THESIS

RETROFITTING A WATER QUALITY CONTROL STRUCTURE TO MAXIMIZE POLLUTANT REMOVAL EFFICIENCY FOR AN EXISTING WETLAND

Submitted by

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ABSTRACT

RETROFITTING A WATER QUALITY CONTROL STRUCTURE TO MAXIMIZE POLLUTANT REMOVAL EFFICIENCY FOR AN EXISTING WETLAND

An existing seven acre wetland captures stormwater runoff from a 505 acre watershed located in Fort Collins, CO. The wetland has shown measureable pollutant removal with its current outlet design, but the pollutant removal efficiency could be increased through the installation of a water quality control structure (WQCS). The wetland is bounded by an adjacent park, stream, bike path, and building limiting water quality improvement options. Thus, the wetland dimensions cannot be altered. The objective of this project is to design a water quality control structure that would maximize pollutant removal efficiency and the mass of total suspended solids (TSS) removed in the wetland without causing additional flooding at the site and adversely affecting the adjacent properties. An additional objective of this project was to develop a method to calculate the hydraulic retention time (HRT) for a stormwater wetland.

EPA's Stormwater Management Model Version 5 was used to model the existing conditions and various proposed WQCS drawdown times. The modeled drawdown times ranged from 2 hours to 72 hours. Continuous simulation modeling was used because the wetland volume could not be adjusted to contain the water quality capture volume. It was assumed that all stormwater runoff entering the wetland was captured and treated. Using the model generated volume, depth, and flow data, the non-steady state hydraulic retention times and hydraulic loading rates (HLR) were calculated for each drawdown time analyzed. The k-C*

method developed by Kadlec and Knight (1996) and measured data from the wetland were used to calculate the effluent pollutant concentration, removal efficiency and the total annual TSS removed.

The results indicate that a drawdown time of 30 hours will provide the best removal efficiency while considering the site constraints. The installation of the WQCS will have an HRT of approximately 14 hrs and increase the removal efficiency by 14.2% and the total annual TSS removed by 31,100 lbs from existing conditions. Furthermore, the addition of the WCQS will only increase the maximum flooding depth and duration at the overflow locations by a maximum of 0.02 ft and 0.2 hrs, respectively, for the 100yr storm event. For the 2yr storm event, the addition of the WCQS will only increase the maximum flooding depth and duration at the overflow locations by a maximum of 0.01 ft and 0.1 hrs, respectively. The depth of water in the wetland, for both storm events analyzed, will not exceed the wetlands embankment at any location besides the overflow locations. At brimful conditions, the detained runoff water remains in the main channel and permanent pool areas of the wetland. The methods developed in this project can be used to retrofit an existing wetland with a WQCS that would maximize removal efficiency while considering site constraints.

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LIST OF KEY TERMS

BMP	Best Management Practice
C*	Irreducible Pollutant Removal Concentration
C_i	Influent Pollutant Concentration
City	City of Fort Collins
C_o	Effluent Pollutant Concentration
EMC	Event Mean Concentration
HLR	Hydraulic Loading Rate
HRT	Hydraulic Retention Time
HSB	Howes Street Basin
k	Rate Constant
TSS	Total Suspended Solids
SOL	Sum of Loads
UDFCD	Urban Drainage Flood Control District
Wetland	Howes Street Basin Wetland
WQCS	Water Quality Control Structure
WQCV	Water Quality Capture Volume

1.0 INTRODUCTION

Stormwater management has evolved over the years in response to the continued degradation of receiving waters. Urbanization alters the quantity and quality of stormwater runoff by changing the nutrient, chemical, metal, and organic loading rates and as a consequence affecting the hydrology and morphology of receiving waters (WEF and ASCE 1998). Non-point source pollutants and stormwater discharges are federally regulated by the Clean Water Act of 1972 and its amendments. Following the regulations, stormwater-best management practices (BMPs) were developed as a means to capture, mitigate and/or remove pollutants and excess runoff volumes. Along with BMPs, public education and involvement, low impact development strategies, and management of materials are all used as a means of stormwater pollution prevention and control (U.S. EPA 2009, WEF and ASCE 1998). Once pollutants become part of the stormwater runoff, BMPs are the last resort before runoff reaches receiving waters.

Numerous mathematical equations and models have been developed and refined for the design of BMPs. Two commonly used criterion for the design of a water quality BMPs are water quality capture volume (WQCV) and drawdown time. The WQCV is designed to capture and treat the stormwater runoff that is generated from a watershed for the 80th percentile rainfall event and smaller (UDFCD 2011). An outlet control structure of a BMP is designed to regulate the discharge rate and drawdown time of the BMP. The drawdown time is selected based on the BMP type and the desired particle settling time (Urbonas and Stahre 1993). In general, the longer the captured water is retained the greater the hydraulic residence time

(HRT) and improvement in treatment (Urbonas and Stahre 1993, Conn and Fiedler 2006, and Ghosh and Gopal 2010).

A BMP generally requires a relatively large area to accommodate water quantity and quality treatment volumes. In new developments land space can be allocated for larger BMPs. However, in many retrofit and redevelopment situations BMP design standards cannot easily be achieved because of land and property limitations. In instances where the entire WQCV cannot be captured and treated, the BMP design should focus on maximizing pollutant removal by utilizing the available basin volume and outlet controls. Hathaway and Hunt (2009) performed a study on the pollutant removal occurring in a wetland that was not large enough to contain the WQCV and concluded that an undersized wetland can provide improvement to the runoff water quality in urban watersheds. Ideally, a method that establishes a means to optimize pollutant removal for a BMP with volume restrictions would provide the best practical design for a site.

In this study, continuous simulation stormwater modeling and pollutant removal equations are used to design a water quality control structure (WQCS) for the Howes Street Basin wetland, referred to from here on as "the wetland". The wetland consists of three cells in a series that capture stormwater runoff from the Howes Street Basin, located in Fort Collins, Colorado, before discharging to the adjacent Poudre River. Currently, the wetland is considered an uncontrolled BMP because the outlet controls and wetland volume were not specifically designed for water quantity or quality control. The existing land use of the surrounding area and topography of the wetland do not allow for modifications to the dimensions of the wetland.

Stormwater runoff volumes and pollutant concentrations at the inlet and outlet were measured from 2009 to 2011 and the data were analyzed to determine the treatment efficiency of the wetland (Messamer, 2011). The study found measurable pollutant removal occurring within the wetland and prompted Messamer (2011) to recommend that the installation of a WQCS could increase the HRT and in turn increase the pollutant removal. The City of Fort Collins (City) requested a conceptual design proposal for a WQCS to be installed at the wetland outlet. The objective of this project is to design a water quality control structure that would maximize pollutant removal efficiency and the mass of total suspended solids (TSS) removed in the wetland without causing additional flooding at the site and adversely affecting the adjacent properties. An additional objective of this project was to develop a method to calculate the hydraulic retention time (HRT) for a stormwater wetland.

2.0 LITERATURE REVIEW

Several BMPs have been developed to mitigate the effects of increased urbanization and impervious surfaces within a watershed. Stormwater management methods are evolving from capturing runoff and reducing discharge rates for larger storm events to include actively reducing pollutant concentrations in captured runoff for smaller more frequent storm events. While the physical design is dictated by the type of BMP, the capture volume is dependent on watershed characteristics, rainfall rates, and assumed pollutant removal rates. The overall performance of a BMP is controlled by the designed capture volume and drawdown time. BMP performance is defined as the achievement of pollutant, volume, and flow reduction objectives.

2.1. Management of Stormwater Quality

The Environmental Protection Agency (EPA) is the primary federal regulator for stormwater discharges through the National Pollutant Discharge Elimination System (NPDES) permit program (U.S. EPA 2009). Urban water quality degradation prompted the implementation of the NPDES program as a means to provide regulation of point and nonpoint sources contributing pollutants to receiving waters from municipal separate sewer systems (MS4s). The EPA's multi-faceted stormwater management approach requires an MS4 to develop a program of action for stormwater mitigation from the source to receiving waters. The concept of BMPs for stormwater management has been in effect since the 1970s, but the practice of designing and implementing BMPs as a water quality control only started emerging in the 1990s (WEF and ASCE 1998). Water quality BMPs are a means of capturing and treating stormwater runoff not controlled at the source.

BMPs with water quality controls are considered volume-based structural BMPs because they store stormwater over a period of time before releasing it. The primary mechanism of treatment for volume-based BMPs is sedimentation. It is generally assumed that if a treatment method achieves settling of TSS, other pollutants will also settle out (Urbonas and Stahre 1993). The Urbonas and Stahre (1993) study established that most pollutants attach to smaller particles which take longer to settle than larger sediment particles. Design specifications and standards for structural BMPs are based on minimum drawdown times which will provide adequate settling rates and treatment. Additionally, drawdown times can be adjusted to attain desired effluent concentrations.

The drawdown time of a BMP is defined as the time it takes for the BMP to empty from brimful conditions. Brimful volume is not always met and sometimes exceeded because storm events produce variable inflow volumes to the BMP. Studies by Urbonas and Stahre (1993) outlined field and laboratory settling rates that are used to establish a minimum drawdown time for each BMP type. The measured pollutant settling rate data were used to determine the drawdown time required to achieve the desired average HRT over the event period. The HRT is the average time a particle of water spends in the BMP. Most dry BMPs have a designated minimum drawdown time of 40 hours, which produces an average HRT of approximately 24 hours (Urbonas and Stahre 1993). For BMPs with permanent pools the minimum drawdown time is reduced to 12 hours because the HRT of the effluent is increased by the existence of the permanent pool (UDFCD 2011). While using a minimum drawdown time is an acceptable design practice, the use of a longer drawdown time increases the time the stormwater is in the BMP and the probability of pollutants settling (Toet et al 2005). However, if the drawdown time is increased, the outlet discharge rate must be decreased and the BMP brimful volume increased to accommodate the WQCV.

2.1.1. Non-Steady State Flow

Stormwater runoff and flow through a wetland BMP exhibits non-steady state conditions. Flow rate variations into a BMP occur because storm events have fluctuating intensities, durations, and inter-event times (Werner and Kadlec 1996). The flora, channelization, ponding areas, and wetland layout alter the flow pattern within a wetland BMP. Existing models calculate treatment and flow within a BMP assuming steady state plug flow conditions because the equations were derived for wetlands treating wastewater flows (Kadlec and Knight 1996). Wastewater wetlands are assumed to have a constant influent and effluent flow rate and permanent pool volume. The assumed steady state conditions for both the influent flow and pollutant concentrations contradict stormwater runoff characteristics. However, in general the assumed steady state conditions are still used to calculate the HRT of the BMP using the following equation:

$$\mathrm{HRT} = \frac{\mathrm{V}}{\mathrm{Q}} * 3600 \qquad \textbf{Equation 2.1}$$

Where: HRT= Hydraulic retention time, hr

V= permanent pool volume or basin WQCV, ft^3

Q= flow rate, ft^3/yr

Additional methods to calculate the HRT for a stormwater BMP have been used that attempt to account for the stochastic nature of stormwater runoff. Wong et al (2004) used the average of the calculated time step HRTs to simulate contaminant reduction using the k-C* model. The k-C* model uses the influent flow rate and pollutant concentration, BMP surface area, and the wetland characteristics to calculate the effluent pollutant concentration (Kadlec and Knight 1996). Somes et al (2000) outlines and compares common practices used to calculate a flow weighted mean HRT. The two most common practices included calculating the time difference between the centriods of the inflow and outflow hydrographs and computing the ratio of the storage volume to the mean influent flow rate. Both methods do not acknowledge the stochastic nature of stormwater flow because the methods only use the totals or averages of the storm event flows in the calculations. Currently, there is no standard method to calculate the HRT for a proposed stormwater BMP without assuming steady state conditions.

2.1.2. BMP Sizing

BMPs are sized to capture the runoff volume of a specific storm event calculated from the storm precipitation depth and the watershed's characteristics. A BMP can be designed to capture and treat the largest storm event for a region. However, the allocated space for the BMP would be substantial in order to capture and treat the entire runoff volume and also provide treatment for smaller storm events. Smaller more frequent storms contribute a larger portion of the annual pollutant load in runoff than larger less frequent storm events and need to be included in the design considerations (WEF and ASCE 1998). If the WQCV for a BMP is sized based on a relatively small storm event, the majority of runoff from larger storms will overflow the BMP and not be treated. Therefore, it is important choose a WQCV that is large enough to not bypass most of the larger storm events, but not so large that the BMP cannot be installed due to size and cost constraints. Flood and discharge control for larger storms above the WQCV also need to be considered in the final design volume of the BMP. Urbonas et al (1990) developed a method to optimize the WQCV of a BMP that is both reasonably sized and provides adequate removal.

Urbonas et al's (1990) method uses rainfall depth data over a period of time to calculate the total runoff volume captured for a given basin volume. The total runoff volume from a watershed for a given storm event is estimated from the percent imperviousness, land use, and/or soil type of the watershed. Several sources have provided an accepted method to calculate the watershed's total runoff volume for a given storm event's precipitation depth (Urbonas et al (1990), the Urban Drainage Flood Control District's (UDFCD) Criteria Manual (2011), the EPA's Stormwater Management Model Application Manual (2009), and the Natural Resources Conservation Service's Technical Release (TR-55) Manual (1986)).

To calculate the optimized WQCV, using Urbonas et al's (1990) method, the total volume of runoff from a period of record is routed through a proposed basin volume and the total volume of water captured is calculated. This captured runoff volume is the water that exits through the basin outlet structure and receives treatment, while runoff that overflows the basin goes untreated. The basin volume is increased incrementally while the other independent variables; drawdown time, inter-event time, and volume of runoff for the period of record, are kept constant. To maintain a constant brimful drawdown time as the basin volume is increased, the outlet control discharge rate is also increased until the desired drawdown time is achieved. The total runoff volume captured is recalculated for each new basin volume. The basin volume is increased until its volume is equivalent to the 99.9% probability runoff event, the largest storm event runoff volume for the period of record. The basin volumes and subsequent total runoff capture volumes are used to develop a capture volume curve and ultimately determine the point of diminishing return of the capture volume, also known as the optimized capture volume.

Urbonas and Stahre (1993) developed a capture volume curve for a watershed in Denver, CO, using 36 years of rainfall data, Figure 2.1. The drawdown time, inter-event time, and maximum storm event runoff volume all remained constant at 12 hours, 6 hours, and the 99.9% probability runoff storm volume, respectively. Furthermore, the detention volume and runoff capture volume are normalized to allow the optimized capture volume ratio to be easily identified as the point on the curve where there is a 1:1 slope. The detention volume is the basin volume that is being simulated. The runoff capture volume is the volume of water that discharges through the basins outlet structure. The detention volume and the runoff capture volume are both normalized by dividing the proposed detention volume and runoff capture volume by the 99.9% probability runoff volume. The normalized detention volume is also referred to as the relative detention volume.

The optimized capture volume occurs at the point of diminishing return. As shown in Figure 2.1, the capture volume curve has a steep increasing slope below the optimized point and a gradual increasing slope after the optimized point, indicating that the use of a basin volume larger than the optimized volume will not yield a significant increase in the captured volume for the costs associated with a larger basin. Furthermore, storm events larger than the optimized volume occur less frequent meaning most of the basin volume would rarely be utilized. Basin volumes smaller than the optimized capture volume will yield similar treatment results as the optimized capture volume, but only for storm events that can be captured by the smaller basin. Furthermore, larger storm events that can be captured by the optimized capture volume will be bypassed and untreated by the undersize basin.

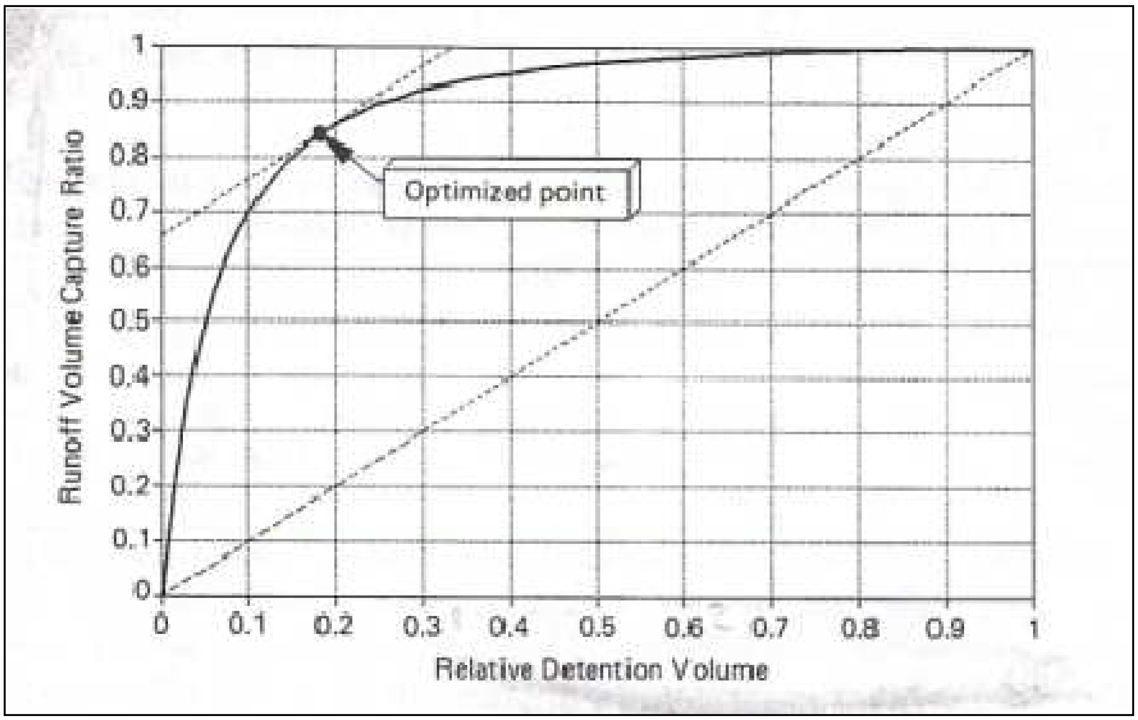

Figure 2.1 Optimizing the Capture volume (Urbonas and Stahre 1993)

Roesner et al (1991) used Urbonas et al's (1990) method to establish the capture volume curves for six study watersheds in various cities, Figure 2.2. Their results confirmed that a capture volume curve can be generated for a given watershed and used to determine an optimized WQCV. The optimized WQCV is equivalent to the basin detention volume at the optimized point.

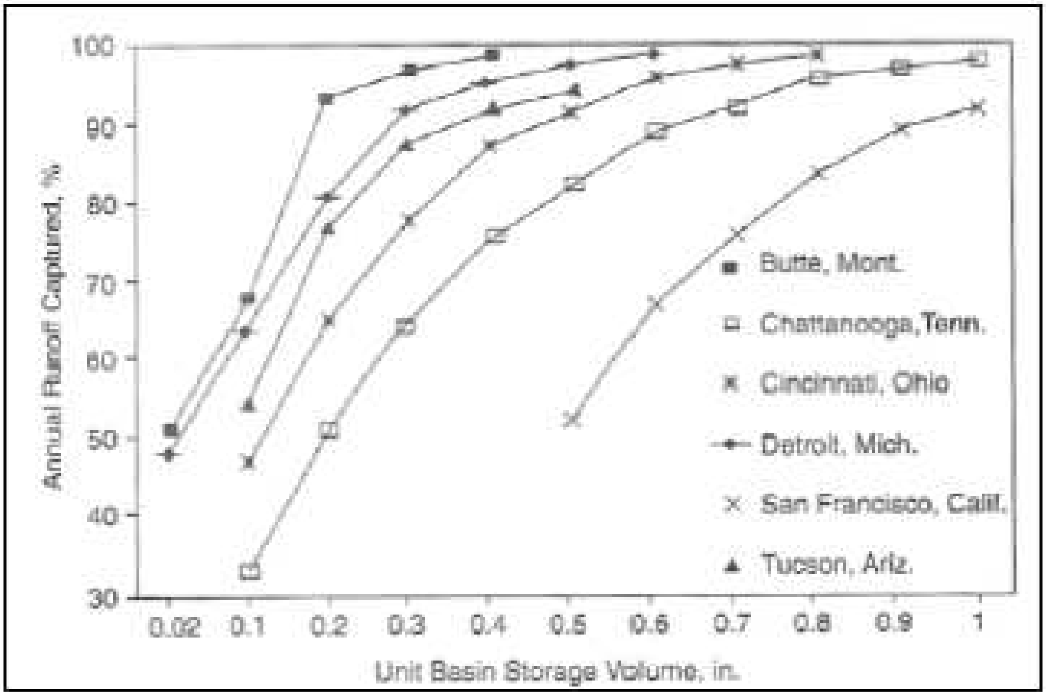

Figure 2.2. Runoff capture rates versus unit storage volume at six study sites (Roesner *et al.* 1991)

When precipitation and watershed data are not available to develop a capture volume curve, the optimized WQCV can be estimated using a predetermined runoff event's precipitation depth that is assumed to provide the optimized capture volume (UDFCD 2011 and Urbonas et al 1990). Guo and Urbonas (1995) developed a regression equation that relates average precipitation depth for a region to the optimized capture volume for the entire United States, Equation 2.2, Equation 2.3, and Figure 2.3. The regression equation assumes the 85th percentile runoff event provides the optimized capture volume and was used to calculate the regression constant for three drawdown times, Table 2.1. The percentile runoff event is identified by Roesner et al (1991) as the annual runoff captured and by Urbonas and Stahre (1993) as the runoff volume capture ratio on their respective capture volume figures, Figure 2.2 and Figure 2.1, respectively.

$$P_0 = (a * C) * P_6 \qquad \textbf{Equation 2.2}$$

$$C = 0.858i^3 - 0.78i^2 + 0.774i + 0.04 \qquad \textbf{Equation 2.3}$$

Where: P_0=optimize basin volume, watershed inches

a= regression constant

P_6= mean storm precipitation volume, watershed inches

C= watershed runoff coefficient

i= watershed imperviousness ratio, %/100

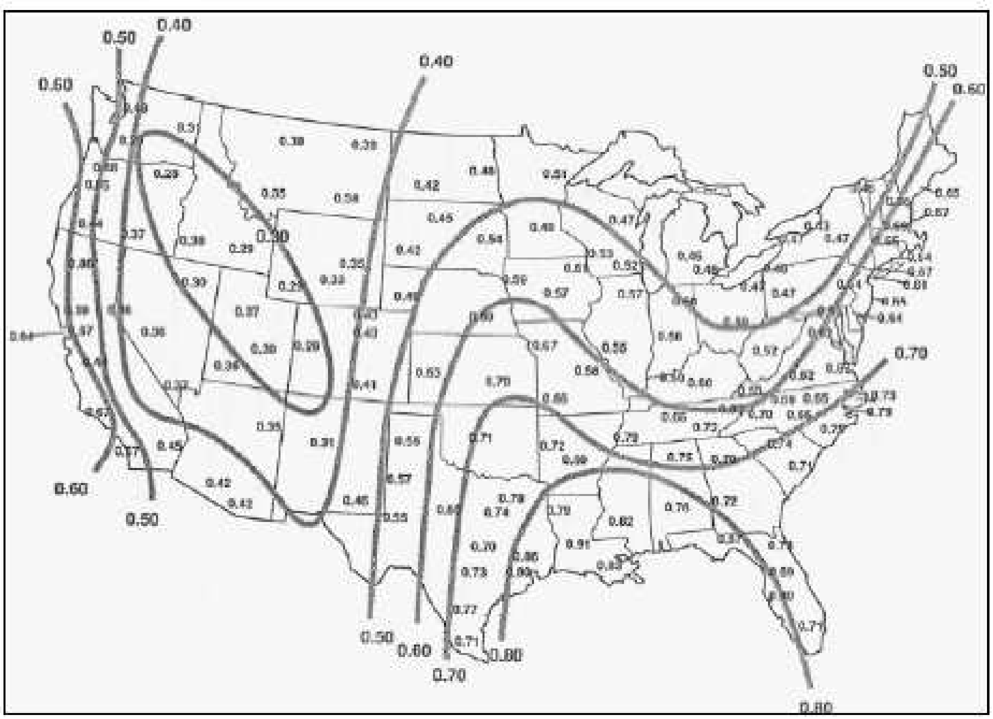

Figure 2.3. Map of the mean storm precipitation depth in the United States in inches (Discoll et al. 1989).

Table 2.1. Regression constant values for varying drawdown time based on the volume capture ratio (Guo and Urbonas 1995)

Drawdown Time (hr)	Coefficient, a
12	1.312
24	1.582
48	1.963

UDFCD Volume 3 (2011) used Guo and Urbonas (1995) method to develop WQCV calculations which can also be used for the entire United States. Analysis of capture volume curves and pollutant removal data for varying capture volumes resulted in the use of the 80^{th} percentile runoff event for the optimized WQCV calculations (UDFCD 2011 and Urbonas et al 1990). UDFCD's WQCV equation for the Denver region, Equation 2.4, uses a drain time coefficient and the percent imperviousness of the watershed to calculate the WQCV. Equation 2.5 is used for other locations throughout United States. The equation uses the map of the mean storm precipitation depth in the United States from Discoll et al. 1989, Figure 2.3, and the WQCV calculated from Equation 2.4.

$$WQCV = a(0.91i^3 - 1.19i^2 + 0.78i)$$ **Equation 2.4**

Where: WQCV=Water Quality Capture Volume, watershed inches

a= Coefficient corresponding to WQCV drain time (Table 2.2)

i= Imperviousness of watershed, %/100

Table 2.2 Drain time coefficients for WQCV calculations (UDFCD 2011)

Drain Time (hrs)	Coefficient, a
12	0.8
24	0.9
40	1.0

$$\mathrm{WQCV_{other}} = \mathrm{d_6} * \left(\frac{\mathrm{WQCV}}{0.43}\right) \qquad \textbf{Equation 2.5}$$

Where: $WQCV_{other}$=WQCV outside of the Denver region, watershed inches

WQCV= WQCV calculated from Equation 7, watershed inches

d_6= Depth of average runoff producing storm from Figure 2.3

2.2. BMP Performance

BMP design using the optimized capture volume technique has been shown to remove 80-90% of the annual TSS load from the captured runoff volume (Urbonas et al 1990, Urbonas and Stahre 1993). However, studies have shown that a storm events influent pollutant concentration greatly influences the performance of a BMP (Strecker et al 2001, Urbonas and Stahre 1993, Park and Roesner 2012). Specifically, runoff with larger influent pollutant concentrations has greater pollutant removal efficiency than runoff with smaller influent pollutant concentrations (Urbonas and Stahre 1993). Assuming the same effluent concentration, the runoff with higher influent concentrations will have a greater efficiency because there is more of the pollutant to remove. Generally, the knowledge that 80-90% pollutant reduction is occurring if the WQCV is captured and treated is acceptable. However, in situations where pollutant discharge concentrations are regulated, quantification of a BMP's performance during the design process is necessary.

Efficiency equations are used to analyze BMP performance and effectiveness, both of which are a measure of how well a BMP has met its pollutant removal objectives. BMP effectiveness differs from BMP performance by including an analysis of the bypassed flow as well as an analysis of the captured and treated flow (Strecker et al 2001). Pollutant removal efficiency is usually calculated using one of the following three methods: a statistical characterization of influent and effluent concentrations, a comparison of total influent and effluent loads, or the percent removal by storm event (Strecker et al 2001). Both the statistical characterization and comparison methods acknowledge the effect of influent concentration on removal efficiency by analyzing the influent and effluent storm event data for the period of record as a whole. The percent removal by storm event method calculates the event period average efficiency using the estimated efficiency of each storm event.

Storm events with low influent pollutant concentration usually have a low efficiency because there is less pollutant in the runoff to be removed. The variability in the storm event efficiencies skews the event period average efficiency. Furthermore, discharge concentration standards are generally achieved for storms with low influent concentrations, but this fact can be overlooked when only analyzing the BMP efficiency. Strecker et al (2001) indicates that using total influent and effluent loads for the efficiency analysis is adequate; provided several storm events are used in the analysis. Gulliver et al (2010) and Geosyntec et al (1999) recommends two methods for calculating long-term efficiency; using the average influent and effluent event mean concentrations (EMC), Equation 2.6, or using the sum of influent and effluent loads, which was also suggested by Strecker et al (2001), Equation 2.7.

$$\text{Efficiency Ratio} = 1 - \frac{\text{Average EMC}_{\text{out}}}{\text{Average EMC}_{\text{in}}}$$ **Equation 2.6**

$$\text{Summation of Loads Efficiency} = 1 - \frac{\sum \text{Effluent Loads}}{\sum \text{Influent Loads}}$$ **Equation 2.7**

The influent and effluent EMCs and summation of loads (SOLs) values are calculated using Equation 2.8 and Equation 2.9. The efficiency ratio uses the arithmetic mean of the EMCs for the period of record.

$$\mathrm{EMC_i} = \frac{\sum_{i=1}^{n}(V_i * C_i)}{\sum_{i=1}^{n}(V_i)}$$ **Equation 2.8**

$$\text{Sum of Loads} = \sum_{\mathrm{j}=1}^{\mathrm{m}} \mathrm{EMC_j} * \mathrm{V_j}$$ **Equation 2.9**

Where: EMC_i= event mean concentration during an event period, i

C_i= average concentration associated with period i

V_i= volume of flow during an event period, i

n= total number of measurements taken during an event

EMC_j= event mean concentration during entire period, j

V_j= volume of flow during entire period, j

m= number of events measured

These efficiency methods have limitations associated with their use. The principle deficiency for the efficiency ratio method is that all storms are considered equal and weighted equally regardless of the magnitude of the storm event and influent loading. The SOL efficiency method assumes that the mass removed during a single event is less important than the total mass removed for the period of analysis. For both methods, the BMP performance for a single storm event may not have the same efficiency as reported for the period of record because removal is dependent on the pollutant influent concentration, hydraulic loading rate, and the BMP characteristics. The efficiency ratio approach lacks the necessary detail for an event based analysis especially if discharge standards are in place. Overall, both methods are appropriate when considering the long term efficiency of a BMP.

2.3. Pollutant Concentrations

Several factors influence pollutant concentrations entering a BMP including watershed characteristics, storm intensity, inter-event time, and climate (Park and Roesner 2012, Kadlec and Wallace 2009 and Kadlec 1997). Variability between and during storm events compels the use of a long term average EMC to establish pollutant loadings in stormwater runoff. Measured data of runoff pollutant loadings for a specific watershed and for an entire storm event is usually not readily available for a site. Kadlec and Wallace (2009) compiled data from multiple studies and created tables presenting composite stormwater mass loading rates and long-term mean pollutant concentrations for source areas and in stormwater runoff, shown in Table 2.3 and Table 2.4. Influent EMCs can be estimated using the documented data tables in conjunction with watershed characteristics.

Table 2.3. Composition and mass loading rates for stormwater (taken from Kadlec and Wallace (2009), Table 16.8)

Constituent	Urban		Industrial		Residential/Commercial		Agricultural	
	Concentration (mg/L)	Load (kg/ha·yr)	Concentration (mg/L)	Load (kg/ha·yr)	Concentration (mg/L)	Load (kg/ha·yr)	Concentration (mg/L)	Load (kg/ha·yr)
BOD_5	20 (7-56)	90	9.6	34-98	3.6-20	31.59-135.2	3.8	11.59
COD	75 (20-275)	-	-	-	-	-	-	-
TSS	150 (20-2890)	360	93.9	672-954.5	18-140	84.28-797	55.3	24.14
VSS	88 (53-122)	-	-	-	-	-	-	-
NH_3N	0.582	-	-	-	-	-	0.33-0.48	-
TKN	1.4 (0.57-4.2)	-	-	-	-	-	2.16-2.27	-
TN	2.0 (0.7-20)	11.2	1.79	7.8-18.06	1.1-2.8	9.144-32.18	2.32	10.61
Ortho-P	0.12	-	0.13	1.321	0.05-0.40	0.568-3.302	0.13-0.227	0.942
TP	0.36 (0.02-4.3)	3.4	0.31	2.2-3.151	0.14-0.51	1.412-4.85	0.344	1.362
Copper	0.05 (0.01-0.40)	0.049	-	0.077	-	0.045	-	-
Lead	0.18 (0.01-1.20)	0.174	0.202	0.269-2.053	0.065-0.214	0.157-2.431	-	-
Zinc	0.20 (0.01-2.9)	0.63	0.122	0.98-1.240	0.046-0.170	0.218-1.88	-	-
Chromium	-	0.28	-	0.044	-	0.026	-	-
Cadmium	0.0015	0.16	-	0.024	-	0.013	-	-
Iron	8.7	-	-	-	-	-	-	-
Mercury	0.00005	0.043	-	0.065	-	0.038	-	-
Nickel	0.022	0.032	-	0.030	-	0.029	-	-
Oil and Grease	2.6	-	-	-	-	-	-	-

Table 2.4. Pollutant concentrations for source area for stormwater (taken from Kadlec and Wallace (2009), Table 14.2)

Constituent	TSS (mg/L)	TP (mg/L)	TN (mg/L)	*E coli* (1,000 #/mL)	Cu (µg/L)	Pb (µg/L)	Zn (µg/L)
Residential roof	19	0.11	1.5	0.26	20	21	312
Commercial roof	9	0.14	2.1	1.1	7	17	256
industrial roof	17	-	-	5.8	62	43	1390
Comm./res. Parking	27	0.15	1.9	1.8	51	28	139
Industrial parking	228	-	-	2.7	34	85	224
Residential street	172	0.55	1.4	37	25	51	173
Commercial street	468	-	-	12	73	170	450
Rural highway	51	-	22	-	22	80	80
Urban highway	142	0.32	3	-	54	400	329
Lawns	602	2.1	9.1	24	17	17	50
Landscaping	37	-	-	94	94	29	263
Driveway	173	0.56	2.1	17	17	-	107
Gas station	31	-	-	-	88	80	290
Auto recycler	335	-	-	-	103	182	520
Heavy industrial	124	-	-	-	148	290	1,600

Influent EMCs can be used to estimate a BMPs long-term effluent EMCs using a pollutant removal model. A common model used is the first-order k-C* model proposed by Kadlec and Knight (1996) and originally developed for the analysis of constructed wetlands treating wastewater discharge. The model assumes steady state and plug flow conditions involving two parameters: a rate constant (k) and the irreducible background concentration (C*), Equation 2.10 (Kadlec and Knight 1996). Wong and Geiger (1997) addressed the stochastic nature of stormwater runoff to adapt the k-C* model for stormwater analysis and suggest the use of a pilot study for a specific site to calibrate the model variables, k and C*, before implementation of the BMP.

$$\frac{(C_o - C^*)}{C_i - C^*} = e^{(\frac{-k}{q})}$$ **Equation 2.10**

$$q = \frac{Q}{A}$$ **Equation 2.11**

Where: C_i= influent concentration, lb/ft^3

C_o= effluent concentration from the outlet orifice, lb/ft^3

C*= irreducible background concentration, lb/ft^3

k= areal rate constant, ft/yr

q= hydraulic loading rate, Equation 2.11, ft/yr

Q= influent flow rate, ft^3/yr

A= BMP surface area, ft^2

Major factors in determining the k and C* values are the influent pollutant concentration, hydraulic loading rate, and the physical and ecological characteristics of the BMP (Schueler 1996; Wong and Geiger 1997; Kadlec 2000). The k value characterizes the physical and ecological properties of the BMP. The C* is the pollutant concentration that cannot be removed from the runoff discharge no matter how large the HRT (Schueler 1996; Wong and Geiger 1997; Minton 2005; Kadlec 2000). Schueler (1996) used data from multiple studies including the National Urban Runoff Program (NURP) study and Kehoe et al (1994) study of stormwater ponds and wetlands in the Tampa Bay Florida area to calculate an average C* value for several stormwater runoff water quality parameters, Table 2.5.

Table 2.5. Irreducible concentrations in wastewater wetlands and stormwater management practices (taken from Schueler (1996), Table 1)

Water Quality Parameter	Wastewater (mg/L)	Wastewater (mg/L)	Stormwater Practices (mg/L)
Total Suspended Solids	2 to 15	8	20 to 40
Total Phosphorus	0.02 to 0.07	0.5	0.15 to 0.2
Total Nitrogen	1.0 to 2.5	1.0	1.9
Nitrate-Nitrite	0.05	0.00	0.7
TKN	1.0 to 2.5	1.0	1.2

Uncertainty in both the k and C* values occurs in treatment wetlands because of the variation in the characteristics and processes throughout the wetland. In stormwater treatment wetlands the variant influent hydraulic loading rates and pollutant concentrations add additional uncertainty and variability (Kadlec 1997, 2000; Wong et al 2004; Wong and Geiger 1997). The k-C* model is currently used to produce initial effluent pollutant concentration estimates for long-term analysis of pollutant removal for conservative stormwater BMP design practices using constant k and C* values (Wong and Geiger 1997).

The effluent concentrations calculated by the k-C* model do not account for the pollutant concentration of the untreated bypassed flow which occurs during storm events with larger runoff volumes than the WQCV. Therefore, the use of a blending mass balance equation is required to determine the collective discharge pollutant concentration that reaches receiving waters, Figure 2.4 and Equation 2.12 (Kadlec 2000). The k-C* model can be used in conjunction with Equation 2.12 to estimate the performance and efficiency of a BMP. The combined estimation of the bypassed and treated flows provides a more accurate estimation of discharge pollutant concentrations than the singular use of the k-C*model which ignores the impact of bypassed flows.

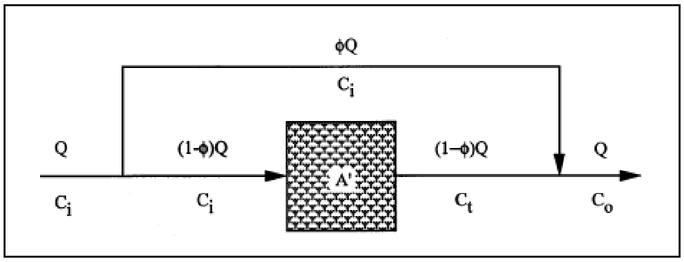

Figure 2.4. Zero treatment bypass of flow, (taken from Kadlec (2000), Figure 1)

$$\mathbf{C_o = \phi C_i + (1 + \phi) C_t} \qquad \textbf{Equation 2.12}$$

Where: C_o= final discharge effluent concentration, lb/ ft^3

C_t= treatment concentration from the outlet orifice, lb/ft^3

Φ= fraction of flow bypassed with no treatment

3.0 SITE DESCRIPTION

The Howes Street Basin (HSB) located in Fort Collins, CO is approximately 505 acres of residential (80%), commercial (10%), and open space (10%) draining to a 7 acre wetland, Figure 3.1. The watershed can be separated into two sections, an upper and lower, based on the land use and stormwater runoff conveyance system. The upper portion of the watershed is primarily low density residential and open space land use and has long stretches of gutters before stormwater runoff enters the pipe system. The lower portion of the watershed consists of medium density residential and commercial land use and has shorter gutter sections to quickly move the stormwater runoff into the pipe system.

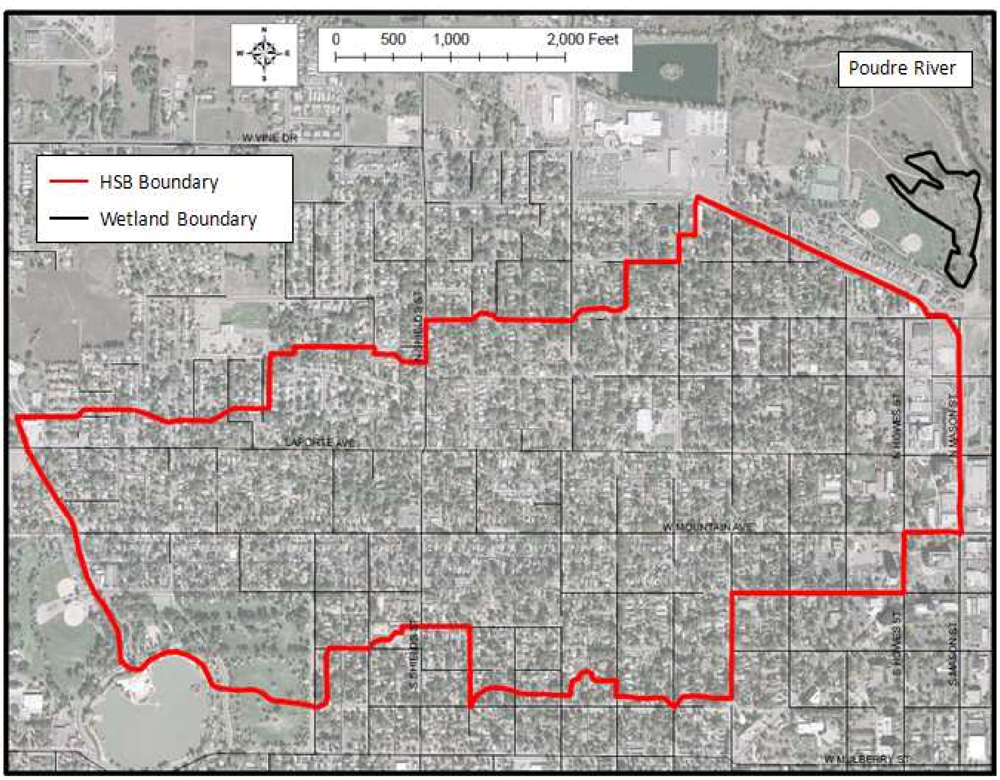

Figure 3.1 Howes Street Basin and Howes Street Basin wetland boundaries (Fort Collins Utilities 2012

The wetland is split into three different cells, Cell 1, Cell 2, and Cell 3, which have a mix of open space, large trees, tall grasses, cattails, manicured grass areas, channelization, and permanent pool areas, Figure 3.2. The wetland has a total length to width ratio of 1.6 ft:1 ft and the wetland channel and permanent pool area is L-shaped. Observations of the wetland during varying seasons indicated that there is continuous baseflow and permanent pools year-round. The permanent pools are found along the channels and at the outlet structures in all three cells. The permanent pool area of the wetland is approximately 5.6% of the total area and is surrounded by large trees with an undergrowth of downed trees, tall grass ranging from 1 ft to 4 ft in height, and cattails. Approximately 51.5% of the wetland area contains large trees. The cattails located near the permanent pools and channel make up 14% of the undergrowth. Tall grasses and manicured grasses are also located around the edges of the wetland at 41.9% and 1.0% of the wetland area, respectively.

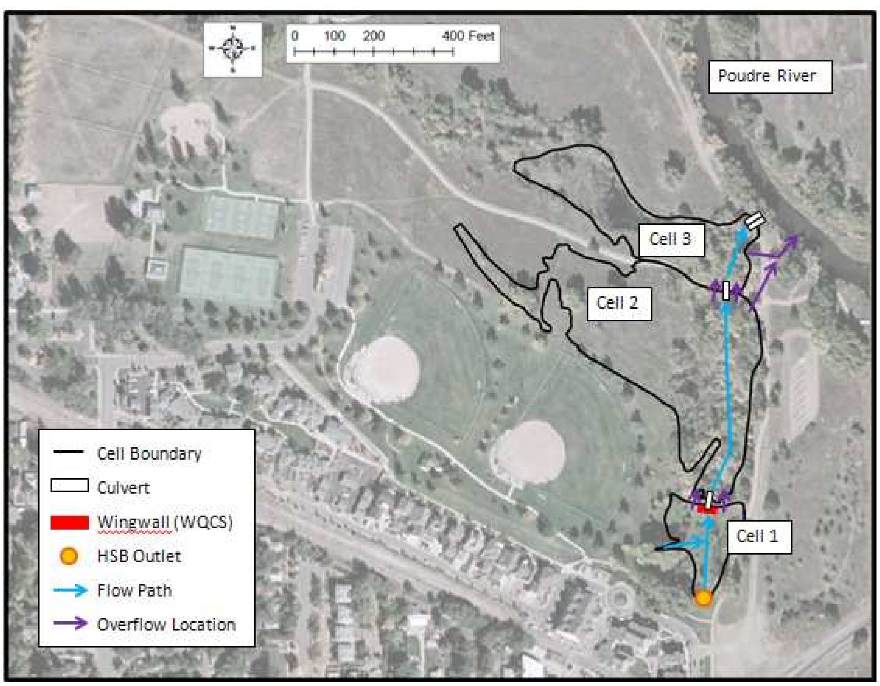

Figure 3.2. The Howes Street Basin wetland layout with flow pathways (Fort Collins Utilities 2012)

The stormwater runoff from the HSB discharges into Cell 1 through a 20ft wide double box rectangular culvert, Figure 3.3. A grass swale also discharges runoff from a small section of the adjacent park into Cell 1, shown as a flow path in Figure 3.2. An existing WQCS at the outlet serves as a means to detain and attenuate incoming flow through the rest of the wetland, Figure 3.4. The WQCS design includes a headwall and wingwall structure, with a 1.5ft by 1ft cutout in the wingwall structure and a slide gate in place to control flow. A 6ft by 3ft culvert connects Cell 1 to Cell 2 downstream of the WQCS.

Figure 3.3. The Howes Street Basin main outlet to the wetland, a 20ft by 20ft double box culvert

Figure 3.4. The Howes Street Basin wetland water quality control structure located at the outlet of Cell 1

Cell 2 is the largest cell and is the result of the construction of a bike path through the wetland, Figure 3.5. The western boundary of Cell 2 encroaches on the open space of the adjacent park. A permanent pool, with a depth of approximately 1ft, exists along the flow channel and at the outlet structure of the cell. Long-term erosion within the cell has further shaped the channel and permanent pool, creating areas of bare soil along the base and bank of the channel. A 6ft by 3ft culvert connects Cell 2 to Cell 3 and acts as a control structure for Cell 2.

Figure 3.5. Bike path that bisects the wetland into Cell 2 and Cell 3.

As the final section of the wetland, Cell 3 contains the outlet culverts that discharge to the Poudre River; a 2.5ft circular concrete pipe and a 3.5ft elliptical concrete pipe, Figure 3.6 . The elliptical pipe is offset 0.65ft above the circular pipe. The permanent pool encompasses most of the cell floor surface, with a depth of approximately 1.5 ft.

Figure 3.6. The Howes Street Basin wetland outlet culverts to the Poudre River

Large influent flow rates can create successive overflows through the cells and eventually to the Poudre River, Figure 3.7. Overflows in the wetland occur at low spots in the embankment, shown in Figure 3.2 as purple arrows. Overflows between cells are located at the outlet culverts of Cell 1 and Cell 2. Bypass overflows to the Poudre River are located along the eastern embankment of Cell 2 and Cell 3.

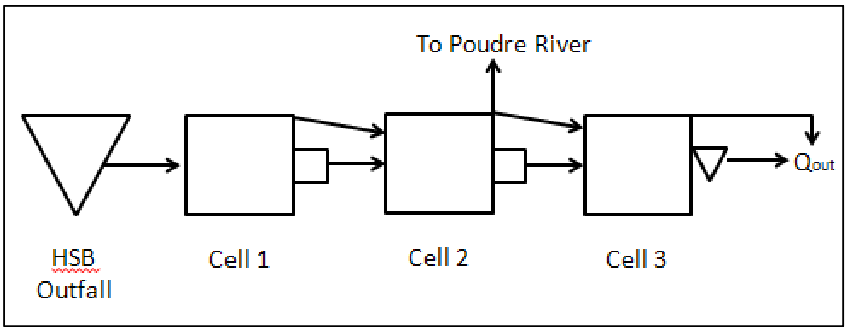

Figure 3.7 The Howes Street Basin wetland profile of the flow pathways through each cell (not to scale)

4.0 METHODS

The purpose of this project was to create a method to design a WQCS for the existing wetland which would maximize the BMP performance while acknowledging the site constraints. A model was created of the HSB and wetland to analyze the existing conditions, the proposed WQCSs, and provide data for the BMP performance calculations. The BMP performance was assessed by calculating the removal efficiency and the total annual TSS removed using measured data from the wetland (Messamer 2011) and the k-C* model (Kadlec and Knight 1996).

4.1 Storm Sewer System Model

The US EPA's Stormwater Management Model Version 5 (SWMM) was used to model the HSB and the wetland for both the existing conditions and the proposed WQCS designs. SWMM was used for its ability to run continuous simulation, hydrologic processes, and hydraulic flow routing. Continuous simulation modeling was required because the existing wetland volume limitations prohibit the use of UDFCD's WQCV design method. Also, City code requires the use of SWMM to calculate runoff quantities when the area of the watershed is greater than 90 acres (Fort Collins 2011).

4.1.1 Watershed and Drainage

The watershed data used to develop the SWMM model were obtained from the City's MODSWMM, AutoCAD and GIS files (Fort Collins 2012), Stormwater Criteria Manual (Fort

Collins 2011), and the SWMM Application and User Manuals (US EPA 2010). The MODSWMM files and AutoCAD and GIS maps were used to identify 28 sub-basins within the HSB and supply SWMM parameter values, Figure 4.1. The SWMM parameter values include: area, slope, width, percent impervious area, infiltration constants, and the impervious and pervious Manning's n and depression storage values (US EPA 2010).

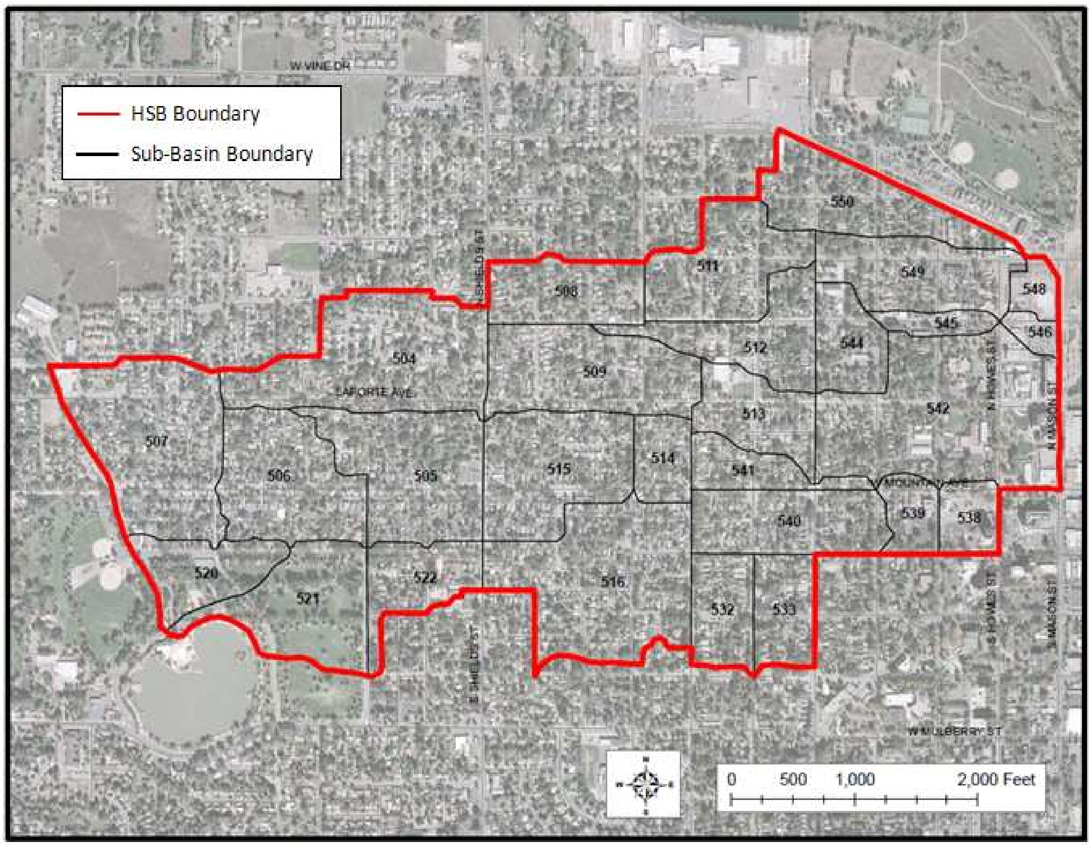

Figure 4.1. Delineation of the Howes Street Basin into the 28 sub-basins.

Calculations for the percent impervious area are based on the type of imperviousness; gross imperviousness (all impervious surfaces) or effective imperviousness (connected impervious surfaces). The original percent impervious values for the sub-basins were taken from the City's MODSWMM files which assume the watershed is comprised of all connected impervious surfaces. Observations of the residential areas in the HSB indicated that residential roof drains are generally unconnected, with runoff discharging to pervious surfaces rather than to driveways or roadways. Therefore, only the driveways, sidewalks, and streets are connected impervious surfaces and roof runoff should not be included in the imperviousness calculations for the residential sub-basins. It was also observed that commercial runoff is directly connected to the stormwater sewer system. Using either gross or effective percent imperviousness can cause variation in the calculated runoff volume. Therefore, to represent the actual watershed conditions, the MODSWMM residential sub-basins gross percent imperviousness values were adjusted manually to effective percent imperviousness.

Aerial imagery and GIS files (Fort Collins 2012) were used to analyze sample areas within the HSB to estimate the average percent effective imperviousness for residential lots. The percent effective imperviousness was calculated as the percentage of the lot that is directly connected to the storm drainage system: the driveway, sidewalk, and roadway. The percent impervious area for sub-basins with only residential land use decreased from 50% to 32%. For mixed land use sub-basins, the percent impervious area was amended by first calculating the existing area of each land use type in a sub-basin. The weighted average percent imperviousness was then calculated using the land use areas, the residential effective imperviousness, and the commercial and/or open space percent gross imperviousness. The percent gross imperviousness and the percent effective imperviousness values for each land use type are shown in Figure 4.1, while sub-basin areas, land uses and percent imperviousness values are shown in Table 4.2.

Table 4. 1. Percent effective and gross imperviousness based on land use for the Howes Street Basin's sub-basins.

Landuse	Percent Gross Impervious	Percent Effective Impervious
Residential	50%	32%
Commercial	90%	90%
Open Space	20%	20%
Mixed Residential and Commercial	70%	61%
Mixed Residential and Minimal Commercial	55%	37%
Mixed Condensed Residential and Commercial	85%	76%

Table 4.2. The Howes Street Basin's sub-basin identification numbers, areas, land use types, and percent effective impervious values.

Sub-basin ID	Area (acre)	Land Use Type	Effective Impervious
504	36.1	Residential	32%
505	27.4	Residential	32%
506	24.8	Residential	32%
507	35.6	Residential	32%
508	15.1	Residential	32%
509	26.0	Residential	32%
511	21.1	Residential	32%
512	16.6	Residential	32%
513	13.3	Residential	32%
514	7.5	Residential	32%
515	25.3	Residential	32%
516	38.8	Residential	32%
520	12.9	Open Space	20%
521	22.0	Open Space	20%
522	12.4	Residential	32%
532	10.0	Residential	32%
533	10.5	Residential	32%
538	6.6	Commercial	90%
539	6.3	Commercial, Residential	76%
540	18.9	Residential	32%
541	8.5	Residential	32%
542	47.4	Commercial, Residential	61%
544	10.4	Commercial, Residential	37%
545	4.1	Commercial, Residential	76%
546	2.7	Commercial	90%
548	3.8	Commercial	90%
549	19.8	Commercial, Residential	37%
550	21.0	Residential	32%

Sub-basin input parameters and the Horton infiltration equation constants that were not supplied by the MODSWMM files were taken from the City's Stormwater Criteria Manual (2011), Table 4.3. Only one inconsistency, the Manning's n value for pervious surfaces, was identified to have a different value in the Criteria Manual than the MODSWMM data. The manual suggests using a Manning's n of 0.025 while the MODSWMM model used 0.25. In the SWMM User's Manual (2010), the Manning's n values of 0.025 and 0.25 correspond to cement rubble and dense grass, respectively. To be consistent with the City's MODSWMM model, 0.25 was used for the Manning's n value for pervious surfaces in the SWMM model.

Table 4.3. SWMM input parameters from the City of Fort Collins Stormwater Criteria Manual Table RO-13.

Depth of Storage on Impervious Areas	0.1 inches
Depth of Storage on Pervious Areas	0.3 inches
Maximum Infiltration Rate	0.51 inches/hour
Minimum Infiltration Rate	0.50 inches/hour
Decay Rate	0.0018 inches/sec
Zero Detention Depth	1%
Manning's n Value for Pervious Surfaces	0.025
Manning's n Value for Impervious Surfaces	0.016

The stormwater runoff routing system in SWMM was developed using the SWMM User's manual (US EPA 2010), the Howes Street Outfall construction plans (Fort Collins 2000), and AutoCAD and GIS maps (Fort Collins 2012). The data for the storm sewer system in the lower portion of the watershed was obtained from the Howes Street Basin construction plans, the black pipes in Figure 4.2 (Fort Collins 2000). The stormwater runoff routing system for the upper portion of the watershed was modified to simplify the model and decrease run times, the red pipes in Figure 4.2. By changing the existing combined gutter and storm sewer system to a system consisting of three foot concrete circular pipes, the stormwater runoff is quickly directed through the upper portion of the watershed without losing any runoff volume. When data were available, any existing pipe slopes were used; otherwise, the slope of the ground surface was used as the pipe slope. Pipe lengths were determined using the length function in GIS.

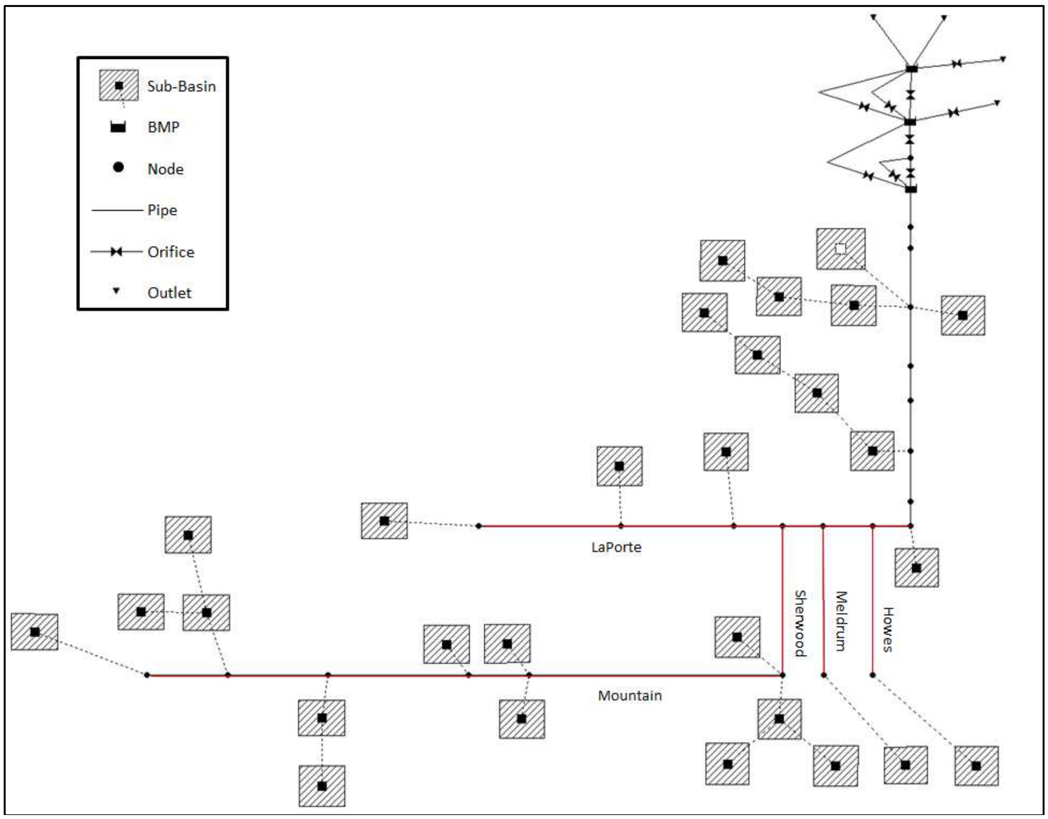

Figure 4.2. The Howes Street Basin's SWMM model layout including the connection locations of the sub-basins, the pipe routing system, and the existing wetland layout. A red pipe indicates the pipe was modified from the existing storm sewer system. A black pipe indicates no change from the existing storm sewer system.

4.1.2 Existing Wetland Layout

The SWMM wetland model included the three wetland cells, Cell 1, Cell 2 and Cell 3, and their outlet culvert(s) and overflow weir(s), Figure 3.2 and Figure 4.3. The stage storage curves for each cell were developed using depths and elevations from survey data, construction plans, and a topographic map, Table 4.4 (Fort Collins 2009 and Fort Collins 2012). Locations, elevations, and lengths of the outlet culverts and overflow weirs were also measured in the field to confirm the construction plans inlet and outlet rim and invert elevations, Table 4.4 (Fort Collins 2000 and Fort Collins 2012). The culverts connecting each cell were modeled as orifices, the outlet culverts discharging to the Poudre River were modeled as conduits, and the overflow weirs were modeled as weirs. The slide gate for the WQCS in Cell 1 was assumed to be always fully open and was modeled as 1.5ft by 1ft rectangular culvert, Figure 4.3.

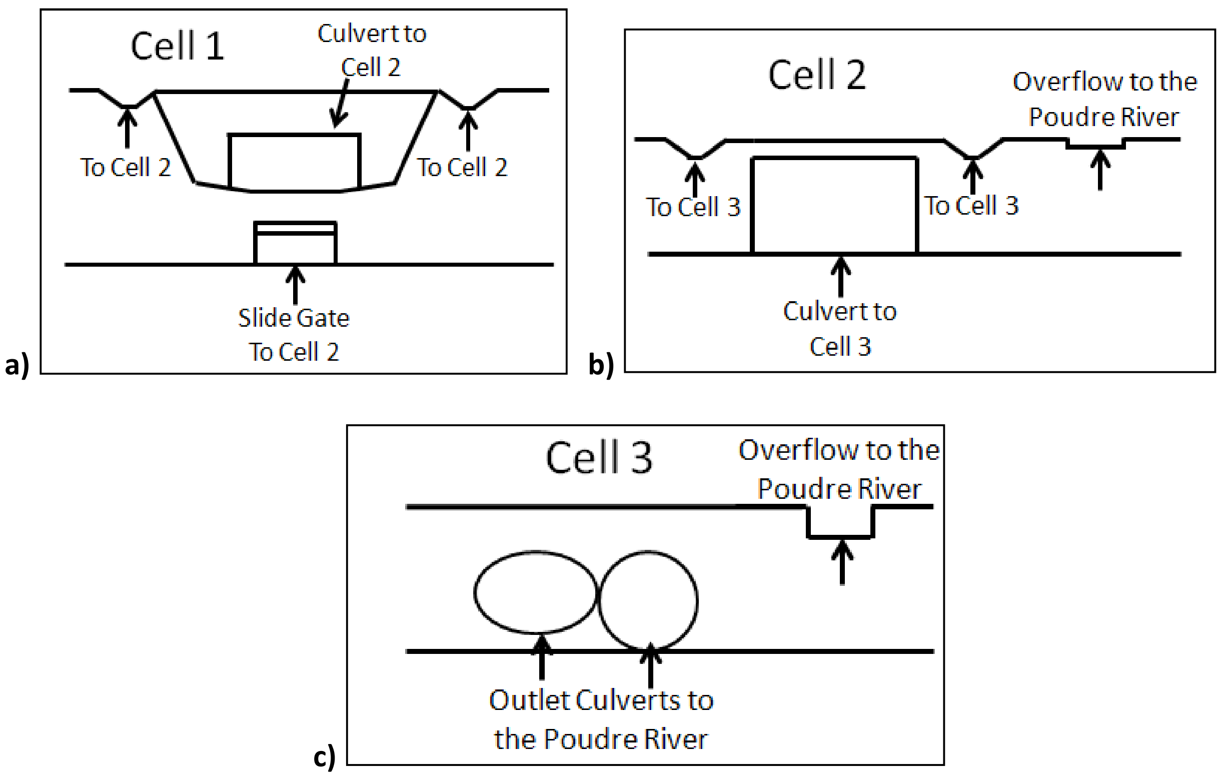

Figure 4.3. Sketches of the wetland's current outlets that are included in the SWMM model (not to scale): a) Cell 1's WQCS, culvert, and overflow weirs b) Cell 2's culvert and overflow weirs, and c) Cell 3's culverts and overflow weir.

Table 4.4. SWMM wetland and overflow weir characteristics

	Wetland Details				Overflow Weir(s)			
	Elevation	Depth (ft)	Area (ft^2)	Volume (ft^3)	Elevation	Depth To Top (ft)	Length (ft)	Overflow Direction
Cell 1	4960	0	770	0	4963	1.5	120	Cell 2
	4961	1	2766	1768				
	4962	2	7581	6942				
	4963	3	14447	17955				
	4964.5	4.5	30035	51317				
Cell 2	4958	0	2170	0	4960.6	0.5	50	Cell 3
	4959	1	7934	5052	4961.1	0.9	284	Cell 3
	4960	2	20577	19307	4961.2	0.8	175	Poudre River
	4961	3	87442	73317				
	4962	4	189613	211845				
Cell 3	4957.8	0	14097	0	4960.8	1.2	20	Poudre River
	4959	1.2	20984	21049				
	4960	2.2	27546	45314				
	4961	3.2	55132	86653				
	4962	4.2	83330	155884				

4.1.3 Model Simulation

Two separate model simulations were performed for the existing conditions and proposed WQCSs; continuous simulation of a year of rainfall data and single storm simulation of the 2yr and 100yr storm events. The SWMM models were run using dynamic wave flow routing to allow for diversions within the routing system, flow reversal, and backwater effects between the three interconnecting wetland cells. The routing and runoff time steps were set at one second in order to reduce flow routing continuity errors and stability issues, which arise from the use of the dynamic wave flow routing model. For the watershed runoff calculations, the monthly average evaporation rates were included in the model, Table 4.5 (US EPA 2009).

Table 4.5. Monthly average evaporation values for the City of Fort Collins from the SWMM Applications Manual Table 9-1 (2009).

	Jan	Feb	Mar	April	May	June	July	Aug	Sept	Oct	Nov	Dec
Monthly Average Rate (in/day)	0.00	0.00	0.06	0.11	0.12	0.15	0.16	0.14	0.11	0.07	0.03	0.00

Evaporation was not included in the wetland cells because it can cause depth and volume variation in the permanent pools. Baseflow through the wetland was observed to be year-round. Therefore, the assumption was made that any evaporation of the permanent pool volume would be replenished by the baseflow and the wetland cells can be modeled as dry basins. The cell base elevation was modeled starting at the surface of the permanent pool.

Continuous simulations were run for the year 2009, using 10-minute precipitation data obtained from the Colorado State University weather station (CCC), located approximately 2 miles east of the HSB, Table 4.6 (Colorado State University 2012). Only one year of rainfall data was simulated due to the model run times and the output data volume. The 2009 rainfall data were used because there was a variation of storm event precipitation depths in 2009, including a large storm event that would flood the wetland.

Table 4.6. 2009 Fort Collins Rainfall Events

Total Rainfall Depth (inches)	Number of Events	Percent of Total Storm Events	Percentile of Runoff Producing Storms*
0 to 0.1	60	58.3%	0%
0.1 to 0.5	31	30.1%	72.1%
≥0.6	93	90.3%	76.7%
0.5 to 1.0	9	8.7%	20.9%
1.0 to 1.5	1	1.0%	2.3%
1.5 to 2.0	0	0%	0%
2.0 to 3.0	1	1.0%	2.3%
3.0 to 4.0	1	1.0%	2.3%
4.0 to 5.0	0	0%	0%
>5.0	0	0%	0%
TOTAL :	103	100%	100%

*Runoff producing storms are assumed to be storms greater than 0.1" (Urbonas et al. 1989)

To abide by City requirements, a flood analysis of the current wetland layout and the proposed WQCS drawdown times were computed for the 2yr and 100yr storm events (Fort Collins 2011). Furthermore, an objective of this project was to confirm that the proposed WQCS would not greatly increase flooding duration and depth at the overflow locations or effect public safety. The City provides storm event design curves in its Stormwater Criteria Manual, Table 4.7 (Fort Collins 2011). Using the SWMM output data, the changes in flood depth and duration at overflow locations and adjacent properties were analyzed.

Table 4.7. City of Fort Collins 2yr and 100yr storm event design curves taken from the City's Stormwater Criteria Manual (City of Fort Collins 2011).

TIME	Design Storm	
	2yr	100yr
0:00	0.00	0.00
0:05	0.29	1.00
0:10	0.33	1.14
0:15	0.38	1.33
0:20	0.64	2.23
0:25	0.81	2.84
0:30	1.57	5.49
0:35	2.85	9.95
0:40	1.18	4.12
0:45	0.71	2.48
0:50	0.42	1.46
0:55	0.35	1.22
1:00	0.30	1.06
1:05	0.20	1.00
1:10	0.19	0.95
1:15	0.18	0.91
1:20	0.17	0.87
1:25	0.17	0.84
1:30	0.16	0.81
1:35	0.15	0.78
1:40	0.15	0.75
1:45	0.14	0.73
1:50	0.14	0.71
1:55	0.13	0.69
2:00	0.13	0.67

4.1.4 Model Calibration

In order to assess the SWMM model watershed parameter values and calibrate the model, the SWMM total runoff volumes out of the wetland were compared to measured volumes using the percent error. The measured data were taken from the 2010 and 2011 monitoring project on the HSB reported by Messamer (2011). ISCO samplers were used to measure the flow and depth of the runoff into and out of the wetland. The data were used to calculate the wetland's total inflow and outflow runoff volumes for each measured storm event during the study period.

4.2 Water Quality Control Structure

The proposed WQCS was added to the SWMM model as an overflow weir and orifice, Figure 4.4. The WQCS was added upstream of the outlet culverts in Cell 3. The length of the overflow weir was restricted by the shape and topography of the cell's embankment at the outlet culverts and by the desire to provide a flow channel throughout Cell 3 that would not be obstructed by the proposed WQCS. GIS files and physical observations of the wetland were used to determine the maximum length of the weir, 44ft, while acknowledging the constraints, Figure 4.5 (Fort Collins 2012). In order to prevent an increase in the wetland flooding due to the installation of the WQCS, the maximum flow through the existing outlet pipes, 240 cfs, was used as the required flow through the weir. A minimum freeboard of 6 inches below the lowest embankment elevation, the overflow to the Poudre River, was used as a flooding safety factor. The maximum weir height was 2.5ft. The SWMM transverse weir flow equation, the maximum weir length and the maximum weir flow were used to calculate the weir height, Equation 4.1 (US EPA 2010). At a length of 44ft and a maximum flow of 240 cfs, the head above the weir bottom was calculated to be 1.4 feet. Therefore, the weir height was 1.6 feet. The designed WQCS weir overflow decreases the existing capture volume from 134,600 ft^3 to 39,200 ft^3. Using UDFCD's capture volume equation, the WQCV for the HSB would be 250,300 ft^3. The existing conditions and the proposed WQCS capture volumes are 53.8% and 15.7% of the required WQCV, respectively.

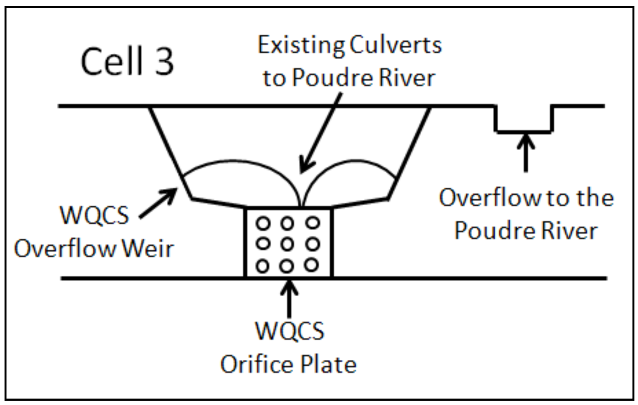

Figure 4.4. Sketch of the proposed WQCS design, not to scale.

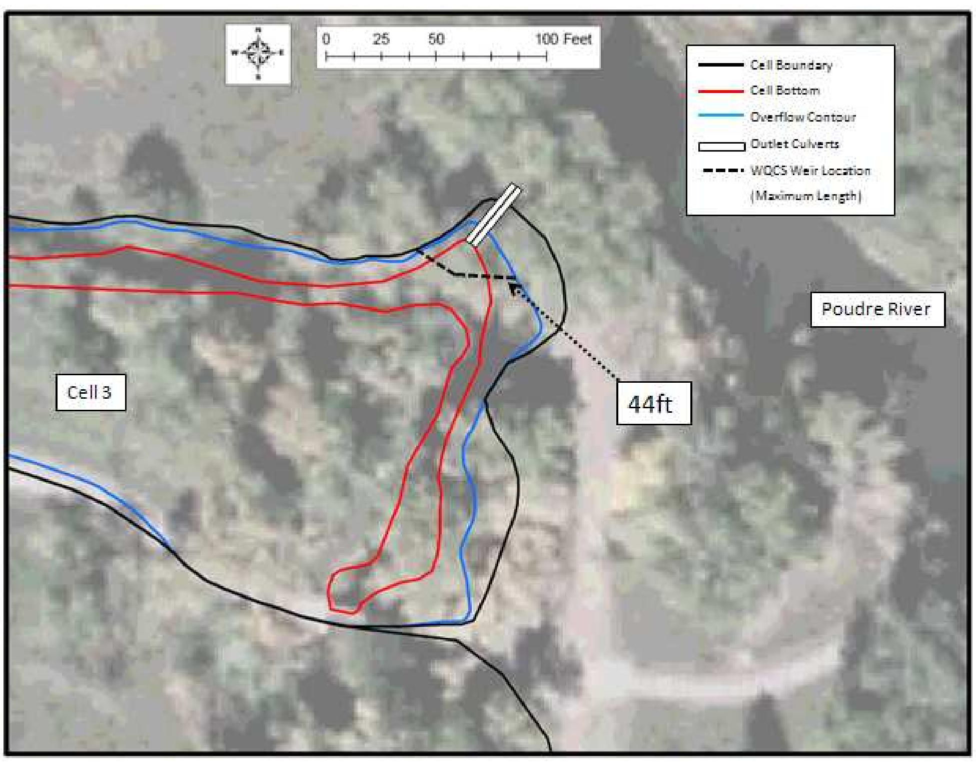

Figure 4.5. GIS map and contours used to determine maximum length of the WQCS weir.

$$L = \frac{3.33 * h^{3/2}}{Q}$$ **Equation 4.1**

Where: Q= flow through the weir, cfs

L= horizontal length of the weir, ft

h= head above the weir bottom, ft

Drawdown times ranging from existing conditions (2 hours) to 72 hours were evaluated for the design of the WQCS orifice. A maximum drawdown time of 72 hours was used because it is the minimum inoculation time for mosquito larvae (Deatrich and Brown 2004) and is the maximum time that runoff can be held in storage according to Colorado water law. An orifice diameter was established for each of the varying drawdown times. This was done by modeling the wetland with a single circular outlet orifice and adjusting the orifice diameter until the wetland drain time from brimful conditions to empty was equal to the desired drawdown time, Table 4.8. The brimful depth was equal to the depth below the weir, 1.6ft. The wetland was considered empty when the water depth reached 0.01 ft because the basin rarely emptied to a depth of 0.0 ft. This depth was chosen because it was the point where the basin depth to volume curve became asymptotic before reaching 0. A model was created for each drawdown time WQCS to provide output data for the analysis calculations of the HRT, efficiency ratio, and mass of TSS removed.

Table 4.8. Drawdown time with corresponding WQCS orifice shapes, diameters, and areas.

Drawdown Time (hr)	WQCS Orifice Shape	WQCS Orifice Diameter (ft)	WQCS Orifice Area (ft^2)
72	Circle	0.27	0.057
66	Circle	0.285	0.064
60	Circle	0.305	0.073
54	Circle	0.325	0.083
48	Circle	0.35	0.096
40	Circle	0.385	0.116
36	Circle	0.43	0.145
30	Circle	0.49	0.189
24	Circle	0.58	0.264
20	Circle	0.66	0.342
12.6	Circle	1	0.785
8.0	Circle	1.5	1.767
5.3	Rectangle (HXW)	1.55X2.03	3.147
3.3	Rectangle (HXW)	1.55X4.56	7.068
Existing Culverts	Circle and Ellipse	3.50 and 2.75	13.621

4.3 HRT Calculations

There is currently no standard equation available to calculate a single HRT for a stormwater wetland over a period of record. Furthermore, the steady state HRT equation does not account for the stochastic nature of stormwater runoff for a single event. Therefore, two methods were developed that utilize the HRT definition to calculate the storm event mean HRT, Equation 4.2 and Equation 4.3.

$$Event\ Average\ \ HRT_1 = \frac{\sum_{t=0}^{n}\left(\frac{V_t}{Q_{ot}}\right)}{n} \qquad \textbf{Equation 4.2}$$

$$Event\ Average\ \ HRT_2 = \frac{\sum_{v=0}^{n}\left(\frac{\mathrm{V_v}}{\mathrm{Q_{ov}}}\right)}{n} \qquad \textbf{Equation 4.3}$$

Where: Q_o= flow rate out of the wetland at time t or volume *v*, cfs

V= volume of water in wetland at time t or volume *v*, ft^3

t= time step, min

v= volume step, ft^3

n=total number of volume or time steps during an event

The volume of water in the wetland is the sum of the volume of water in all three cells at a given time or volume interval. The wetland discharge flow rate is the sum of the flow rates from the WQCS and overflows to the Poudre River at a given time step or volume interval. All culvert and overflow outflows are included in the HRT calculations because it was assumed that all runoff that reaches the wetland receives treatment. The three cell layout and the WQCSs in Cell 1 and Cell 3 restrict the runoff in each cell long enough to provide a degree of settling, mixing, and flow attenuation. Flow that bypasses one cell is assumed to receive treatment in the downstream cell. If the wetland was a single basin, the flow discharging through the WQCS orifice would be the only water treated.

Both HRT methods acknowledge the dynamic nature of stormwater runoff as the wetland fills and empties, by calculating the HRT throughout a storm event. Equation 4.2, HRT_1, uses the definition of HRT and a constant time step of 1 minute to capture the peak(s) of a storm event. By using a smaller time step and calculating the HRT at each time step and then taking the average of those HRTs, the stochastic nature of stormwater runoff is accounted for. Equation 4.3, HRT_2, calculated the HRT for a constant volume interval to capture the peak(s) of a storm event. The volume interval is equal to 0.3% of the brimful volume, 117 ft^3. A constant volume interval allows the HRT to be calculated at relatively the same wetland volume for every storm event, no matter the intensity or duration of the storm event. For the existing conditions and the proposed wetland drawdown times, the average annual HRT was calculated for both methods by averaging the event mean HRTs. The average annual HRT values from the two methods were also compared to confirm that the equations followed the same trend.

The volume interval used in the HRT_2 method was the largest volume interval that allows every storm event that produces runoff and fills Cell 3 to a depth greater than 0.01ft to have at least one calculated HRT value. Cell 3 must fill to a depth of at least 0.01ft because that is the cutoff for when the Cell 3 was considered empty and a storm event is over. As mentioned in the WQCS section, the wetland was considered empty when the depth in Cell 3 was less than 0.01ft because, at that point, the cell's volume to depth curve becomes asymptotic. In order for a storm event to be considered over, Cell 3 must be at a depth of less than 0.01ft and there was no stormwater runoff inflow from a new storm event into Cell 1. If there was inflow from a new storm into Cell 1 when the depth in Cell 3 becomes less than 0.01ft, the storm event was not over and the two storm events are combined. Depending on the WQCS's drawdown time, the number of measurable storm events for 2009 range from 33 to 46 storms. The number of storm events decreases as the drawdown time increases because the wetland does not empty before the next storm event occurs. Therefore, the larger the drawdown time, the more combined storm events and the less overall number of storm events analyzed.

4.4 Wetland Performance

The wetlands performance for each of the proposed WQCS drawdown times was analyzed using removal efficiency and total annual TSS removed. As mentioned in Section 2.2 of the Literature Review, the event mean influent and effluent concentrations are required to calculate removal efficiency, Equation 4.4 . To calculate the total annual TSS removed, the EMC effluent concentration and the measured influent concentrations (Messamer 2011) were also required, along with the SWMM output runoff volumes, Equation 4.5. The pollutant TSS was analyzed in this study for its ability to follow the first order removal model used in the k-C* method and it is assumed that if TSS settle out, other pollutants will also settle out (Kadlec and Wallace 2009).

$$\text{Removal Efficiency} = 1 - \frac{\sum_{i=1}^{n}(\text{EMC}_{in})}{n} \div \frac{\sum_{i=1}^{n}(\text{EMC}_{\text{out}})}{n} \qquad \textbf{Equation 4.4}$$

$$\text{Total Annual TSS Removed} = \sum_{i=1}^{n}(\text{EMC}_{\text{in}} * \text{V}_{\text{i}}) - \sum_{i=1}^{n}(\text{EMC}_{\text{out}} * \text{V}_{\text{i}}) \qquad \textbf{Equation 4.5}$$

Where: EMC_{in}= influent EMC of the storm event

EMC_{out}= effluent EMC of the storm event

V_i= total runoff volume of the storm event

i= storm event number

n= total number of storm events

Messamer (2011) used sample data from the 2009-2011 HSB study to calculate pollutant influent EMCs and the irreducible pollutant concentrations for the HSB. The measured TSS influent EMC was 216 mg/L. While Messamer (2011) used a C* value of 20 mg/L for the HSB's relative efficiency analysis, as suggested by Schueler (1996), the measured minimum TSS effluent EMC was 14mg/L and used in this analysis. As mentioned in the literature review, the k and C* constants are reliant on several variables within the wetland and watershed. The k constant is not dependent on inlet pollutant concentrations, but is a function of wetland characteristics and operating conditions (Kadlec 1997). Using the measured data and Equation 4.6, the k value was calculated for the wetland, Table 4.9. For the existing conditions and the proposed drawdown times, the k-C* method and measured data were used to calculate the TSS effluent EMC concentration, which were needed for the removal efficiency and total annual TSS removed calculations, Equation 4.6.

Table 4.9. The HSB measured TSS influent EMC, the irreducible constituent concentration, and calculated k values (taken from Messamer 2011)

Constituent	C_i mg/L (lb/ft^3)	C* mg/L (lb/ft^3)	k m/yr (ft/yr)
Total Suspended Solids (TSS)	261 (0.0163)	14 (0.00087)	2670 (8760)

$$C_o = e^{\left(\frac{-k}{HLR}\right)} * (C_i - C^*) + C^* \qquad \textbf{Equation 4.6}$$

Where: C_o= effluent concentration of the system, mg/L

C_i= influent event mean concentration, mg/L

C*= irreducible background concentration, mg/L

k= areal rate constant, m/yr

HLR= hydraulic loading rate, m/yr

The k-C* method requires a single HLR value. In order to account for the varying intensity and duration of a storm event, an equation was developed to calculate an event mean HLR, Equation 4.7. The HLR equation uses the same concept as the HRT_2 equation by averaging the calculated HLRs at each volume interval for a storm event. The storm event HLRs were averaged to calculate an annual average HLR. The volume interval was the same as the HRT volume interval, $117 ft^3$ or 0.3% of the brimful volume. The wetland volume, inflow rate, and surface area data for the HLR calculations were acquired from the SWMM output files.

$$Event\ Mean\ HLR = \frac{\sum_{i=1}^{n}\left(\frac{Q_v}{A_V}\right)}{n} \qquad \textbf{Equation 4.7}$$

Where: Q= flow rate into the wetland at volume *v*, cfs

A= surface area of wetland at volume *v*, ft2

i= storm event number

n= total number of storm events analyzed

5.0 RESULTS AND DISCUSSION

SWMM model output results were used to calculate and analyze the removal efficiency, the total annual TSS removed, the HRT and the effects of the WQCS on the wetland and surrounding area. First, the SWMM volume output data were analyzed for calibration of the model. The calibration process included comparing the measured volume data from Messamer's study (2011) to the SWMM output volume data using the percent error equation. The data used in the comparison was from six storm events with depth measurements between 0.1 to 0.7 inches observed during the 2009-2011 HSB study, Table 5.1 (Messamer 2011). These parameters were chosen to ensure runoff would occur during the storm event, but would not flood the wetland. When available, the measured wetland outflow data were used to estimate the watershed's total runoff volume because all flow into the wetland was assumed to be accounted for.

Table 5.1. Measured and SWMM total runoff volume from the HSB for six storm events occurring in 2010 and 2011.

Storm Event	Preciptitation (in)	Measured Total Volume (cf)	SWMM Total Volume (cf)	Percent Error
8/8/2010[A]	0.58	175906	204861	16%
10/22/2010[A,B]	0.37	70255	134072	91%
4/24/2011[B]	0.49	116955	190198	63%
6/16/2011	0.37	160508	121720	24%
6/30/2011	0.41	133446	163467	22%
7/6/2011	0.54	185969	117992	37%

A Outflow data used
B Storm encompassed entire watershed

The percent errors between the SWMM model and the measured data were acceptable for the majority of events, with the exception of two storm events. The largest percent error between the model and measured data was from an October 22, 2010 storm event, which occurred over the entire watershed. A storm event on June 16, 2011 had equal precipitation as the October 22, 2010 event, but had more than double the amount of measured runoff volume and was only over part of the watershed. The inconsistency in the October 22nd storm event presented a degree of uncertainty in the measured data. The runoff volume from October 22nd storm event should be larger because it has a larger area receiving an equal depth of precipitation as the June 16th event. The same irregularity was seen with the April 24, 2011 storm event. Rainfall data from gauges throughout the City showed that precipitation occurred over the entire watershed for the April 24th event. However, there was less measured total runoff volume for the April 24th event than was measured for other events over smaller areas with lower precipitation depths. The percent errors for the rest of the storm events were considered acceptable. Therefore, the SWMM sub-basin parameter values were considered adequate to represent the watershed and the runoff volume entering the wetland. The initial model input parameter values were not adjusted and the model was not calibrated using the measured data.

After the SWMM model input parameters were justified, the removal efficiency and total annual TSS removed were analyzed for the existing and proposed drawdown times to determine which drawdown time would maximize efficiency, while also considering physical and environmental constraints. The results are shown both graphically and in tabular form, Figure 5.1 and Table 5.2.

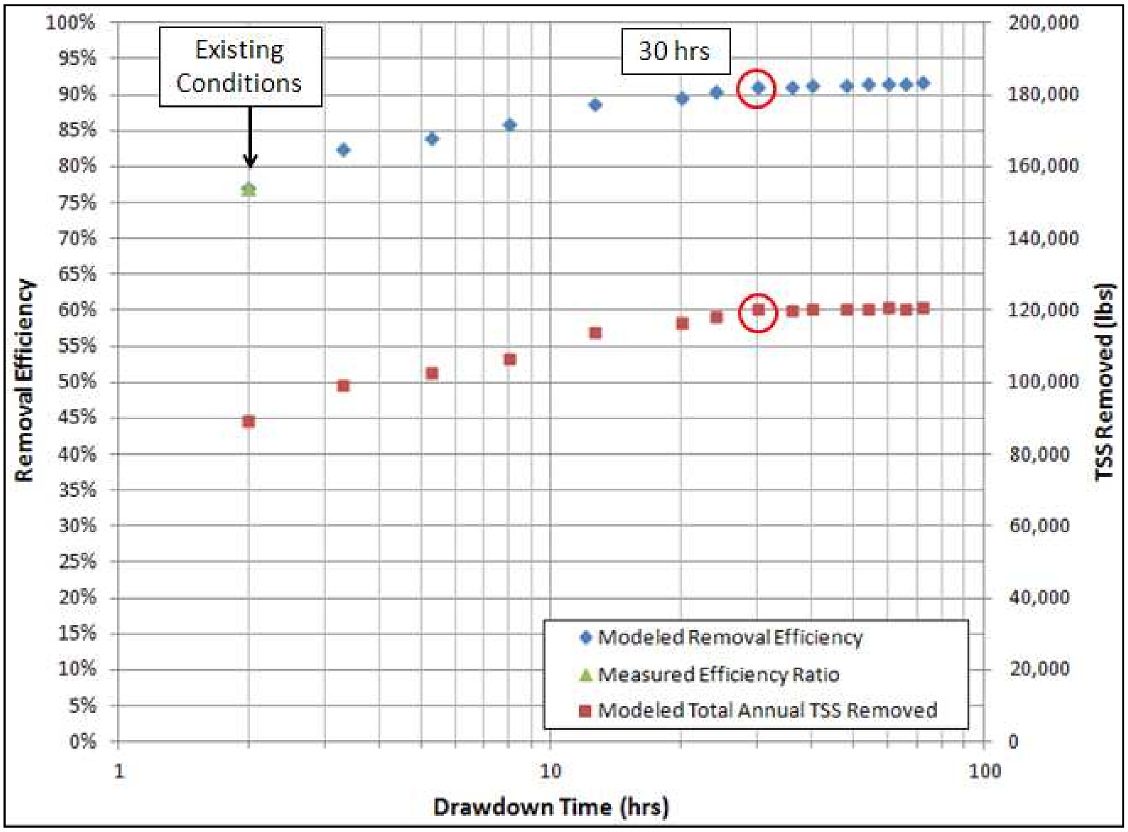

Figure 5.1. Calculated removal efficiency and total annual TSS removed values for the proposed drawdown times and the measured efficiency for existing conditions provided by Messamer (2011).

Table 5.2. The proposed drawdown times, corresponding log drawdown times, and the calculated HRT, removal efficiency, and total annual TSS removed

Drawdown Time (hr)	Orifice Area (ft2)	Efficiency Ratio	Total Annual TSS Removed (lbs)	Avg Annual HRT	
				HRT_1: equal ΔT (hr)	HRT_2: equal ΔV (hr)
72	0.057	91.7%	120,800	22.7	23.2
66	0.064	91.6%	120,600	21.7	22.0
60	0.073	91.5%	120,800	20.4	20.3
54	0.083	91.5%	120,600	19.4	19.1
48	0.096	91.3%	120,600	18.4	17.8
40	0.116	91.3%	120,500	17.3	16.3
36	0.145	91.2%	120,100	16.2	14.6
30	0.189	91.2%	120,200	15.1	13.1
24	0.264	90.4%	118,300	13.8	11.0
20	0.342	89.6%	116,500	13.2	10.3
12.6	0.785	88.7%	114,000	11.7	8.5
8.0	1.767	86.0%	106,600	10.7	7.8
5.3	3.147	84.0%	102,500	9.6	6.8
3.3	7.068	82.5%	99,100	8.8	6.2
Exisitng	13.621	77.0%	89,100	6.4	4.5

The total annual TSS removed and the removal efficiency follow the same trend. Both values increase as the drawdown time increases. However, the irreducible pollutant concentration in the k-C* model kept the curves from increasing linearly. The removal efficiency and the total annual TSS removed curves level off at around 91% and 120,000 lbs, respectively. From existing conditions to the maximum drawdown time of 72 hrs, there was a 14.7% increase in removal efficiency. The removal efficiency only increases by 0.5% from a drawdown time of 30 hrs to 72 hrs. The total annual TSS removed from the existing drawdown time to a drawdown time of 30 hrs was approximately 31,100 lbs, while there was only an increase of 600 lbs of TSS removed from 30 hrs to 72 hrs.

The WQCS orifice area at a 30hr drawdown time was 27.2 in^2. A smaller drawdown time of 24 hrs or 20 hrs would increase the orifice area to 38.0 in^2 and 49.2 in^2, respectively. The WQCS in Cell 1 has an orifice area of 216 in^2 and still clogs, Figure 3.4. Therefore, using a drawdown time of 20 hrs instead of 30 hrs will increase the orifice area by 22 in^2, but it will decrease the efficiency and the total annual TSS removed by 1.6% and 3,700 lbs, and will not stop the WQCS from clogging. Furthermore, it would not be beneficial to increase the drawdown time past 30 hrs because there will be minimal improvement in the removal efficiency, 0.5%, and it will also decrease in the orifice area.

The developed HRT calculation methods were analyzed to determine if the results of the two methods were consistent and followed the same trend. The HRT was calculated using a constant time interval in the HRT_1 method and constant volume interval in the HRT_2 method, Figure 5.2 and Table 5.2. Generally, the two HRT methods followed the same trend; the HRT increased as the drawdown time increased. However, the HRT_1 method increased at a faster rate than HRT_2 method. The HRT_1 method also estimated larger HRTs at smaller drawdown times than the HRT_2 method. The average difference was approximately 2.5 hrs. The difference between the two HRT methods could be attributed to the HRT values calculated by the HRT_1 method at the end of the storm event. The outflow flow rates at the end of a storm event were the lowest outflows, which caused the calculated HRTs to be relatively high. There were several HRTs calculated for the HRT_1 method at the end of the storm event, which increased the event average HRT. The HRT_2 method only calculated a few HRT values at the end of a storm event, which did not affect the HRT_2 method event average HRT. At a 30 hour drawdown time for HRT1 and HRT2 are 15.1 hrs and 13.1 hrs, respectively.

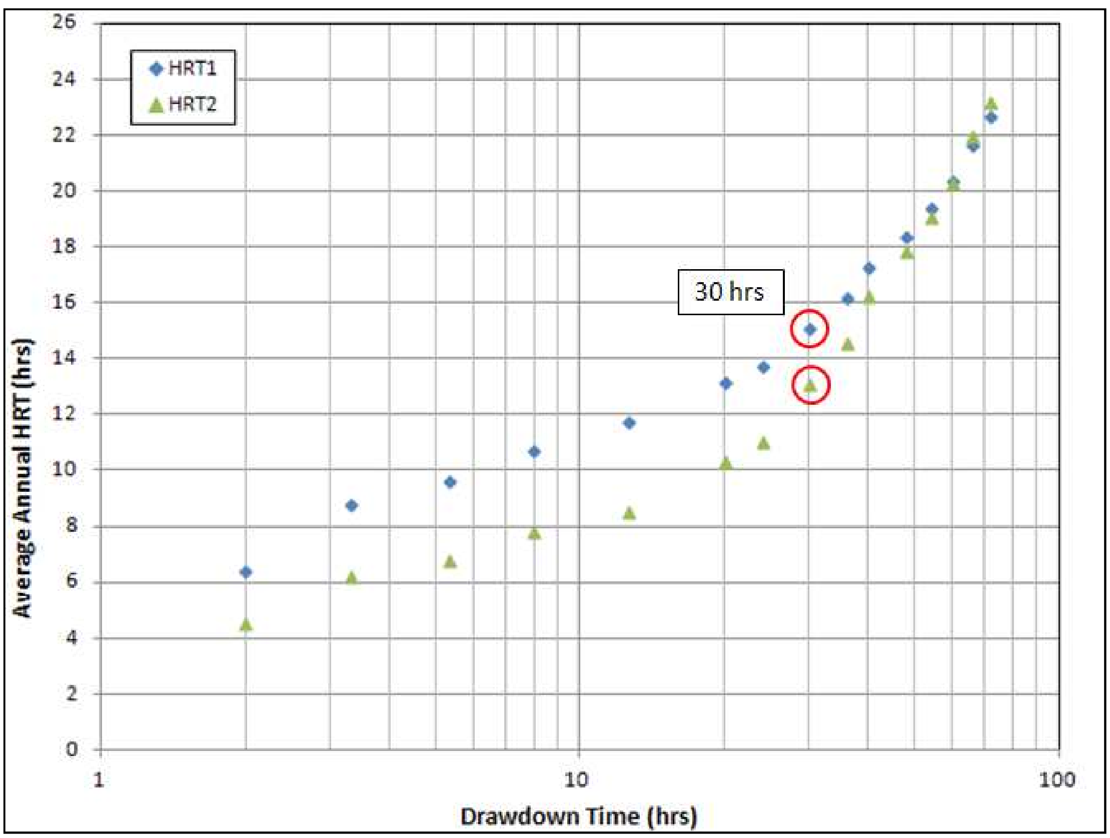

Figure 5.2. Wetland HRT curves for the proposed drawdown time. HRT_1 uses a constant time interval and HRT_2 used a constant volume interval.

One of the major constraints in this project was the possibility of extended flooding and ponding water associated with extended drawdown times. A primary goal for this project was to design a WQCS, that when installed, would not adversely affect the bike path and adjacent park. The WQCS weir height of 1.6 ft will cause extended periods of ponding water in Cells 2 and 3. The detained water will have a brimful surface elevation of 4959.4 ft, which should only minimually disturb the adjacent park property that is a part of the wetland while the wetland drains, Figure 5.3. At brimful conditions, the detained water in Cell 3 remains within the confines of the permanent pool area. However, Cell 2 would experience some flooding into the adjacent park area near its the outlet culvert during brimful conditions. The impacted park area is primarily wooded and the detain water should not adversly affect the public use of the park.

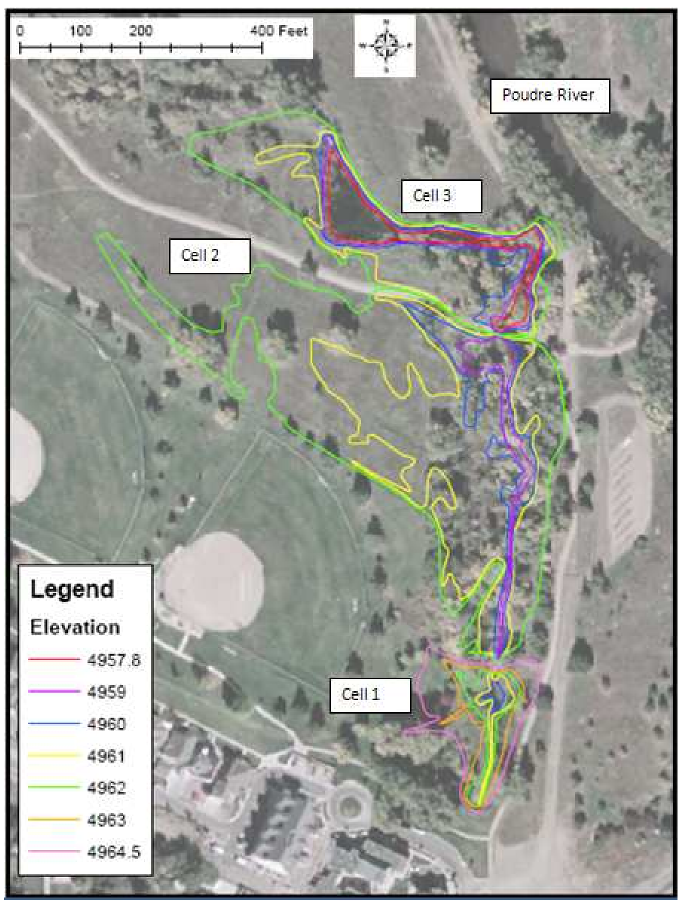

Figure 5.3. Wetland cell contours

City code requires new projects to assess the 2yr and 100yr storm events for potential flooding and drainage issues at the site (Fort Collins 2011). Therefore, the proposed drawdown times were analyzed for the 2yr and 100yr storm events, Table 5.3 and Table 5.4. If the park or bike path is under water for an extended period of time, there could be issues with public use and safety. The intent of the design of the proposed WQCS overflow weir was to not increase flooding for the 2yr and 100yr events. This was done by designing the overflow weir to accommodate the maximum flow through the existing outlets.

Table 5.3. 2yr and 100yr storm event flooding durations at overflow locations for each drawdown time modeled

Drawdown Time (hr)	Flooding Duration at Overflows (hrs)									
	2yr					100yr				
	HSB Outlet Culvert	Cell 1 to Cell 2	Cell 2 to Cell 3	Cell 2 to the Poudre River	Cell 3 to the Poudre River	HSB Outlet Culvert	Cell 1 to Cell 2	Cell 2 to Cell 3	Cell 2 to the Poudre River	Cell 3 to the Poudre River
72	0	1.2	0.2	0	0	0	2.4	1.9	1.7	1.4
66	0	1.2	0.2	0	0	0	2.4	1.9	1.7	1.4
60	0	1.2	0.2	0	0	0	2.4	1.9	1.7	1.4
54	0	1.2	0.2	0	0	0	2.4	1.9	1.7	1.4
48	0	1.2	0.2	0	0	0	2.4	1.9	1.7	1.4
40	0	1.2	0.2	0	0	0	2.4	1.9	1.7	1.4
36	0	1.2	0.2	0	0	0	2.4	1.9	1.7	1.4
30	0	1.2	0.2	0	0	0	2.4	1.9	1.7	1.4
24	0	1.2	0.2	0	0	0	2.4	1.9	1.7	1.4
20	0	1.2	0.2	0	0	0	2.4	1.9	1.7	1.4
12.6	0	1.2	0.1	0	0	0	2.4	1.9	1.7	1.4
8.0	0	1.2	0.1	0	0	0	2.4	1.9	1.7	1.4
5.3	0	1.2	0	0	0	0	2.4	1.9	1.7	1.4
3.3	0	1.2	0	0	0	0	2.4	1.9	1.7	1.4
Existing Culverts	0	1.1	0	0	0	0	2.4	1.8	1.6	1.2

Table 5.4. 2yr and 100yr storm event flooding depth in feet at overflow locations for each drawdown time modeled.

Drawdown Time (hr)	Maximum Flooding Depth at Overflows (ft)									
	2yr					100yr				
	HSB Outlet Culvert	Cell 1 to Cell 2	Cell 2 to Cell 3	Cell 2 to the Poudre River	Cell 3 to the Poudre River	HSB Outlet Culvert	Cell 1 to Cell 2	Cell 2 to Cell 3	Cell 2 to the Poudre River	Cell 3 to the Poudre River
72	0	0.46	0.01	0	0	0	1.45	0.84	0.74	1.10
66	0	0.46	0.01	0	0	0	1.45	0.84	0.74	1.10
60	0	0.46	0.01	0	0	0	1.45	0.84	0.74	1.10
54	0	0.46	0.01	0	0	0	1.45	0.84	0.74	1.10
48	0	0.46	0.01	0	0	0	1.46	0.84	0.74	1.10
40	0	0.46	0.01	0	0	0	1.46	0.84	0.74	1.10
36	0	0.46	0.01	0	0	0	1.45	0.84	0.74	1.10
30	0	0.46	0.01	0	0	0	1.45	0.84	0.74	1.10
24	0	0.46	0.01	0	0	0	1.45	0.84	0.74	1.10
20	0	0.46	0.01	0	0	0	1.45	0.84	0.74	1.10
12.6	0	0.46	0.01	0	0	0	1.45	0.84	0.74	1.10
8.0	0	0.46	0.01	0	0	0	1.46	0.84	0.74	1.10
5.3	0	0.46	0	0	0	0	1.46	0.84	0.74	1.10
3.3	0	0.46	0	0	0	0	1.46	0.84	0.74	1.10
Existing Culverts	0	0.46	0	0	0	0	1.45	0.82	0.72	1.08

The wetland floods at the overflow locations during both the 2yr and 100yr events for the existing conditions and the proposed WQCS. For existing conditions, the 2yr event floods the embankment at the outlet structure between Cell 1 and Cell 2 for 1.2 hrs with a maximum depth of 0.46 ft, at an elevation of 4963.46 ft. For the WQCS with a proposed drawdown time of 30hrs, the flooding duration and maximum depth from Cell 1 to Cell 2 does not change. However, flooding does occur at the overflow between Cell 2 and Cell 3 for a duration of 0.2 hrs at a maximum of 0.01 ft. The flooding from the 2yr event should not affect the surrounding area outside of the wetland boundary.

The 100yr event causes flooding to occur at all four overflow locations for the existing conditions and the proposed WQCS. The overflow between Cell 1 and Cell 2, for both the existing conditions and the WQCS, floods for a duration of 2.4 hrs at a maximum depth of 1.45 ft. This will cause the depth of water in Cell 1 to be at an elevation of 4964.45 ft, which is very close to the cells maximum embankment elevation of 4964.5 ft. For existing conditions, the 100yr storm floods the overflow between Cell 2 and Cell 3 for a duration of 1.8 hrs with a maximum depth of 0.82 ft, at an elevation of 4961.42ft. The proposed WQCS with a 30 hr drawdown time only increases the flooding duration by 0.1 hrs and the maximum depth by 0.02 ft. Approximately 300ft of the sidewalk at the culvert between Cell 2 and Cell 3 will flood. In both Cell 2 and Cell 3 the overflows to the Poudre River flood causing two sections of the sidewalk along the west embankment to be flooded. The depth of water from 100 yr storm event in both Cell 2 and Cell 3 does not cause the wetland to overflow at any other location besides the overflow locations.

The addition of the WQCS will also affect the wetlands outflow hydrograph. While the WQCS is not meant to be a quantity control structure, the peak discharge flow is decreased and the flow rate at the end of the event is changed, Figure 5.4. The peak outflow was not greatly affected because the overflow weir structure of the WQCS was designed to discharge the maximum flow rate of the existing outlet culverts. The WQCS extends the discharge time and rate at the end of the event compared to the existing conditions.

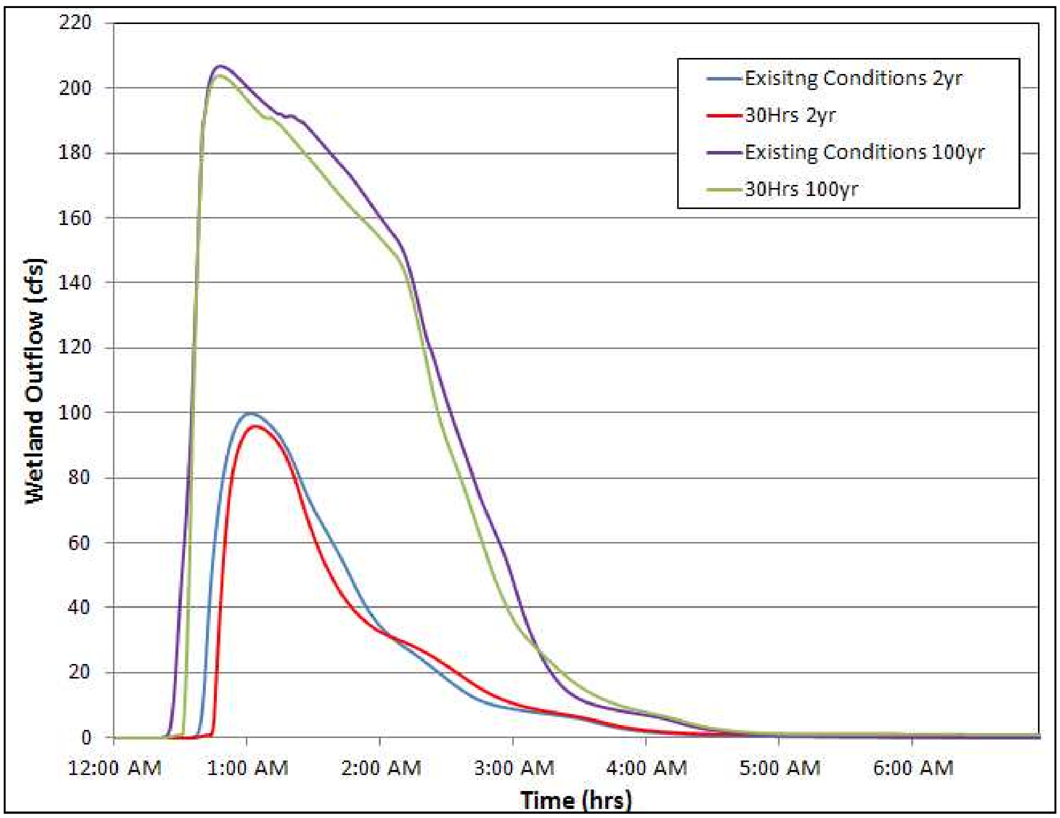

Figure 5.4. Outflow hydrograph for the 2yr and 100yr storm events for existing conditions and the proposed 30hr drawdown time WQCS.

6.0 CONCLUSIONS AND RECOMMENDATIONS

The objectives of this project was to design a WQCS to maximize pollutant removal without causing additional flooding at the site and to quantify the increase in the mass of total annual total suspended solids removed from existing conditions. The project results also proved that installing a WQCS will increase the HRT and the removal efficiency of the wetland, Figure 5.1 and Figure 5.2. While the largest drawdown time produces the largest HRT and removal efficiency, wetland cell flooding and clogging of the WQCS orifice should also be incorporated in the WQCS design.

We propose a WQCS design with a 30 hr drawdown time and an HRT of approximately 14 hrs. The design includes an overflow weir with a length of 44 ft and height of 1.6 ft and an orifice with an area of 27.2 in^2. The proposed WQCS design decreases the existing capture volume of the wetland from 134,600 ft^3 to 39,200 ft^3. If the wetland dimensions were adjusted to capture the designed WQCV, the wetland volume would need to be increased to a volume of 250,300 ft^3. Therefore, the proposed capture volume of the WQCS is 15.7% of the required WQCV.

The proposed WQCS will remove approximately 120,200 lbs of TSS on an annual basis, increasing the mass removed from existing conditions by 30,100 lbs. Furthermore, the removal efficiency will increase by 14.2%. For all the modeled WQCS designs, the wetland flooding maximum depths and durations at the overflow weirs increased minimally from existing conditions for both the 2yr and 100yr storm events. The flooding increases were not enough to overtop the wetland embankment at locations other than at the existing overflows or affect public safety and use. Furthermore, the detained runoff by the WQCS did not extend into the open space areas of the adjacent park. The clogging of the WQCS orifice was a primary constraint after maximizing the removal efficiency. With appropriate design, the proposed orifice diameter should increase the probability of clogging.

We recommend completing a field study of the wetland after the WQCS installation to confirm the results of the model. The field study should include sampling of the stormwater runoff at the inlet, the outlet culverts, and the overflow weirs to verify the assumption that all stormwater runoff entering the wetland is treated. Tracer studies should also be performed to calculate the wetland's average HRT for multiple storm events. The results can then be used to determine if the two HRT methods developed are valid equations to calculate the HRT in a stormwater wetland.

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