be incorporated into the wetland design even though they were not directly simulated by the model.

Next, NRCS (2009) guidelines for water depth allocation for waterfowl, amphibians and reptiles, wetland furbearers, and shorebirds (see Section 2.2.4) were used to develop a wetland design that incorporated habitat areas for all wildlife types of concern. NRCS (2009) wetland depth specifications are also compiled in the current section in Table 9-1. In order to satisfy the specifications defined by NRCS (2009) for waterfowl, amphibian and reptile, wetland furbearer, and shorebird needs, the wetland surface area was divided accordingly by water depth: 20% 4-ft depth, 15% 1.5-ft depth, 15% 3-ft depth, 25% 1-ft depth, and 25% 3-in depth.

Table 9-1 NRCS (2002) specifications for allocation of water depths within a wetland design for different wildlife types.

Wildlife type	Depth requirements
Waterfowl	≤ 20% depths of 3-4 ft ≤ 30% depths 1.5-3 ft Remainder area < 1.5 ft
Diving ducks	≤ 50% areas with emergent vegetation
Wading shorebirds	Seasonal mudflats with depths of 1-4 in.
Amphibians/Reptiles	≤ 20% depths of 3-5 ft > 50% depths < 1.5 ft
Wetland furbearers	≥ 20% depths 3-5 ft Remainder area < 3 ft

9.2 FINAL HABITAT WETLAND DESIGN

The current study made a number of assumptions with respect to the wetland design due to the lack of quantitative design requirements specified by NRCS (2009). While NRCS (2009) set specific area percentages of water depths for different wildlife, it did not mention design features such as relative drainage area size, total wetland storage volume, or outlet structure design. In order to promote sufficient water depths within the wetland, a contributing drainage area of 413 acres was defined for the habitat wetland. This drainage area was also assumed to have a rational C value of 0.36, which was the same as that used for the stormwater wetland design in Section 6.8. Additionally, the model incorporated an outlet weir with a length of 15 ft, which was arrived at by calibration, into the design to ensure proper internal wetland water depth maintenance as well as to promote sufficiently transfer water out of the wetland. The outlet structure design procedure for stormwater wetlands was not followed for the habitat wetland because the current study found that the contributing area of 413 acres was too large to produce rational numbers through such methods (MDE 2009). The TR-55 method discussed in Section 11.2, did not allow for sufficiently long flowpaths for a drainage area analogous to that used in Chapter 6 with an area of 413 ac.

Once the habitat wetland design specifications were defined, it was divided into cells with dimensions of 150 by 150 ft and organized as shown in Figure 9-1. The overall wetland area was slightly altered to ensure all areas of different wetland depths could be divided evenly into cells. The resulting surface areas and corresponding cell numbers are shown in Table 9-2. Table 9-3 also summarizes all cell specification for the habitat wetland design.

The habitat wetland design was not calibrated given that wildlife habitat performance as defined by NRCS (2009) and within the model was dependent on wetland design features (i.e., the inclusion of habitat islands, etc.) rather than simulated hydrologic and water quality outputs. Because the habitat wetland was also assumed to be fed with stormwater from a drainage area with the same rational C value as that of the stormwater wetland design from Section 6.8, influent water quality concentrations were assumed to be equal to those used in the stormwater wetland design. Additionally, because the habitat wetland received water from a drainage area with similar characteristics as the stormwater wetland, it was assumed that the calibrated inputs used for the stormwater wetland were reasonable estimates for those of the habitat wetland. All resulting user inputs for the habitat wetland are summarized in Table 9-4.

Table 9-2 Total area of each water depth zone within the habitat wetland design as well as the corresponding number of cells used to represent each zone.

Zone Type	Zone Water Depth (ft)	Surface Area (ft^2)	# of cells
Deep zone	4	90,000	4
Low-marsh	3	67,500	3
High-marsh	1.5	67,500	3
High-marsh	1	112,500	5
Mudflats	0.25	112,500	5
Σ	---	435,600	20

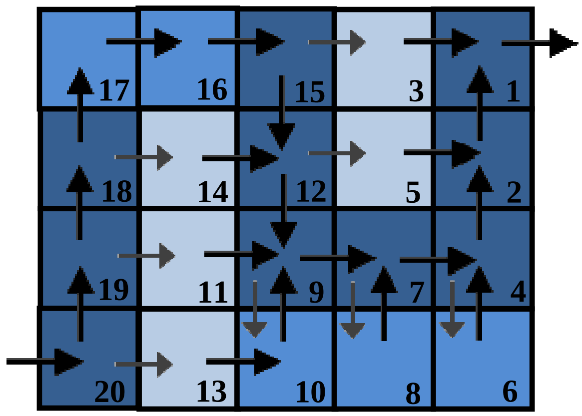

Figure 9-1 Habitat wetland cell design. Each cell has dimensions of 150 x 150 ft. Black arrows show FID flowpath while grey arrows show FID2 flowpath.

Table 9-3 Habitat wetland cell specifications for FID1, FID2, vegetation type (VEG), initial design depth in ft SS, and cell elevation above a datum EL in ft.

Cell	FID	FID2	SS (ft)	VEG	EL (ft)	BERM (ft)
1	0	1	4	2	0	0
2	1	2	3	2	1	0
3	1	3	0.25	0	3.75	0
4	2	6	1.5	1	2.5	0
5	2	5	0.25	0	3.75	0
6	4	6	1	1	3	0
7	4	8	1.5	1	2.5	0
8	7	8	1	1	3	0
9	7	10	1.5	1	2.5	0
10	9	10	1	1	3	0
11	9	11	0.25	0	3.75	0
12	9	5	3	2	1	0
13	10	13	0.25	1	3.75	0
14	12	14	0.25	1	3.75	0
15	12	3	3	2	1	0
16	15	16	1	1	3	0
17	16	17	1	1	3	0
18	17	14	4	2	0	0
19	18	11	4	2	0	0
20	19	13	4	2	0	0

Table 9-4 User inputs for the habitat wetland design

User input	Input value
Number of years of simulation	25
Cell length (ft)	150
Contributing drainage area (ac)	413
Number of cells in wetland design	20
FID vector	See Table 9-3
Vegetation specification for each cell (no vegetation = 0, emergent = 1, submerged = 1)	See Table 9-3
Initial water depth in each cell	See Table 9-3
Bottom elevation in each cell	See Table 9-3
Berm height at exit of each cell	no berms
Orifice or Weir (Orifice = 1, Weir = 2)	2
Weir length (ft)	15
Orifice area (ft^2)	---
Weir invert height H_I (ft)	4
Hydraulic conductivity K_V (ft/d)	0
Shelter factor f_s	0.75
Wetland albedo a	0.159
Leaf area index LAI	6.5
Maximum leaf conductance C^*_{leaf} (mm/s)	9.7
Emergent vegetation height z_v (m)	1.65
Wind speed measurement height z_m (m)	2
Maximum photosynthesis rate $PMAX$ (mg-O_2/m^2-hr)	910
TSS particle diameter D (m)	$6.5x10^{-6}$
Initial water temperature $T_{w(o)}$ (°C)	15.5
Nitrification reaction rate K_{NIT} (hr^{-1})	0.004
Denitrification reaction rate K_{DNT} (hr^{-1})	0.055
TSS wetland background concentration TSS_o (mg/L)	3
NH_4^+ wetland background concentration $NH4_o$ (mg/L)	0
NO_3^- wetland background concentration $NO3_o$ (mg/L)	0
DO initial concentration in wetland DO_o (mg/L)	2
Influent DO concentration DO_{in} (mg/L)	2
Influent TSS concentration TSS_{in} (mg/L)	43.4
Influent NH_4^+ concentration $NH4_{in}$ (mg/L)	0.13
Influent NH_3^- concentration $NO3_{in}$ (mg/L)	0.50
Wetland perimeter (ft)	2,700
Number of wetland habitat types	5
Number of habitat islands	4
Goal high-marsh design depth (ft)	0.667
Goal low-marsh design depth (ft)	4

9.3 HABITAT WETLAND PERFORMANCE EVALUATION

All habitat wetland PC values and corresponding metrics were computed in the same manner as they were in Section 6.8 for the stormwater wetland design and are compiled respectively in Tables 9-5 and 9-6. In order to evaluate the habitat wetland performance, three weighting schemes were developed, the first of which weighted all 18 wetland metrics equally. The second weighting scheme weighted the wildlife habitat, the wetland water balance, and the aesthetic metrics equally while assigning all other metrics a weight of zero. The third weighting scheme was the BMP-weighting scheme defined in Section 6.8 for the stormwater wetland design, which equally weighted the water quality, wetland water balance, flood control, and downstream hydrologic regime metrics. While the first weighting scheme evaluated all aspects of the habitat wetland's performance, the second weighting scheme focused only on the metrics relevant to the wildlife habitat and corresponding wetland water depths within the wetland. The third weighting scheme was evaluated for comparison with the stormwater wetland design performance.

Table 9-5 All computed performance criteria (PC) values for the habitat wetland designed in the current section.

Performance criterion	Base design
Mean daily TSS conc. (mg/L)	12.3
Mean daily DO conc. (mg/L)	10.9
Mean daily NH4 conc. (mg/L)	0.07
Mean daily NO3 conc. (mg/L)	0.279
Vegetative cover PC	0.50
Habitat island PC	1
High-marsh PC	0.981
Low-marsh PC	0.994
GW-recharge PC	0.00
Wetland perimeter PC (ft)	1.14
Wetland diversity PC	5
Wetland area PC (acres)	10.3306
High-flow PC	0.192
Low-flow PC	0.185
Flow-variation PC	3.42
Flood-control PC	0.382

Table 9-6 Computed metrics, weights and final Wetland Sustainability Indices (WSI's) resulting from three different metric weighting schemes.

Performance criterion	Raw metrics	Weights		
		Equally-weighted	**Habitat-weighted**	**BMP-weighted**
Mean daily TSS conc. (mg/L)	0.392	0.0625	0	0.100
Mean daily DO conc. (mg/L)	1	0.0625	0	0.100
Mean daily NH4 conc. (mg/L)	0.600	0.0625	0	0.100
Mean daily NO3 conc. (mg/L)	1	0.0625	0	0.100
Vegetative cover PC	0.142	0.0625	0.250	0
Habitat island PC	1	0.0625	0.250	0
High-marsh PC	1.00	0.0625	0.250	0.100
Low-marsh PC	1.00	0.0625	0.250	0.100
GW-recharge PC	0.00	0.0625	0	0
Wetland perimeter PC (ft)	0.227	0.0625	0	0
Wetland diversity PC	1.00	0.0625	0	0
Wetland area PC (acres)	1.00	0.0625	0	0
High-flow PC	0.347	0.0625	0	0.100
Low-flow PC	0.335	0.0625	0	0.100
Flow-variation PC	0.00	0.0625	0	0.100
Flood-control PC	0.619	0.0625	0	0.100
Final *WSI* score	---	0.604	0.785	0.629

Final WSI scores for the habitat wetland (see Table 9-6) were computed for the equally-weighted, the habitat-weighted, and the BMP-weighted evaluation schemes. The respective WSI scores for these weighting schemes were found to be 0.604, 0.785, and 0.629, which implied that the habitat wetland performed best with respect to providing wildlife habitat. The habitat wetland also performed similarly to the stormwater wetland design, which produced a BMP-weighted WSI score of 0.640. These results suggest that wildlife habitat wetlands provide water quality and quantity treatment and could possibly be designed to further improve stormwater water quality and hydrology in addition to providing habitat for important wetland species.

As expected, the habitat weighting scheme resulted in the largest WSI score with a value of 0.785. This habitat-weighted WSI score was lowered due to the low vegetation cover PC value of 0.142 that pertained to marsh wren habitat (see Section 3.4.1). According to Gutzwiller and Anderson (1987), marsh wrens required at least 50% of the wetland area to be covered by emergent vegetation, but thrive with higher percentages. The habitat wetland design dedicated exactly 50% of the wetland area to emergent vegetation, meeting the minimum needs of the marsh wren. While this design was not optimized for the marsh wren, it did provide sufficient habitat for all wildlife types specified as important by NRCS (2009) (see Table 9-1). Given these depth specifications, the model user could develop different wildlife habitat metrics if different wetland wildlife where of concern. Additionally, different wetland designs could be developed to optimize habitat for different wildlife types. A wetland comprised completely of areas with emergent vegetation, for example, could be designed to maximize marsh wren habitat. Similarly, a wetland design excluding areas with emergent vegetation would be optimal for diving ducks (see Table 9-1). If multiple wildlife types are desired in a given wetland design, the user must incorporate all relevant habitat types within the wetland. As with any wetland design, habitat wetlands must be designed on a case-by-case basis, with specific wildlife needs in mind.

Chapter 10: Conclusions and Recommendations

10.1 INTRODUCTION

The overall goal of the current study was to develop a spatio-temporal model with which different wetland functions that could be evaluated through the use of sustainability metrics. Included under this overall goal were five study objectives, which were (1) to formulate a spatio-temporal model of a multipurpose constructed wetland, which includes relevant processes and components of different types of wetland systems, (2) to define sustainability as it applies to wetlands and to develop metrics based on sustainability principles that connect wetland design with intended wetland functions, (3) to calibrate the model using both hydrologic and water quality data from a real wetland, (4) to quantify the sensitivity and uncertainty associated with each design function and contributing variable, and (5) to evaluate the reliability of design criteria currently used in the design of wetlands and assess whether or not they lead to sustainable designs. These five objectives were met and resulted in a constructed wetland modeling tool that is capable of evaluating and optimizing wetland designs based on stakeholder-defined sustainability metrics. The spatio-temporal model developed herein successfully simulates wetland design sensitivity to changes in design criteria and user inputs. Wetland Sustainability Indices (WSI's) were also proposed as a new decision tool and found to provide reliable indications of the sustainability of a wetland design with respect to its intended functions. The core findings of the study are summarized herein. A number of recommendations with respect to future calibration and use of the model are also identified.

10.1.1 Model Scope and Applicability

The model and sustainability metrics were developed to be used as tools for the comparison of different wetland designs with the eventual goal of wetland design optimization. It should be noted that the model is not a predictive tool, but rather a comparative tool. Output values from the model do not necessarily predict reality, but rather illustrate how changes to a model affect wetland performance. Additionally, because the model does not currently simulate BOD and organic nitrogen, it should not be used to make conclusive water quality assumptions in wastewater treatment wetlands, which are known to treat water with high BOD and organic nitrogen concentrations. The model also assumes that periodic maintenance, as is suggested in a number of design manuals (USEPA 2000; NRCS 2002; MDE 2009), is performed for any given wetland design. Therefore, processes such as sediment accretion, invasive vegetation species growth, and design vegetation death are not accounted for within the model. Ammonia toxicity to vegetation is also assumed negligible in the model, which may not be the case in wetlands that treat agricultural wastewater with NH_4^+ exceeding 200 mg/L. Therefore, when designing such wetlands with the model, caution should be taken when choosing vegetation and in interpreting results.

10.2 STUDY CONCLUSIONS

10.2.1 Formulation of a spatio-temporal model

A spatio-temporal model was developed to simulate the hydrologic and water quality processes of a constructed wetland. This model is unique in several aspects.

First, it allows the user to define a proposed or existing wetland design by dividing it into cells, which enables the simulation of water quantity and variation of water quality processes within each cell of a given wetland design. Second, the model uses a very short time increment. A 1-min time interval was chosen for the model so as to rationally move water through the wetland without producing irrationally large volumes of water moving from one cell to the next. Third, the model is process-oriented. Each wetland cell is assumed to behave as a completely mixed flow reactor (CMFR) and can have unique characteristics. Each cell is assigned a surface water depth, bottom elevation, a vegetation type, and primary and secondary flowpath designations. Additionally, infiltration, ET, surface aeration, photosynthesis, TSS settling, nitrification, and denitrification are simulated in each individual cell. Fourth, the model allows for a realistic representation of inflows from multiple sources including rain that falls directly onto the wetland, runoff from a user-defined drainage area, a primary-treated municipal wastewater, and an anaerobic treatment lagoon. Fifth, the flexible structure of the model enables the design of multiple wetland types such as stormwater wetlands, municipal wastewater treatment wetlands, agricultural wastewater treatment wetlands, and wildlife habitat wetlands. This flexibility enables the same concepts to be applied in regional analyses where different types of wetlands are needed. Given these characteristics, the model is a useful tool that can be used for the design of a wide range of wetland types with varying types of inflow and for the development of policies related to water quantity and quality control.

10.2.2 Definition of sustainability metrics

The sustainability metrics developed in Chapter 3 worked well in evaluating wetland design performance. Additionally, weighting these metrics based on the intended purpose of different wetland designs resulted in WSI scores representative of the specific goals of a given wetland design. In Section 6.9.1.8, WSI scores for five different weighting schemes were evaluated for the same stormwater wetland design. The weighting schemes were developed to compute the WSI scores with respect to (1) all wetland functions, (2) water quality functions, (3) flood control, wetland water balance, and downstream hydrologic regime functions (i.e., all relevant hydrologic functions), (4) habitat and aesthetic functions, and (5) both water quality and hydrologic functions. The resulting WSI scores for their weighting schemes were respectively 0.498, 0.714, 0.591, 0.314, and 0.640. The variation of these scores emphasizes the discriminatory power of the model when combined with alternative metrics and corresponding weights. The variation observed in these WSI scores illustrated that the definition of wetland sustainability could be adapted based on the wetland functions desired for a given wetland design. Based on these results, the sustainability metrics and their corresponding performance criteria were reliable measures of wetland sustainability with respect to different wetland functions. The flexibility of the PTM (Performance-to-Metric) functions that relate performance criteria and corresponding metrics also allowed for model users to define sustainability for a given wetland function based on the intended purpose of a given wetland design as well as the sensitivity of that function to changes.

Wetland sustainability metrics reflect a different approach to the design and evaluation of wetlands. They offer the opportunity to examine the long-term consequences of a design with respect to wetland and downstream ecosystem sustainability. Such a method of wetland design and optimization in the context of sustainability is not documented in the literature. Therefore, the model and coupled with the sustainability metrics provide a new method and perspective with which to design constructed wetlands. By designing wetlands for longevity rather than maximum, short-term performance, degraded downstream ecosystems could have more time to recover and to establish resilience and stability as effluent water properties remain fairly consistent from year to year.

10.2.3 Model calibration

A specific implementation model was successfully calibrated with respect to the hydrologic data and water quality provided by Jordan (2013). Calibration of relevant user input parameters was effective in producing wetland outputs comparable to those of corresponding observed outputs. Hydrologic calibration was especially successful, resulting in $\overline{e}/\overline{Y}$ magnitudes below 0.05 and $\overline{S}_e/\overline{S}_y$ values below 0.500. The model calibration also demonstrated that model outputs were rationally related to changes in user inputs, which indicated that the model was mathematically sound and was a reasonable representation of the hydrologic and water quality processes within an actual constructed wetland.

As part of the model calibration, the effects of cell size and flowpath definition on model results were also evaluated. It was shown that flowpath has a significant impact on both hydrologic and water quality model outputs and corresponding wetland performance. Therefore, in the design of a wetland it is crucial that wetland cell flowpaths, elevations, and water depths be prescribed with both accuracy and precision. Additionally, an experiment was run to demonstrate the effect of cell size on model performance. Smaller cells were found to more accurately predict the observed hydrologic and water quality values given that the corresponding flowpath was sufficiently designed. Therefore, wetland cells should be sized based on the precision with which the internal wetland flowpath is defined for a given wetland design.

10.2.4 Design sensitivity to changes in design criteria and inputs

The model was also successfully used to demonstrate the relative importance of changes to current design criteria defined by MDE (2009) as well as to changes in user input parameters. From these analyses, the location of wetland features such as deepwater areas and their respective retention time were identified as the factors that dominated wetland performance. Therefore, it is suggested that future constructed wetlands be designed to optimize the relative locations and retention times of all wetland areas (i.e., high marsh, low marsh, deepwater areas, etc.). The model demonstrated sufficient sensitivity to changes in internal wetland area type location and storage to facilitate such design optimization. Model performance was most sensitive to error in the water quality input parameters such as influent concentrations and TSS particle diameter size. Similarly, the nitrification and denitrification rate constants were found to be the most important calibration parameters affecting wetland effluent pollutant concentrations.

These results demonstrated that the model is a useful tool in evaluating the sensitivity of design sustainability to changes in both design criteria and input/calibration parameters. Therefore, the model has potential for use in constructed wetland design and optimization, as well as in making policy decisions related to constructed wetland performance. While current design guidelines specified by MDE (2009) allow for a relatively simple and universal method for constructed wetland design, this model could aid in the development of process-based wetland designs. Therefore, in place of using generalized design criteria such as sizing the forebay to be 10% of the WQ_v (MDE 2009), engineers could use the model to design constructed wetlands to optimize specific wetland functions for a given site. This method of design allows for better design optimization as well as better understanding of wetland behavior.

10.2.5 Use of model to evaluate current wetland designs

Implementations of the model were successfully calibrated for and used to simulate the hydrologic and water quality performance of a number of different wetland designs including a stormwater wetland, a municipal wastewater wetland, an agricultural wastewater wetland, and a wildlife habitat wetland. Different experiments were also tested on each wetland type to evaluate the effects of changes to design criteria, user-defined inputs, influent water characteristics, and internal model mechanisms on the performance of a given wetland design with respect to the relevant sustainability metrics. The results from these experiments demonstrated that the model could be successfully used to evaluate a large range of wetland design aspects.

The effects of estimated population growth on a hypothetical municipal wastewater wetland, which was design according to USEPA (2000) guidelines, was analyzed. The resulting water quality performance of the wetland designs EPA1 and EPA2 showed that the municipal wastewater wetland designed according to USEPA (2000) specifications was resilient to increases in service population water use. Given these results, the model was shown to be a powerful tool in evaluating the resiliency of a given wetland design to changes in influent water characteristics as well as environmental changes.

Due to anoxic conditions in the agricultural wastewater wetland designed in Chapter 8, the effluent NH_4^+ concentrations simulated by the model did not initially agree with those determined through the field test design method defined by NRCS (2002). In order to address this discrepancy, two additional ammonia removal mechanisms were added to the model toolkit and evaluated in the AG1, AG2, and AG3 designs. This process showed (1) that different mechanisms may dominate in different wetland designs and (2) that mechanisms such as plant oxygen transfer and NH_3 volatilization could be added relatively easily to the model due to its simple structure. The addition of these two ammonia removal mechanisms allowed for the final calibration of the AG3 design with mean daily effluent TN concentrations of 164 mg/L, which was comparable to the NRCS (2002) defined value of 162 mg/L.

Finally, the design and simulation of a wildlife habitat wetland demonstrated that the model could be used to develop optimal designs for different wildlife types such as wading shorebirds, amphibians and reptiles, and wetland furbearers. The resulting BMP-weighted WSI score for this habitat wetland was also computed to be

0.629, which was comparable to that of 0.640 for the stormwater wetland design. The hydrologic and water quality performance of the habitat wetland design within the model suggested that wetlands could be designed to serve both as BMP facilities as well as habitat wetlands.

10.3 RECOMMENDATIONS FOR FUTURE WORK

10.3.1 Model water quality characterization

Calibration with respect to the Jordan (2013) weekly water quality data was less successful that of the hydrologic data due to (1) the long time interval (weekly) over which the water quality data were recorded with respect to the model time step of 1 min, (2) the poorly defined flowpath through the Barnstable 1 wetland, and (3) the exclusion of TSS resuspension and NH_4^+ generation via plant decay within the model. While the Barnstable 1 database can be valuable to some types of analyses, a model with a short time interval was necessary to make analyses relevant to wetland sustainability. The observed Barnstable 1 weekly effluent TSS and NH_4^+ concentrations were found to equal or even exceed corresponding observed influent concentrations, which suggested that internal wetland mechanisms contributed to both the removal and generation of these constituents. While the model did allow the user to define wetland background levels for all constituents, this model construct represented a lower limit of internal wetland concentrations rather than mechanisms that directly generated TSS and NH_4^+ via wetland functions such as resuspension and plant decay. These water quality calibration results showed that constructed wetlands are complex facilities that promote a number of water quality processes, all of which are functions of wetland conditions such as DO levels, water velocity and depth, flowpath, etc. Given the complexity of these facilities, modeling can be an exceptionally useful decision tool.

The model could be further calibrated and modified using a wide variety of constructed wetland datasets as they become available in order to better characterize the most important wetland water quality mechanisms. In order to improve the usefulness of the model, more water quality mechanisms could also be added to future versions of the model and that future studies could focus on the collection of more comprehensive water quality and hydrologic wetland data. Future versions of the model should include mechanisms that simulate resuspension and wetland generation of water quality constituents such as TSS, NH_4^+ and NO_3^- in order to more accurately simulate wetland water quality performance. The incorporation and simulation of additional water quality constituents such as BOD and organic nitrogen could also prove helpful to more accurately represent wetland water quality behavior especially with respect to wastewater treatment wetlands.

Model calibration results would also be greatly improved by water quality data collected on a smaller time scale (i.e., hourly or daily rather than weekly). While water quality data collection is both expensive and labor intensive, such data are crucial to the development of better models. Additionally, more detailed internal wetland elevation and flowpath data would lead to more accurate simulated hydrologic, especially internal wetland storage volume, outputs. Therefore, the need for more comprehensive water quality and hydrologic data for better model calibration was shown. A model such as the model developed herein could be used to show the benefits of different levels of data availability, both on temporal and spatial scales.

While the model allowed for the simulation and evaluation of different wetland designs, a number of changes could improve its usefulness. As exemplified by the FB1, FB2, and FB3 designs analyzed in Section 6.14.1, the model did not fully characterize the importance of the forebay with respect to TSS settling. This model limit was due to the use of one mean TSS particle diameter size within the model when, in reality, influent TSS particle size varies greatly. The use of an input TSS particle diameter distribution could result in more realistic wetland TSS performance characterization. A model, however, can only be as good as the data used to calibrate it. Given the limited TSS particle diameter data generally available, the current study suggests that more studies be dedicated to the characterization of both stormwater and wastewater TSS particle diameters. Having the distribution of TSS data, rather than simply the mean, would then enable the corresponding increase in the accuracy of wetland trap efficiency estimates.

10.3.2 Data-Model relationship

Given that data limitations restrict the usefulness of any model, the model may also help scientists and engineers determine the data that would yield the most accurate model. Monitoring of wetlands provides data necessary to calibrate wetland models, assess the functioning of experimental wetlands, and contribute knowledge that can be transferred for use at other wetlands with similar properties. Poorly designed monitoring programs can limit the knowledge content of measured data. A model of a proposed wetland can permit an *a priori* assessment of the benefits of alternative proposed monitoring programs, thus maximizing the information content of the measured data. Simulation with the wetland model can be used to identify the most effective locations within and immediately outside of the wetland area to place monitoring instruments. A spatio-temporal model of the wetland will allow assessment of the expected time required to experience the variety of conditions needed to characterize the long-term state of the wetland.

10.3.3 Wetland model applications

The simulation and evaluation of different wetland designs proved useful in a number of applications. Within the current study, the model was successfully used to evaluate the effects of changes in wetland design components, in influent water characteristics, and in the internal water quality mechanisms simulated by the model. Therefore, the model could be used to design wetlands with resilience to stressors such as climate change, population growth, contributing drainage/service area land use changes, extreme events such as droughts and floods, etc. Additionally, the model could be used to evaluate both existing and proposed wetland designs as well as to optimize designs based on stakeholder’s needs. The resulting, optimized design approach would emphasize long-term, sustainable designs, which are crucial to healthy, stable downstream ecosystems.

Another interesting application of the model could be to assess the effects of incorporating different best management practices (BMPs) into a watershed system that includes a wetland. Because the bottoms of wetlands are often lined with clay to maintain a given water depth, they do not typically contribute significantly to groundwater recharge or baseflow maintenance. The model could assess the effect of replacing part of the wetland with a an infiltrative BMP such as a bioretention cell or infiltration ditch would affect these functions, as well as other wetland functions. The buffer surrounding the wetland, for example, could be designed as a BMP as well. Such combinations should make the wetland perform its functions more effectively. Rating BMP services and components based on both their maintenance within the BMP and by their impact downstream, could better determine BMP suitability for a given area. Scoring constructed wetlands and all BMP designs based on sustainability could also lead to the design of longer-lasting and self-maintaining BMPs, possibly saving money and enhancing downstream ecosystems.

10.3.4 Model Optimization Applications

The existing approach to wetland design, i.e., the use of generalized indices, fails to reflect the multiple functions of every wetland. Therefore, non-optimum designs can result. Optimization of metrics could remove much of the subjectivity and greatly improve the effectiveness of a design. While a number of studies have used models to optimize performance for one function (i.e., phosphorus removal), studies thus far have not been reported that optimized multiple criteria such as hydrologic, water quality, and habitat functions as a whole, especially using sustainability criteria as the focus. The model could be used to design a wider range of constructed wetlands based on treatment needs and existing site conditions.

Optimizing a wetland design for often competing functions such as groundwater recharge, water quality control, and wildlife habitat performance is often necessary because of stakeholder conflicts. Allowing multiple wetland functions to be addressed in the optimization could yield more satisfactory decisions.

Multifunctional wetlands should be used on a smaller scale (communities, towns, or farms) to control both stormwater and wastewater rather than large-scale wastewater treatment plants (WWTPs). If water is treated earlier in the watershed, a longer contact time may allow for greater pollutant and nutrient removal before entering major bodies of water such as the Chesapeake Bay. With more localized solutions, wetland outflow could be used for irrigation and other grey water uses (Campbell and Ogden, 1999), thus contributing to a more sustainable water use cycle. Optimization will allow for the inclusion of regional criteria along with design criteria relevant to the components of the wetland. Total sustainability requires that development be designed with a life cycle in mind and that the components be optimally configured.

Chapter 11: Appendix

11.1 USER INPUT CHARACTERIZATION

The current section summarizes all user inputs for the mode developed in the current study. In its current state, the model is calibrated with climatic forcings for Baltimore, MD as discussed in Sections 4.2.3 and 4.2.4. The following climactic inputs were incorporated into the model:

1. Daily mean air temperature (°C)
2. Daily maximum air temperature (°C)
3. Daily minimum air temperature (°C)
4. Daily dew point temperature (°C)
5. Daily wind speed (m/s)
6. Hourly incident solar radiation (MJ/m^2-hr)

In addition to these climatic conditions, the model requires a total of 37 inputs in order to fully characterize a given wetland design. Table 11-1 lists all of these inputs and their corresponding units. In addition to these main user inputs, a total of five inputs were added to the model for computation of sustainability metrics. These supplementary inputs are discussed and defined in Chapter 3: and summarized in Section 3.6.

Table 11-1 All user input parameters and their corresponding units.

Model parameter	Units
Number of years of simulation	years
Annual number of wet days	days
Cell length	ft
Number of cells in wetland design	---
FID vector	---
Vegetation specification for each cell (no vegetation = 0, emergent = 1, submerged = 1)	---
Initial water depth in each cell	ft
Bottom elevation in each cell	ft
Berm height at exit of each cell	ft
Wetland albedo a	---
Leaf area index LAI	---
Shelter factor f_s	---
Maximum leaf conductance C_{leaf}^*	mm/s
Emergent vegetation height z_v	m
Hydraulic Conductivity K_V	ft/d
Drainage area DA	acres
Outlet type (orifice = 1, weir = 2)	---
Orifice coefficient (English units)	---
Outlet orifice area	ft^2
Outlet weir length	ft
Outlet weir/orifice invert height	ft
Runoff coefficient C	---
Weir coefficient C_w	---
Orifice discharge coefficient C_O	---
Particle diameter D	m
Maximum photosynthesis rate $PMAX$	mg-O_2/m^2-hr
Initial wetland water temperature $T_{w(o)}$	°C
Initial DO concentration in wetland DO_o	mg/L
Irreducible background TSS concentration TSS_o	mg/L
Irreducible background NH_4^+ concentration $NH4_o$	mg/L
Irreducible background NO_3^- concentration	mg/L
Influent TSS concentration (TSS_{in})	mg/L
Influent NH_4^+ concentration ($NH4_{in}$)	mg/L
Influent NO_3^- concentration ($NO3_{in}$)	mg/L
Influent DO concentration (DO_{in})	mg/L
Nitrification rate constant (K_{NIT})	hr^{-1}
Denitrification rate constant (K_{DNT})	hr^{-1}

11.1.1 Number of years of simulation

The user can input any given number of years for simulation for a given wetland design. Generally, the most useful simulation duration would be long enough to allow for extreme events (i.e., periods of flood and drought) while short enough to reduce unnecessary computational time. It also may be relevant to factor in the estimated life span of a given wetland system as it may be appropriate to simulate wetland performance for a predicted life time.

The lifespan of a wetland system can depend on a number of factors including the characteristics of influent water, pre-treatment measures, geographic location (chemical processes faster in warmer areas), wetland design, etc. (Cronk 1996; USEPA 2000; MDE 2009). USEPA (2000) states that free water surface (FWS) treatment wetlands (wetlands that treat water via surface flow as opposed to subsurface flow) receiving effluent from oxidation ponds can operate over 10 to 15 years before requiring removal of accumulated sediments. Hammer (1992) also cited that while the literature lacks sufficient data on constructed wetland lifespans, they have been estimated to have a projected life span of about 20 years. Natural wetlands used for secondary wastewater treatment have even been reported to perform consistently over 60 years (Hunter et al. 2009).

11.1.2 Annual number of wet days

The current model was calibrated with an annual number of wet days of 93 days, which is characteristic of the Baltimore, MD area (McCuen 2013). This input, however, can vary greatly depending on the proposed geographic location of a given wetland. The model user should calibrate this input based on rainfall records of the location of the proposed wetland.

11.1.3 TSS particle diameter characterization *D*

Influent TSS particle diameters are dependent on the characteristics of water entering the wetland. Water with less pretreatment and runoff from areas with greater imperviousness generally carry TSS particles with larger diameters (MDE 2009). Conversely, more pretreatment and lower imperviousness often contribute to smaller particle diameters. However, because both stormwater and wastewater pollutant concentrations can be highly variable, it is often difficult to predict corresponding mean particle diameters or distributions. As a general rule of thumb, TSS particle diameters should not go below $1x10^{-7}$ m (0.1 μm), which corresponds to USDA-defined clay particles (USDA 1987). TSS particles with smaller diameters result in irrationally slow settling velocities.

The TSS particle size distribution of stormwater often has a complex relationship with flow and is dependent on a number of factors including upstream soil characteristics and land use, season, rainfall intensity/duration, droughts and floods, and construction within the drainage area (Pathak et al. 2004; Rinker Materials 2004; DeGroot and Weiss 2008). For urban stormwater runoff, MDE (2009) specified that mean particle diameter of 40 μm for drainage areas with percent imperviousness greater than 75% and of 20 μm for drainage areas with less than or equal to 75% imperviousness. The Nationwide Urban Runoff Program (NURP), a study done by the EPA to characterize urban runoff water quality and its effects downstream, estimated a general particle diameter distribution for urban stormwater in the US (USEPA 1983). As is shown in Figure 11-1 and summarized in Table 11-2, 80% of all particles were found to have a diameter of 28 µm or less. 20% of all particles also have diameters smaller than 3 µm. From this distribution, a median value of 9.5 µm was estimated to be most representative of typical urban runoff particle diameters based on UESPA (1983).

Table 11-2 Particle diameter distribution reported by USEPA (1983) for urban runoff in the US.

Particle Diameter (µm)	% Weight finer
100	100
28	80
13	60
6	40
3	20

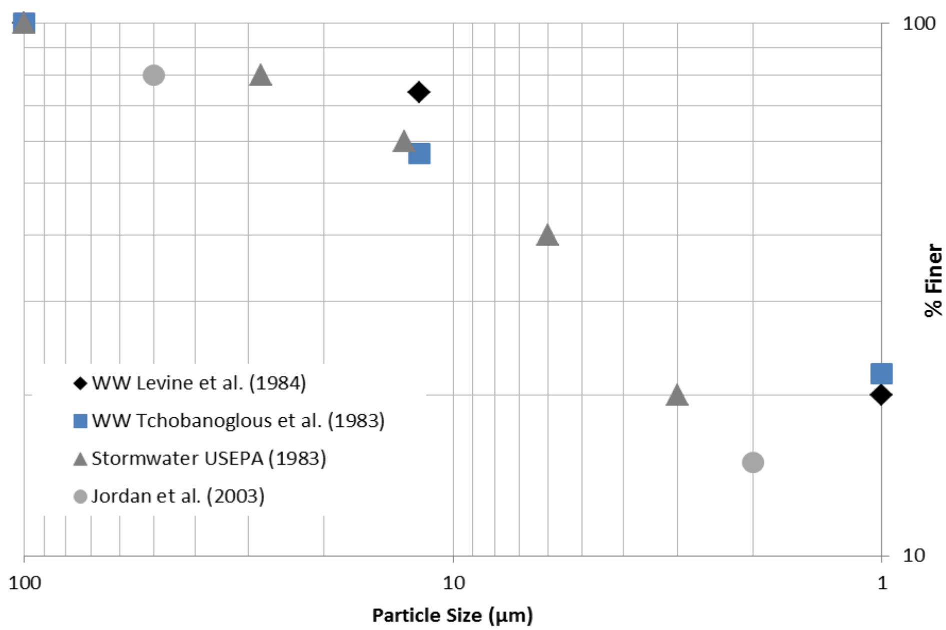

Figure 11-1 Literature-derived particle size distributions from two sources for primary effluent, Levine et al. (1984) (diamonds) and Tchobanoglous et al. (1983) (squares); one urban runoff, USEPA (1983); and one for agricultural runoff entering the Barnstable 1 wetland as estimated from drainage area soil characteristics specified by Jordan et al. (2003) (circles).

Analogous particle diameter guidelines for agricultural runoff were not found in the literature. While reports have studied the particle size distributions of agricultural runoff, distributions varied greatly between study sites with different soil types and geographic location (Liebens 2001; Pathak et al. 2004). Additionally, particle size studies were not found for the mid-Atlantic region of the US. Pathak et al. (2004) did, however, observe that agricultural runoff particle distribution in India closely followed topsoil distributions during large storm events as well at peak flows for all storms. Liebens (2001) also found that particle diameter sizes in swales that receive agricultural runoff reflected the high content of sand in the contributing drainage areas, which were located in Escambia County, Florida. Given these results, the current study assumed that the contributing drainage area particle size distribution was a sufficient estimation of corresponding runoff particle diameter distribution for agricultural runoff.

USEPA (2000) summarized the particle diameter distributions for municipal primary effluent from two separate sources, both of which are reprinted in Table 11-3. The resulting distribution from Levine et al. (1984) and Tchobanoglous et al. (1983) were also plotted in Figure 11-1. Both Levine et al. (1984) and Tchobanoglous et al. (1983) reported that 100% of all primary effluent particles had a diameter of less than 100 μm. Additionally, both studies also reported about 20% of all particle diameters to be finer than 1 μm. A median particle diameter of 6.5 μm was estimated for primary effluent based on both Levine et al. (1984) and Tchobanoglous et al. (1983) values. Again, due to the skew towards smaller diameters, the median was assumed to be the best representative of primary effluent particle size.

Table 11-3 Particle diameter distributions of primary effluent for two different WWTP. Table adapted from USEPA (2000).

	Primary effluent particle diameter distribution (% Weight TSS)	
Particle Diameter (μm)	**Levine et al. (1984)**	**Tchobanoglous et al. (1983)**
$<10^{-3}$	---	---
10^{-3}-1.0	20	22
1.0-12	54	35
>12	26	43
1-100	---	---
>100	---	---

11.1.4 Cell length/Number of cells

Throughout the literature, wetlands have been found to exhibit near plug flow behavior. Therefore, wetlands are most often modeled by plug flow or completely mixed flow reactors (CMFR) in-series (Kadlec and Knight 1996; USEPA 2000; Carleton et al. 2001; Bastviken 2006; Chavan and Dennett 2008; Kadlec 2009). The degree to which wetland flow follow plug flow has been cited to rely on wetland L:W ratio and flowpath lengths and the number of CMFRs in series required can be determined through tracer experiments (Carleton et al. 2001). If both the influent and effluent concentrations of relevant water quality constituents are known, the number of CMFRs in series (n) can also be estimated by solving the following equation (Weber Jr. 2001):

$$C_n = C_{in}\left[\frac{1}{\left(1+\frac{K \cdot t_{HR}}{n}\right)}\right]^n \tag{11-1}$$

where C_n in the constituent concentration (mg/L) after *n* CMFRs, C_{in} is the influent concentration (mg/L), K is the reaction rate (hr^{-1}) associated with the constituent, and t_{HR} is the wetland hydraulic retention time (hr).

The actual effect of the flowpath length has been debated (USEPA 2000). Kadlec and Knight (1996) found that a model consisting of three CMFRs in series reasonably simulated water quality performance for wetlands regardless of shape. Persson et al. (1999), however, found wetland configuration to significantly affect wetland flow, citing that this variation was due to a lack of a reliable flow design tools (i.e., the hydraulic retention time is not always reliable).

Each cell within a given wetland design was assumed to behave like a CMFR. The model does not, however, simulate tanks-in-series flow, but rather defines the flow of cells within a two dimensional grid. Multiple cells are allowed to flow into two receiving cells and often times dead cells exist, which do not receive flow from other cells and are only replenished by rainfall. Given this model construct, it was difficult to determine the appropriate number of cells to represent flow within a given wetland design. Additionally, cells often required sizing based on wetland configuration. Each wetland design, for example, included areas with different water depth and vegetation specifications. Additionally, cell characterization may also be limited by the quality of flow, elevation, and vegetation data available for a given wetland design as was the case with the Barnstable 1 wetland studied by Jordan et al.

(2003). Therefore, cells should be sized in order to best simulate wetland flow as well as to best characterize wetland design specifications within the limits of any related wetland data.

11.2 TR-55 time of concentration determination

As discussed in Section 6.2, MDE (2009) used TR-55 to compute the time of concentration for the Clevenger Community Center drainage area (see Figure 11-2). The resulting time of concentrations was computed to be 0.26 hr and was based on the land uses found within the drainage area as well as the respective lengths of different flow types such as sheet flow and open channel flow (see Figure 11-2). Within TR-55, the sheet flowpath length was restricted to 100 ft, which made this method less useful for larger drainage areas. Therefore, the TR-55 method was not sufficiently complex to determine the time of concentration for the 413 ac drainage area served by the wildlife habitat wetland designed in Chapter 9.

```
                                                        Hydrologic Soil Group
COVER DESCRIPTION                                    A         B         C         D
Acres (CN)
------------------------------------------------------------------------------------
FULLY DEVELOPED URBAN AREAS (Veg Estab.)
Open space (Lawns,parks etc.)
  Good condition; grass cover > 75%                -  3.06(61)       -         -
Impervious Areas
  Paved parking lots, roofs, driveways             -  1.94(98)       -         -
OTHER AGRICULTURAL LANDS
Woods                            good              -   0.3(55)       -         -

Total Area (by Hydrologic Soil Group)                   5.3
------------------------------------------------------------------------------------
                TOTAL DRAINAGE AREA: 5.3 Acres       WEIGHTED CURVE NUMBER: 74*
------------------------------------------------------------------------------------
* - Generated for use by GRAPHIC method

                     TIME OF CONCENTRATION AND TRAVEL TIME        Version 2.00
------------------------------------------------------------------------------------
Flow Type    2 year   Length   Slope  Surface   n   Area     Wp   Velocity  Time
              rain     (ft)   (ft/ft)   code       (sq/ft)  (ft)  (ft/sec)  (hr)
------------------------------------------------------------------------------------
Sheet         3.3      70      0.013    F                                   0.209
Shallow Concent'd     310      0.013    P                                   0.037
Open Channel                                                        5.0     0.007
                                                Time of Concentration = 0.26*
           --- Sheet Flow Surface Codes ---
    A Smooth Surface           F Grass, Dense     --- Shallow Concentrated ---
    B Fallow (No Res.)         G Grass, Burmuda   ---     Surface Codes     ---
    C Cultivated < 20 % Res.   H Woods, Light             P Paved
    D Cultivated > 20 % Res.   I Woods, Dense             U Unpaved
    E Grass-Range, Short       J Range, Natural
* - Generated for use by GRAPHIC method
```

Figure 11-2 Time of concentration and curve number computation via TR-55 as defined in MDE (2009) for the Clevenger Community Center in Charles County, MD. The resulting time of concentration curve number for the developed 5.3 ac drainage area were 0.26 hr and 74, respectively.

11.3 Orifice and Weir sizing for the DA1 and DA2 designs

The following sections discuss in detail the procedures followed to compute the outlet structure designs for the DA1 and DA2 stormwater wetland designs defined in Section 6.13.1. These designs incorporated respective contributing drainage areas of 3.18 and 6.36 ac., which were both assumed to have the same land use and slope properties as that of the base design drainage area. The same outlet structure procedures followed in Sections 6.2 and 6.5 were followed to design analogous outlet structures in the DA1 and DA2 designs in order to ensure the proper flow control in both designs.

11.3.1 DA1 design

Design DA1 served a drainage area of 3.18 ac and, therefore, received less inflow, requiring different-sized outlet structures than the base design, which served a drainage area of 5.3 ac. As shown in Figure 6-7, the outlet structure of the base stormwater wetland design consisted of an orifice for the control of the 1-yr, 24-hr flood, a weir for the control of the 10-yr, 24-hr flood, and a second weir for the safe transfer of the 100-yr, 24-hr flood. The dimensions and relative heights of these outlet structures changed based on the drainage area size. As computed in Section 6.2, the influent volumes associated with the 1- and 10-yr, 24-hr floods were defined as Cp_V and Q_P, which had respective values of 8,865 ft^3 (0.204 ac-ft) and 16,068 ft^3 (0.370 ac-ft). As a result of the reduced inflow into the wetland design DA1, $Cp_V(DA1)$ and $Q_P(DA1)$ volumes were also smaller:

$$Cp_V(DA1) = 8{,}865\ \text{ft}^3\left(\frac{3.18\ \text{ac}}{5.3\ \text{ac}}\right) = 5{,}319\ \text{ft}^3(0.122\ \text{ac-ft}) \tag{11-2}$$

$$Q_P(DA1) = 16{,}068\ \text{ft}^3\left(\frac{3.18\ \text{ac}}{5.3\ \text{ac}}\right) = 9{,}641\ \text{ft}^3(0.221\ \text{ac-ft}) \tag{11-3}$$

Once these volumes were computed, corresponding 1- and 10-yr, 24-hr flood storage depths of 1.54 and 2.78 ft were computed by dividing DA1, $Cp_V(DA1)$ and $Q_P(DA1)$ by the DA1 wetland surface area of 3,463 ft^2 (0.0795 ac). These depths were later used to compute corresponding DA1 weir dimensions. As discussed in Section 6.2, MDE (2009) required a minimum outlet orifice diameter of 3 in. in order to avoid clogging. For this reason the initial computed base design orifice diameter of 1.65 in. (see Equation 6-14) was sized up to 3 in. In order to test if the DA1 design required a larger diameter, the maximum influent discharge rate $q_1(DA1)$ for the post-development 1-yr, 24-hr flood was computed for design DA1 based on the corresponding 1-yr, 24-hr runoff depth and unit peak discharge computed by MDE (2009) for the base stormwater design in Section 6.4 (see Table 11-4):

$$q_1(DA1) = \left(0.995\frac{\text{cfs}}{\text{ac - in.}}\right)(0.72\text{ in.})(3.18\text{ ac}) = 2.28\text{ cfs} \tag{11-4}$$

Once $q_1(DA1)$ was computed, the ratio of the required outflow maximum discharge allowed to exit the weir to the post-development peak discharge (q_o/q_i), which was defined as 0.03 in Section 6.2 for the base design, was multiplied by $q_1(DA1)$ to determine the corresponding required peak outflow discharge rate q_o:

$$q_o = (q_o/q_i)\cdot q_i = (0.03)(2.28\text{ cfs}) = 0.0684\text{ cfs} \tag{11-5}$$

This DA1 q_o of 0.0684 cfs was then input into Equation 6-14 along with the storage depth of 1.54 associated with $Cp_V(DA1)$ in order to determine the required outlet orifice area for design DA1:

$$A_o = \frac{q_o}{C_o\sqrt{2gh_o}} = \frac{0.0684\text{cfs}}{0.6\sqrt{2(32.2\text{ ft/s}^2)(1.54\text{ ft})}} = 0.0114\text{ ft}^2 \tag{11-6}$$

The resulting orifice area A_o corresponded to an orifice diameter of 1.45 in., which was below the MDE-required minimum diameter of 3 in. Therefore, the DA1 wetland orifice also had a diameter of 3 in.

Next, required effluent peak 10- and 100-yr, 24-hr flows were computed from the runoff depths and unit peak discharge rates listed in Table 11-4. While pre-development depths and discharge rates were used to compute the 10-yr effluent rate, post-development values were used to determine the 100-yr effluent peak discharge because the wetland was designed to control the 10-yr, 24-hr flood, but only safely transfer the 100-yr flood:

$$q_{10}(DA1) = \left(0.904\,\frac{\text{cfs}}{\text{ac - in.}}\right)(1.34\,\text{in.})(3.18\,\text{ac}) = 3.85\,\text{cfs} \qquad (11\text{-}7)$$

$$q_{100}(DA1) = \left(1.124\,\frac{\text{cfs}}{\text{ac - in.}}\right)(4.48\,\text{in.})(3.18\,\text{ac}) = 16.0\,\text{cfs} \qquad (11\text{-}8)$$

where $q_{10}(DA1)$ is the required peak discharge out of the weir controlling the 10-yr, 24-hr flood and $q_{100}(DA1)$ is the peak discharge computed to flow through wetland due to the 100-yr, 24-hr flood. Corresponding 10- and 100-yr weir lengths were then computed for design DA1 based on $q_{10}(DA1)$, $q_{100}(DA1)$, and the storage depths corresponding to $Cp_V(DA1)$ and $Q_P(DA1)$ through Equations 6-29 and 6-31:

$$L_{10} = \frac{q_{i,10} - C_o A_o \sqrt{2gh_o}}{C_w h_{w,10}^{3/2}} = \frac{3.85\,\text{cfs} - (0.6)(0.05\,\text{ft}^2)\sqrt{2(32.2\,\text{ft/s}^2)(1.54)}}{(3.1)(1.24)^{3/2}} = 0.831\,\text{ft} \qquad (11\text{-}9)$$

$$L_{100} = \frac{q_{i,100} - C_w L_{10} h_{w,10}^{3/2}}{C_w h_{w,100}^{3/2}} = \frac{16.0\,\text{cfs} - (3.1)(0.831\,\text{ft})(1.74\,\text{ft})^{3/2}}{(3.1)(0.5)^{3/2}} = 9.21\,\text{ft} \qquad (11\text{-}10)$$

The resulting outlet structure is for design DA1 is shown in Figure 6-13.

Table 11-4 Relevant output values from TR-55 as well as the input rainfall depths for the 1, 10, and 100-yr 24-hr storm events as defined by MDE (2009).

	1-yr, 24-hr (Cp_V)	**10-yr, 24-hr (Q_p)**	**100-yr, 24-hr (Q_f)**
Rainfall depth (in.)	2.7	5.3	7.5
Post-development runoff depth (in.)	0.72	2.61	4.48
Post-development unit peak discharge (cfs/ac/in.)	0.995	1.098	1.124
Post-development peak discharge (cfs)	3.79	15	27
Pre-development runoff depth (in.)	0.18	1.34	2.76
Pre-development unit peak discharge (cfs/ac/in.)	0.460	0.904	0.967
Pre-development peak discharge (cfs)	0.439	6.42	14.1

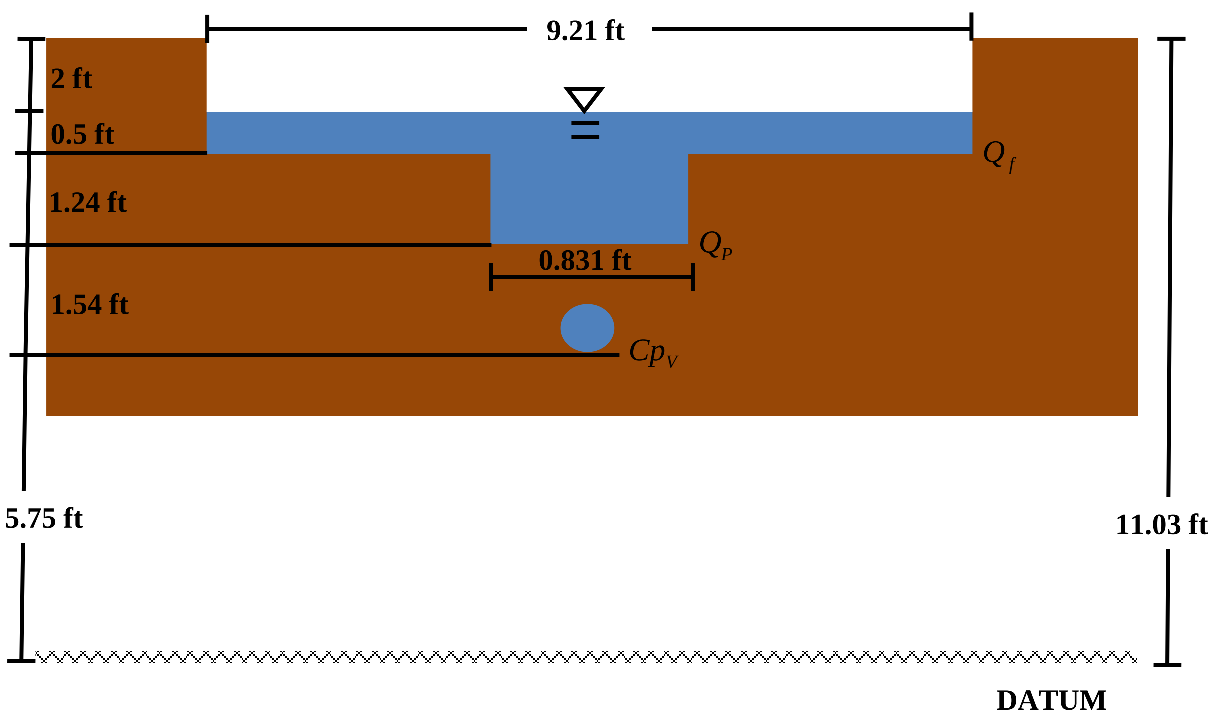

Figure 11-3 Illustrates the DA1 design outlet orifice and double riser design through the MDE (2009) method. The datum represents the bottom of the micropool, which was designed to have a depth of 5.75 ft. The designated Cp_V, Q_P, Q_f depths represent the water depths corresponding to the 1-yr, 10-yr, and 100-yr, 24-hr floods within the wetland.

11.3.2 DA2 design

With a contributing drainage area of 6.36 ac, more water entered the DA1 design that the base design, requiring a different outlet structure. In order to determine the 1- and 10-yr, 24-hr storage depths associated with the DA2 design, $Cp_V(DA2)$ and $Q_P(DA2)$ volumes were computed:

$$Cp_V(DA1) = 8{,}865\,\text{ft}^3\left(\frac{6.36\,\text{ac}}{5.3\,\text{ac}}\right) = 10{,}638\,\text{ft}^3(0.244\,\text{ac-ft}) \quad (11\text{-}11)$$

$$Q_P(DA1) = 16{,}068\,\text{ft}^3\left(\frac{6.36\,\text{ac}}{5.3\,\text{ac}}\right) = 19{,}282\,\text{ft}^3(0.443\,\text{ac-ft}) \quad (11\text{-}12)$$

Once these volumes were computed, corresponding 1- and 10-yr, 24-hr flood storage depths of 3.07 and 5.57 ft were computed by dividing $Cp_V(DA2)$ and $Q_P(DA2)$ by the DA1 wetland surface area of 3,463 ft^2 (0.0795 ac). These depths were then used to compute corresponding DA2 weir dimensions. Next, the required orifice diameter was computed for the DA2 design as was done for the DA1 design:

$$q_1(DA2) = \left(0.995\frac{\text{cfs}}{\text{ac-in.}}\right)(0.72\,\text{in.})(6.36\,\text{ac}) = 4.56\,\text{cfs} \quad (11\text{-}13)$$

Once $q_1(DA2)$ was computed, the ratio of the required outflow maximum discharge allowed to exit the weir to the post-development peak discharge (q_o / q_i), which was defined as 0.03 in Section 6.2 for the base design, was multiplied by $q_1(DA2)$ to determine the corresponding required peak outflow discharge rate q_o:

$$q_o = (q_o / q_i)\cdot q_i = (0.03)(4.56\,\text{cfs}) = 0.137\,\text{cfs} \quad (11\text{-}14)$$

This DA2 q_o of 0.137 cfs was then input into Equation 6-14 along with the storage depth of 3.07 associated with $Cp_V(DA2)$ in order to determine the required outlet orifice area for design DA2:

$$A_o = \frac{q_o}{C_o\sqrt{2gh_o}} = \frac{0.137\text{cfs}}{0.6\sqrt{2(32.2\text{ ft/s}^2)(3.07\text{ ft})}} = 0.0162\text{ ft}^2 \qquad (11\text{-}15)$$

The resulting DA2 orifice area A_o corresponded to an orifice diameter of 1.72 in., which was below the MDE-required minimum diameter of 3 in. Therefore, the DA2 wetland orifice also had a diameter of 3 in.

Next, required effluent peak 10- and 100-yr, 24-hr flows were computed from the runoff depths and unit peak discharge rates listed in Table 11-4:

$$q_{10}(DA2) = \left(0.904\frac{\text{cfs}}{\text{ac - in.}}\right)(1.34\text{ in.})(6.36\text{ ac}) = 7.70\text{ cfs} \qquad (11\text{-}16)$$

$$q_{100}(DA1) = \left(1.124\frac{\text{cfs}}{\text{ac - in.}}\right)(4.48\text{ in.})(6.36\text{ ac}) = 32.0\text{ cfs} \qquad (11\text{-}17)$$

where $q_{10}(DA2)$ is the required peak discharge out of the weir controlling the 10-yr, 24-hr flood and $q_{100}(DA2)$ is the peak discharge computed to flow through wetland due to the 100-yr, 24-hr flood. Corresponding 10- and 100-yr weir lengths were then compared for design DA2 based on $q_{10}(DA2)$, $q_{100}(DA2)$, as the storage depths corresponding to $Cp_V(DA2)$ and $Q_P(DA2)$ through Equations 6-29 and 6-31:

$$L_{10} = \frac{q_{i,10} - C_o A_o\sqrt{2gh_o}}{C_w h_{w,10}^{3/2}} = \frac{7.70\text{ cfs} - (0.6)(0.05\text{ ft}^2)\sqrt{2(32.2\text{ ft/s}^2)(3.07)}}{(3.1)(2.5)^{3/2}} = 0.595\text{ ft} \qquad (11\text{-}18)$$

$$L_{100} = \frac{q_{i,100} - C_w L_{10} h_{w,10}^{3/2}}{C_w h_{w,100}^{3/2}} = \frac{32.0\text{ cfs} - (3.1)(0.595\text{ ft})(3\text{ ft})^{3/2}}{(3.1)(0.5)^{3/2}} = 20.5\text{ ft} \qquad (11\text{-}19)$$

The resulting outlet structure is for design DA2 is shown in Figure 6-14.

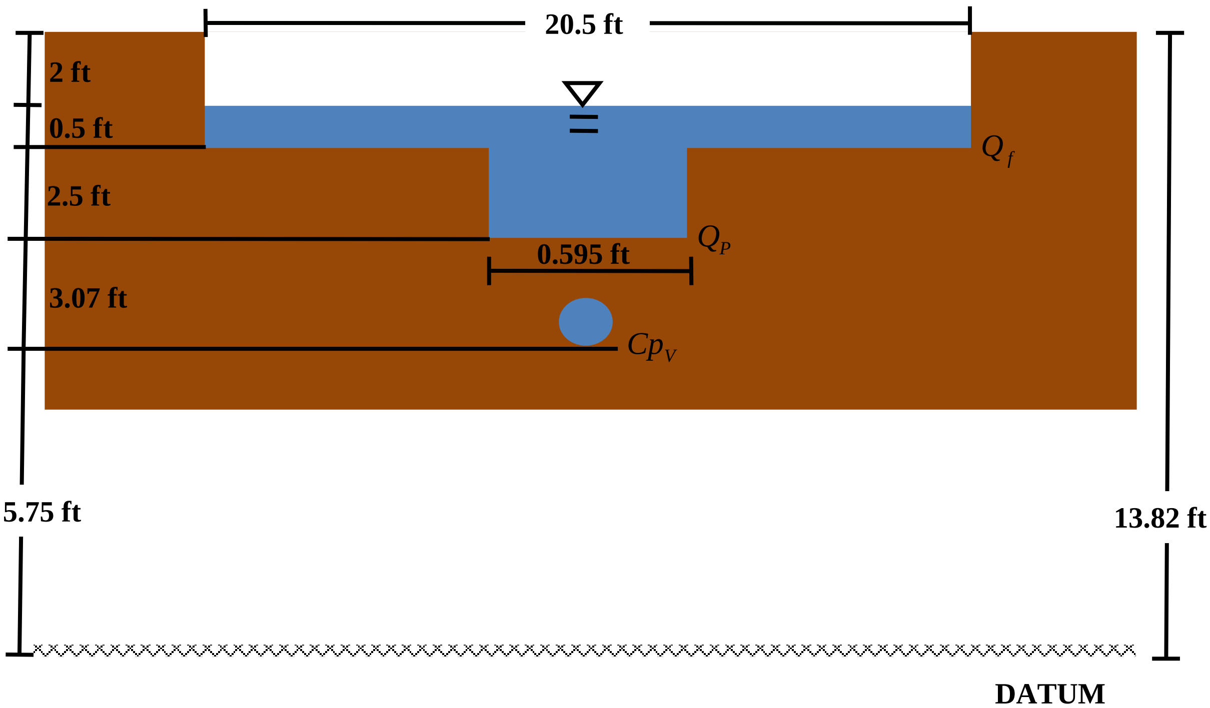

Figure 11-4 Illustrates the DA2 design outlet orifice and double riser design through the MDE (2009) method. The datum represents the bottom of the micropool, which was designed to have a depth of 5.75 ft. The designated Cp_V, Q_P, Q_f depths represent the water depths corresponding to the 1-yr, 10-yr, and 100-yr, 24-hr floods within the wetland.

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