PERFORMANCE OF A RETROFIT DETENTION BASIN IN FARGO, ND

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Performance of a Retrofit Detention Basin in Fargo, ND

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ABSTRACT

“The Fargo Project” located in Fargo, North Dakota, is an 18-acre stormwater detention basin that was retrofitted in 2015 to include an earthen-channel, sediment forebay, and various native vegetation within the floodplain and channels. Goals of this study were to assess how the post-retrofit earthen-channel performs relative to the pre-retrofit concrete-channel in terms of conveyance of small storms, and to estimate infiltration and evaporation from the post-retrofit detention basin during various storm sizes and intensities. Results showed that although channel roughness ultimately increased in the post-retrofit basin and allowed for greater instances of flooding for one channel, erosion of the main channel, with a larger urbanized contributing area, resulted in behavior similar to that of the pre-retrofit main channel for small storms. Modeled infiltration and evaporation showed total abstraction ranging between 2.9% and 11.7% of the maximum ponded volume for various storm sizes and intensities.
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LIST OF ABBREVIATIONS

SCM……………………………………………………………………Stormwater control measurement
WSUD………………………………………………………………Water sensitive urban design
BMP………………………………………………………………………Best management practices
SUDS………………………………………………………………Sustainable urban drainage system
GI…………………………………………………………………………Green infrastructure
LID…………………………………………………………………………Low impact design
CWA……………………………………………………………………Clean water Act
EPA………………………………………………………………………Environmental Protection Agency
NPDES………………………………………………National Pollution Discharge Elimination System
WQA……………………………………………………………………Water Quality Act
MS4……………………………………………………………………Municipal separate stormwater systems
MEP………………………………………………………………………Maximum extent practicable
BAT………………………………………………………………………Best available technology
BCT………………………………………………………………………Best conventional pollutant control technology
TMDL…………………………………………………………………..Total maximum daily load
SWPPP………………………………………………………………Stormwater pollution prevention plan
TSS……………………………………………………………………Total suspended solids
FAO……………………………………………………………………Food and Agricultural Organization
WSE……………………………………………………………………Water surface elevation
HEC-RAS…………………………………..Hydraulic Engineering Center-River Analyses System
DEM……………………………………………………………………Digital elevation model
NDAWN……………………………………………………North Dakota Agricultural Weather Network
NOAA……………………………….……….National Oceanic and Atmospheric Administration

DSM………………………………………………………..Digital surface model

NJDEP……………………………………………………..New Jersey Department of Environmental Protection

USDA……………………………………………………..United States Department of Agriculture

SWMM……………………………………………………..Stormwater Management Model
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1. INTRODUCTION

Within an urban context, stormwater is considered a nuisance due to the possibility of flooding and subsequent damage to roads, buildings, and other infrastructure. Therefore, engineering design for stormwater management traditionally consists of removing water from a site as quickly as possible using concrete pipes and channels (Williams et al., 2006). The increase in urban sprawl alongside the aforementioned method of stormwater management significantly alters the hydrologic regimen and in turn increases incidences of flooding, and alters water temperature and chemistry, ultimately degrading ecosystems of natural water bodies (Roy et al., 2008). Literature regarding stormwater management consists of variable nomenclature such as water sensitive urban design (WSUD), best management practices (BMP), stormwater control measures (SCM), sustainable urban drainage systems (SUDS), green infrastructure (GI), and other labels that generally describe the broad range of stormwater management techniques available (Fletcher et al., 2015). In this study, stormwater control measures (SCM’s) will be used to describe the various techniques used to manage stormwater through reduction or attenuation of water quantity and quality.

It is imperative that more holistic type SCM’s be utilized in an attempt to restore natural hydrology of a watershed and reduce pollutant loads to a waterbody (National Research Council, 2009). While traditional detention/retention basins are regularly used as a means to reduce peak flow and flooding in many urban areas, more holistic approaches to stormwater management have emerged to include infiltration, rainwater harvesting, green roofs, bioretention, and low impact design (LID) type SCM’s that are either in a distributed or centralized manner within a watershed. Although distributed SCM’s have seen success throughout the literature in reducing total runoff volume and increasing water quality (Loperfido et al., 2014; Li et al., 2017), these
techniques are often expensive or difficult to incorporate into an already established urban area. Therefore, retrofitting of in-place detention/retention offers a low-cost alternative in comparison to other SCM’s while potentially increasing infiltration, evapotranspiration, and water quality of urban watersheds (Haberland et al., 2012).

The overall goal of this study was to develop an understanding of how stormwater management has changed through the years, and the difficulties involved in regulating the large-scale processes that are involved in stormwater runoff as a point source water quality issue. While there has been a push towards holistic management techniques, previously built stormwater management structures still widely exist within densely populated urban areas and offer an opportunity for retrofitting. Therefore, current techniques used to retrofit previously established detention basins, that were solely designed for attenuation of stormwater, and the resulting effects on both water quantity and quality were analyzed. This was done through a literature review, albeit brief due to a relatively small pool of retrofit studies compared to research on other structural stormwater techniques. Additionally, in-situ research involving a retrofit detention basin located in Fargo, North Dakota was also completed by utilizing low-cost monitoring techniques similar to that of Toran, 2016, to determine performance of a post-retrofit detention basin with regards to channel flooding. More advanced modeling techniques were also used to compare pre- and post-retrofit basin channel performance, as well as to estimate post-retrofit water volume, infiltration, and evaporation across various size storms between May 1, 2018 and September 31, 2018.

1.1. History of Stormwater Management

The earliest recorded evidence of SCM’s for flood control and water resources date back to the Mesopotamian empire and include engineered structures such as cisterns to hold and store
rainfall and runoff (National Research Council, 2009). Within recent history and modern times, stormwater is viewed as a potential hazard to people and infrastructure due to an increase in runoff from impervious urban areas and subsequent increased potential for flooding. With the need to protect human health and property, 19th and 20th century stormwater management techniques using pipe systems were created in order to quickly route runoff to the nearest natural water body (Subramanian, 2017). This in turn would cause flooding and bank erosion downstream that was soon fixed by enlarging and armoring stream channels, increasing flooding and erosion even further downstream (National Research Council, 2009). Regional and on-site detention basins with concrete lined channels were subsequently used in order to reduce peak flows but offered no solution to reducing total volume and alternatively, allowed runoff from small storms directly through the basin un-attenuated (National Research Council, 2009). These techniques degraded ecosystems of streams, rivers, and lakes through alteration of natural hydrologic regimes and release of various pollutants (Subramanian, 2017). In order to combat degradation of natural water bodies from these stormwater management systems, regulations addressing water pollution began to appear and influenced SCM’s used within cities. Since these regulations focused on a water quality approach by attempting to reduce pollutant discharge, most stormwater management systems still utilize on-site and regional detention to control stormwater quantity despite having better practices available.

1.2. Regulation of Stormwater

The Clean Water Act (CWA) of 1972, reorganized on the basis of the 1948 Water Pollution Control Act, gave the US Environmental Protection Agency (EPA) control to implement programs with the goal of reducing point-source pollution discharge to surface waters (Federal Water Pollution Control Act, 2002). Although the engineered conveyance and outfall of
stormwater falls under the definition of point-source pollution discharge, the CWA ultimately limited pollution discharge of effluent based on best available pollution technology to industrial and municipal dischargers that obtained a National Pollution Discharge Elimination System (NPDES) permit (Franzetti, n.d.). The EPA’s exemption of stormwater from NPDES permitting was due to the difficulty in controlling the unpredictable behavior of stormwater through end-of-pipe controls, and the large number of permits required to be issued (National Research Council, 2009). Despite the fact that stormwater was not initially included in regulation, states such as Florida, Washington, Maryland, and Oregon implemented programs to deal with water quality and quantity (National Research Council, 2009). With ongoing regulation of point source discharges from sewage treatment and other industrial sites, it became apparent that stormwater from urban areas played a larger role in stream degradation and consequently amendments were made to the CWA.

With the passing of The Water Quality Act (WQA) in 1987, revisions to the CWA were made to account for discharge of stormwater from industrial and municipal sources in a two-phase approach (United States Environmental Protection Agency, 2004). This kept permitting authority with the EPA but also gave the agency the ability to delegate permitting authority to states with programs equal or exceeding the federal program standards (Subramanian, 2017). Phase I Stormwater regulations were put into effect in 1990 to require permits for large municipal separate stormwater systems (MS4’s) for cities with populations greater than 100,000, industrial activities, construction activities five acres or more, or any other large contributors to water quality through stormwater discharge (United States Environmental Protection Agency, 2004; National Research Council, 2009). These municipal and industrial/construction permits required holders utilize various control or engineering techniques to reduce pollution discharge
to the maximum extent practicable (MEP), and to implement best available technology (BAT) and best conventional pollutant control technology (BCT) to limit pollution (United States Environmental Protection Agency, 2004). In 1995, Phase II Stormwater regulations were put into place and included small MS4’s in communities with a population less than 100,000 and construction sites between one and five acres. EPA standards regarding permits of municipal, industrial, and construction activities vary in how pollutant discharge is controlled and how monitoring is conducted (National Research Council, 2009). Additionally, since states were granted NPDES permit authority, variations in aforementioned standards are seen between states, especially where total maximum daily loads (TMDL) of impaired water bodies have been calculated.

Municipal, industrial, and construction NPDES permit holders must develop a stormwater plan in order to reduce pollutant discharge to water bodies, though requirements for respective plans differ. Holders of MS4 NPDES permits are required to develop a stormwater management plan including best management practices (BMP’s) that cover six minimum measures of public education/outreach, public participation/involvement, illicit discharge detection/elimination, construction site runoff control, post-construction site runoff control, and pollution prevention/good housekeeping (United States Environmental Protection Agency, 2005a). The goal of this stormwater management plan is to reduce pollutant runoff and discharge to the maximum extent practicable through measurable goals produced by selected structural and non-structural BMP’s (United States Environmental Protection Agency, 2005a). These BMP’s are selected from an EPA issued menu pertaining to all six minimum measures included in the stormwater management plan, while the operator creates measurable goals to be met. Initially, MS4’s under Phase 1 were required to conduct water quality monitoring at a sample of outfalls,
which in turn could be used to create effluent limitations, as well as necessary inspection of industrial and construction activities within the municipal area, but under Phase II permits this is no longer required (National Research Council, 2009).

Holders of industrial and construction NPDES permits must develop a stormwater pollution prevention plan (SWPPP) including BMP’s that will be used to reduce erosion, sediment, and pollution associated with stormwater discharge. BMP’s must be able to meet BAT and BCT standards, or if technology-based controls are ineffective for impaired waters, a TMDL must be calculated and not exceeded (United States Environmental Protection Agency, 2005b). Industrial and construction permits have certain monitoring criteria that must be met in order to determine the effect of the developed SWPPP on stormwater. For industrial permits, a representative number from each industrial category must collect a sample four times in a year to monitor benchmark pollutant parameters, and again if benchmark levels are exceeded (National Research Council, 2009). While all industrial permit holders must visually monitor stormwater via a grab sample four times a year, only visual characteristics are noted (National Research Council, 2009). Similarly, construction permit holders are only required to visually interpret stormwater discharge characteristics regularly (National Research Council, 2009).

1.3. Challenges in Regulating and Managing Stormwater

Challenges in regulating and managing stormwater are present, especially when effluent limitations are not set and stormwater discharge is only managed as a pollutant, but there are additional reasons why stormwater management systems to control quantity still mainly consist of traditional concrete conveyance and regional catch basins. Unlike NPDES permits governing wastewater, which limit discharge effluent to a certain federal standard, federal standards for limits of stormwater pollutant discharge are absent and BMP’s utilized are source specific, acting
as a proxy to reduce pollution without monitoring effectiveness (National Research Council, 2009). This is mainly due to the fact that pollution generated by stormwater discharge varies between locations with different geology, topography, land use, and storm characteristics, making generic one-size-fits all solutions ineffective. Therefore, stormwater discharging sources have the ability to create a stormwater management plan that must be approved by meeting requirements of the permitting state (National Research Council, 2009). Because water quality is the sole focus for regulations on stormwater discharge from urban areas, managing water quantity to protect infrastructure and human health from flooding is done so by traditional methods, and degradation of water bodies is ongoing due to high volumes of flow being discharged to natural streams and rivers (Burns et al., 2012).

Shifting ideas towards regulating water quantity and flow within an urban system could ultimately lead to a more holistic land use approach in dealing with stormwater management because of inherent benefits that involve water quality, groundwater, and ecosystem health (Subramanian, 2017). Stormwater management systems that treat flooding and water quality separately through pipe and catch basins and generic structural and non-structural BMP’s, respectively, fail in comparison to systems that use LID or retrofit GI type BMP’s that promote infiltration and natural hydrology (Subramanian, 2017; Roy et al., 2008). Though these practices of managing stormwater are shown to be effective in reducing flow volume, pollutant loads, and increasing groundwater for some cities and states across the U.S. that had stormwater regulations prior to the federal stormwater program, they are not widely utilized because of cost, lack of performance research, and engineering and regulation standards (National Research Council, 2009; Roy et al., 2008).
Unlike wastewater systems, which are funded under the CWA, stormwater system design, implementation, and research is unfunded for stormwater dischargers (National Research Council, 2009). Therefore, projects involving stormwater quality or quantity have to compete for funding with other projects such as infrastructure, public safety, and flood control, which are usually deemed more important. Although, even when GI or LID type designs are more cost efficient than large traditional conveyance systems, they are not utilized because of uncertainties in performance (Roy et al., 2008). Engineers have historically been taught to deal with stormwater through flood control methods, and since these methods are very effective, adopting more complex approaches are difficult even when they can improve groundwater recharge, water quality, and receiving water ecology.

1.4. Case Studies

Although many studies of SCM type, frequency, and distribution are present throughout the literature across various cities and states, the focus of this case study review is of detention basin retrofits due to the relevance to this thesis. It is also important to note that many cities once used or still utilize traditional detention basins as their main technique for stormwater management, and therefore possible retrofit plans can be put in to place instead of other more complex and expensive SCM’s. Typical of research across all types of SCM’s, the outcome of detention basin retrofit studies focus on water quality and quantity, as well as impacts retrofitting has on flood control. Furthermore, case studies that study retrofitting of traditional concrete lined detention basins do so through alterations to the basin channel, floodplains, soil, and vegetation, or employ changes to the outlet structure type, shape, size, and opening/closing frequency, while others employ a combination of the two. Therefore, this review will be organized on the basis of
the aforementioned retrofit types seen in the literature and are subsequently renamed holistic and outlet retrofit approaches.

While information and projects involving holistic detention basin retrofits are often seen with a quick online search, only a handful of studies are available within the literature for performance evaluation. This is likely due to the generally new concept and therefore limited scientific backing for municipalities to employ such renovations. Nonetheless, holistic approaches to detention basins can include the simple removal of concrete-lined channels and planting of native/wetland vegetation, to introduction of sediment forebays and engineered soil material. Although not considered a typical retrofit detention basin, the California Department of Transportation introduced multiple un-lined detention basins within existing highway infrastructure and one concrete-lined detention basin in order to compare removal efficiencies between the two approaches (Taylor et al., 2001). Results show a 68% average concentration reduction of total suspended solids (TSS) for un-lined basins compared to only 23% for the concrete-lined basin, and increased concentration removal of phosphorous (27%), total copper (54%), total lead (70%), and total zinc (70%), while nitrogen and its various forms had a decrease in concentration removal. Although if accounting for total load reduction in un-lined basins, un-lined basins had a better removal efficiency then lined basins for all nutrients and metals. The authors contributed this difference in load reduction between un-lined and concrete-lined basins to infiltration and re-suspension, respectively. Toran, 2016 placed water level loggers at the inlet and outlet during both pre- and post-retrofit periods for a detention basin. The detention basin initially included a concrete-lined channel and mowed grass floodplains, but was replaced by wetland vegetation. Results show that of 20 pre-retrofit period storms greater than 1 cm, all 20 storms showed a water level response at the outlet; In comparison, of 30 post-retrofit
storms greater than 1 cm, only 5 storms showed an outlet water level response, indicative of increased infiltration within the post-retrofit basin relative to the pre-retrofit basin.

Outlet retrofit approaches can be further divided into outlet re-sizing/dual opening and active control structures, where the former usually has a two-stage structure to limit flow for small storms while still maintaining drainage efficiency for larger storms (Hawley et al., 2017, Guo, 2007), and the latter utilizes outlet closure and opening based on water level, monitored TSS, or ponding time (Sharior et al., 2019). The main goals for re-sizing or dual-opening outlets is to increase ponding time for various storms while minimizing downstream flooding and subsequent erosion, therefore a precursor to actual installation are modeling studies to best optimize outlet size and structure (Marcoon et al., 2004). Hawley et al., 2017, optimized a dual opening outlet system to decrease outlet discharge up to the 2-year design storm in order to reduce downstream erosivity while still maintaining the 100-year drainage capacity. Alongside reducing the total duration of erosive flows, the study also showed an increase in ponding time relative to the pre-retrofit basin leading to increased potential for sediment fallout and evaporation. Marcoon et al., 2004, also completed outlet optimization via modeling for various orifice sizes and weir elevations during the water quality storm in order to increase ponding time with minimal upstream and downstream effect and showed a general increase in ponding time and outlet flow with an increase in weir elevation. Though this increase in ponding time and outlet flow became less exponential and more linear with a decrease in orifice size for respective weir elevations. The above studies offer insight in how to better manage detention basins based on optimized static outlet structures, but these static structures still offer limited control for rainfall events that are dynamic and occur infrequently.
Use of automated or controlled valves at outlet structures give water managers the ability to attenuate and abstract smaller storms or allow runoff from successive small and medium storms to pond together before being released downstream. Automated outlet structures within the literature are controlled based on a certain ponding time, ponded water level, or TSS concentration (Sharior et al., 2019; Middleton et al. 2008) while other studies also utilize weather forecasts in order to initiate valve opening remotely (Klenzendorf et al., 2015). Sharior et al., 2019 compared the exceedance of a maximum TSS and water level failure criteria of multiple outlet control techniques that included on/off, detention, and TSS controls to a passive control outlet, and found that the passive control exceeded the TSS threshold more often in comparison to the other outlet control methods. Alternatively, the passive control had the lowest exceedance of the maximum water level threshold, which was greatest for TSS and detention controls. Middleton et al., 2008 monitored the effectiveness of an automatic outlet valve that stayed closed for a set time after inflow to the basin occurred and subsequently would release the ponded volume once the set time was met (12-hours in the study). Results of this study showed a 91% average reduction between influent and effluent TSS, as well as greater than 50% reduction between influent and effluent copper, lead, zinc, nitrogen, and phosphorous. Klenzendorf et al., 2015 show an average reduction of 94% between influent and effluent TSS for a retrofit detention basin that includes a remotely controlled outlet based on water level data, rain gauge, and weather forecast. For the same study area, Gilpin et al., 2014 shows variability in effectiveness of the retrofit basin based on individual storms sizes; While the retrofit basin removed 98% TSS in comparison to 71% TSS for an unaltered detention basin during a 0.33 cm storm, the unaltered basin actually outperformed the retrofit basin in regards to TSS removal for
a larger 1.40 cm rainfall. The retrofit basin outperformed the unaltered detention basin in
removal of E. coli concentration for both a 3.25 cm and 1.40 cm rainfall.

As previously mentioned, few studies exist regarding the performance of holistic type
retrofit detention basins that have been altered through more natural means. These alterations can
include the replacement of existing concrete lined channels with earthen material, introduction of
native vegetation in floodplains, and installation of sediment forebays. The majority of these
studies focus on the numerical difference between TSS, nutrients, and heavy metals at the basin
inlet and outlet. While a couple studies have a water quantity aspect, no studies take a detailed
look at how channel alteration effects the dynamics of a detention basin in terms of flooding for
small storms, which controls the potential for infiltration and evapotranspiration. It is
hypothesized that an increase in flooding at both channels will occur due to increased roughness
induced by a more natural channel and inclusion of wetland vegetation throughout. Additionally,
while there are studies that estimate infiltration for infiltration basins, which are dramatically
smaller in size then a regional detention basin and usually include an engineered soil material,
infiltration within a detention basin is assumed negligible and is therefore not estimated.

Ultimately, this study develops novel techniques utilizing various modeling software
alongside detailed in-situ data to compare pre- and post-retrofit channel implications on flooding
as well as the estimation of ponding time and infiltration amounts over various storms. These
inexpensive techniques that estimate basin ponding time and total infiltration can be utilized in
future case studies involving detention basins to determine performance with regards to sediment
fallout and runoff abstraction, respectively.

Furthermore, “The Fargo Project” is ultimately a natural laboratory and offers the
opportunity for daily field work and observations. These observations focused on sediment
behavior in terms of channel erosion and deposition throughout both the channel and the basin. Though not necessarily in question, this sediment behavior could potentially explain and control processes that are occurring within the basin such as channel conveyance and outlet efficiency.
2. MATERIALS AND METHODS

2.1. Study Site

The Rabanus drainage basin, located within Fargo, North Dakota, is approximately 18 acres and collects stormwater from more than 400 acres of urbanized commercial/retail areas. Up until the summer of 2015 the basin consisted of a concrete channel and mowed grass, acting as temporary storage for stormwater (Figure 1). This pre-retrofit basin served the purpose of collecting runoff from large rain events and subsequently releasing stormwater through the outlet culvert. Since the basin was developed prior to the introduction of Fargo, North Dakota’s stormwater retention policy, which requires retention/detention basins collect and store runoff from the 10- and 100-year rainfall events and subsequently reduce discharge to the respective pre-development rate via a two-stage outlet (Morlan, 2018), the current outlet likely releases runoff from smaller rain events at a rate exceeding the 10-year pre-development discharge rate. Furthermore, with even more frequent rainfalls that are less than the 1-year rainfall event, water was likely able to flow directly through the basin without attenuation due to the concrete channel. The retrofit, known as “The Fargo Project”, eliminated the concrete channel and introduced native vegetation into the basin, which likely alters the previous pre-retrofit basin hydrologic regime in terms of conveyance of small storms and potential increases in infiltration and evaporation.

Through a partnership with Jacki Brookner, an ecological artist, a community process was developed that included engineers, ecologists, neighbors, and designers to re-introduce natural ecology and provide a usable space for people. The main goals of “The Fargo Project” are to create a public space by engaging residents, provide a natural landscape experience by restoring native prairie and wet meadows, and improve stormwater quality by using ecologically-

The Rabanus drainage basin now includes an earthen channel with intermittent vegetation and rock riffles to attenuate the flow of water and promote meandering. Stormwater is conveyed from south and northeast inlets to a northwest outlet. Immediately beneath the northeast inlet, which drains the majority of the contributing area, is a sediment forebay that is designed to decrease suspended sediment contained in the runoff. Alongside the natural channel and throughout the basin, natural prairie vegetation is seen and promotes habitat and water quality as well as infiltration and evapotranspiration (Figure 2). Although pre- and post-retrofit basins provide similar flood protection, peak-flow reduction, and temporary storage for large storms, the post-retrofit basin should provide attenuation and the possibility of increased infiltration and evapotranspiration for small and more frequent storms due to a significant increase in roughness within the channel.
Figure 1: Pre-retrofit Rabanus detention basin consisting of mowed grass and concrete channel (Bing Images).
Figure 2: Post-retrofit Rabanus detention basin known as “The Fargo Project” with an earthen channel, natural vegetation, and various structures for public use. Logger locations are also shown.

2.2. Models

2.2.1. Hydrus-1D

Created by J. Šimůnek, M. Šejna, H. Saito, M. Sakai, and M. Th. van Genuchten, Hydrus-1D is the one-dimensional version of Hydrus-2D/3D that simulates water, heat, and solute movement within variably saturated media made up of heterogeneous material. Although the latter are very important for environmental health and agricultural/plant success, only the former will be discussed due to the scope of this project.
Many studies and models that incorporate infiltration utilize the Green and Ampt equation, which is an analytical solution of the physically derived Richards equation solved within Hydrus-1D. The Green and Ampt equation is ultimately easier to solve because of the limited amount of data needed, though it assumes deep drainage occurs. This assumption cannot be met due to the relatively shallow depth to groundwater at the study site and therefore the Richards equation must be used. The Richards equation is based on Darcy’s law which simplifies a soil into a bundle of straight and smooth tubes in order to linearly relate the hydraulic gradient and a limiting factor, called hydraulic conductivity, to a volume flux of water through a saturated soil profile:

\[ q = \frac{K \Delta H}{L} \]  

where \( K \) is the hydraulic conductivity, \( \Delta H/L \) is the hydraulic head change per unit distance, or hydraulic gradient, and \( q \) is the flux of water moving through the soil (Hillel, 1998). Hydraulic conductivity is unchanged as long as the soil remains completely saturated, and effectively describes the average macroscopic structures within the soil profile for steady conditions. If unsteady conditions prevail, which usually is the case in a natural system, a differential form of Darcy’s law can be used to account for a change in the hydraulic gradient:

\[ q = -K \nabla H \]  

where \( q \) and \( K \) are the same as previously defined and \( \nabla H \) is the hydraulic head within \( x, y, \) and \( z \) coordinates (Hillel, 1998). Although since the near surface zone of a soil profile is constantly changing in regard to saturation, Darcy’s law is ineffective in estimating water flow due to both soil water and matric suction influence on hydraulic conductivity. Richard’s therefore derived the equation:
\[ \frac{\partial \theta}{\partial t} = \nabla \cdot (K(\psi)\nabla H) \]  

(3)

where \( K(\psi) \) is the hydraulic conductivity as a function of matric suction and \( \nabla H \) is as previously defined, this equation uses the continuity equation in order to account for steady and unsteady flow within a soil profile by relating \( \theta \) (soil water content) to \( q \) (flux) in three-dimensional space at each time step. Hydrus-1D simplifies this equation to only account for flow within the vertical direction:

\[ \frac{\partial \theta}{\partial t} = \frac{\partial}{\partial x} \left( K \left( \frac{\partial h}{\partial x} + 1 \right) \right) \]  

(4)

here \( h \) is the water pressure head (negative of suction head), \( x \) is the vertical spatial coordinate, and \( K \) is the result of:

\[ K_{h,x} = K_s(x)K_r(h,x) \]  

(5)

where \( K_s \) is the saturated hydraulic conductivity and \( K_r \) is the relative hydraulic conductivity (Šimůnek et al., 2013). Therefore, allowing Hydrus-1D to model uniform flow in a profile with differing layers of homogeneous material. Hydrus-1D also allows the user to model non-uniform flow through dual-porosity and dual-permeability methods although this will not be discussed further.

Modeling of water flow using the Richards equation is ultimately dependent on how unsaturated hydraulic conductivity is derived, and despite the various options available within Hydrus-1D, only the van Genuchten soil wetness-hydraulic conductivity relation will be discussed. Water content within the soil is directly related to the matric suction, with an increase in matric suction causing a decrease in water content. van Genuchten related this mainly hyperbola-shaped function of matric suction to wetness with the equation:

\[ \theta (\psi) = (1 + \alpha(-\psi)^n)^{-m} \]  

(6)
where $\theta(\psi)$ is the soil water content in relation to matric suction, $\alpha$ is related to an air entry value, and $n$ and $m$ are shape factors of the moisture retention curve and related to the pore size distribution of the soil. The water available to move within the soil, called the effective moisture content, and residual moisture content, or the point where no water is moving within the soil due to an increase in matric suction can then be used estimated the water content with respect to pressure head. The effective moisture content within the soil can be presented by the equation:

$$\theta_e = \frac{\theta - \theta_r}{\theta_s - \theta_r} \quad (7)$$

where $\theta_e$ is effective water content, $\theta_r$ is the residual water content, and $\theta_s$ is the water content at saturation. Re-arranging this equation to solve for the moisture content ($\theta$) and including the previously defined soil wetness-matric suction relation in terms of pressure head ($h$) gives:

$$\theta(h) = \theta_r + \frac{\theta_s - \theta_r}{1 + |\alpha h|^n} \quad (8)$$

when $h < 0$ and:

$$\theta(h) = \theta_s \quad (9)$$

when $h > 0$. Finally the relative hydraulic conductivity can be solved by:

$$K_r(\theta_e) = K_s \theta_e^{\frac{1}{m}} \left(1 - \left(1 - \theta_e^{\frac{1}{m}}\right)^m\right)^2 \quad (10)$$

where $m = 1 - 1/n$ and subsequently used in equation 5 to obtain the unsaturated hydraulic conductivity ($K$). With the above equations, $\alpha$, $n$, $\theta_s$, and $\theta_r$ are independent parameters that must be calibrated experimentally for individual soils. Within Hydrus-1D, a Rosetta program developed by M. Schaap and G. E. Brown Jr. can be used to estimate needed inputs for the van Genuchten water retention parameters and the saturated hydraulic conductivity. The Rosetta program uses pedotransfer functions to statistically estimate water retention parameters and
hydraulic conductivity from soil textural classes, textural percentages, bulk density, and water content based on a library of 2134 and 1306 calibrated samples of water retention and saturated hydraulic conductivity, respectively (Schaap et al., 1998).

Although there have been studies that use Hydrus-1D and the Richards equations for modeling infiltration within various type SCM’s, most studies that utilize Hydrus-1D are geared towards modeling water and solute transport within agricultural disciplines. This can mainly be attributed to the difficulty in obtaining detailed soil characteristics that are subsequently used to parameterize the van Genuchten equation (Lee, 2011). Furthermore, very few studies actually look at infiltration or evapotranspiration components of detention basins due to the assumed negligible amounts, and instead focus on the quality, flooding, or erosion mitigation provided. Kannan et al., 2014, claimed the first study to develop a water balance approach to a detention basin and accounted the infiltration amount as a constant based on the saturated hydraulic conductivity, while Parolari et al., 2018, assumed infiltration to be negligible in comparison to inflow and outflow amounts. Studies done on infiltration basins, which usually include an engineered soil, neglect a low-flow channel and are often much smaller in size compared to detention basins, utilize the Richards or Green and Ampt equation to model infiltration. Cannavo et al., 2018 and Lassabatere et al., 2010 used the Richards equation to show the effect of sediment deposition on infiltration within infiltration basins using atmospheric upper boundary conditions and lower boundary conditions consisting of a constant water table and deep drainage. Lee, 2011 used the Green and Ampt equation to assess the effects of a variable ponded surface head on infiltration estimates due to the geometry simplification of infiltration basins in models. As previously mentioned, the chosen infiltration model used is ultimately determined on the
amount of soil data acquired, but this choice can also be based on whether the assumptions to the Green and Ampt equation can be met.

Hydrus-1D also allows for modeling of potential evapotranspiration using the Penman-Monteith equation, Hargreaves formula, or surface energy balance equation. Similar to that of the differences between Richards and Green and Ampt equations, choice of model heavily depends on data availability, with the Penman-Monteith equation requiring considerably more meteorological data to parameterize. Though due to an in-situ weather station at the site, collection of needed data to parameterize the Penman-Monteith equation was relatively simple and therefore utilized. The Penman-Monteith equation recommended by the Food and Agricultural Organization (FAO) is as followed:

\[
ET_0 = \frac{1}{\lambda} \left[ \frac{\Delta(R_n - G)}{\Delta + \gamma \left(1 + \frac{r_c}{r_a}\right)} + \frac{1.013 \rho (e_a - e_d)}{\frac{r_a}{\Delta + \gamma \left(1 + \frac{r_c}{r_a}\right)}} \right]
\]  

(11)

where \( ET_0 \) is evapotranspiration, \( \lambda \) is the latent heat of vaporization, \( R_n \) is net surface radiation, \( G \) is soil heat flux, \( \rho \) is atmospheric density, \( c_p \) is specific heat of moist air, \( e_a \) is saturation vapor pressure, \( e_d \) is actual vapor pressure, \( r_a \) is aerodynamic resistance, \( r_c \) is crop canopy resistance, \( \Delta \) is vapor pressure curve slope, and \( \gamma \) is psychrometric constant (Šimůnek et al., 2013).

Additionally, \( \Delta \) is given by:

\[
\Delta = \frac{4098e_a}{(T + 237.3)^2}
\]  

(12)

where \( T \) is the average air temperature, and \( \gamma \) is defined as:

\[
\gamma = 0.00163 \frac{P}{\lambda}
\]  

(13)

where \( P \) is the atmospheric pressure (Šimůnek et al., 2013). The equation was derived in order to better estimate evapotranspiration from specific crop or vegetation via parameters related to
surface and aerodynamic resistance, which describe resistance of water vapor loss from the
surface soil/vegetation and directly above the crop/vegetation canopy surface (Allen et al., 1998).

2.2.2. HEC-RAS

Developed by the United States Army Corps of Engineers, Hydraulic Engineering
Center-River Analysis System (HEC-RAS) is widely used to model steady and unsteady state
hydraulic flow through open channels and within floodplains to aide in bridge, culvert, and levee
designs, and to estimate flood inundation during large storm events. While the steady state model
solves for cross-section water surface elevations (WSE’s) via an iterative approach to the energy
equation, which uses friction and contraction/expansion losses of the channel and floodplain
based on the Manning’s equation, the unsteady model uses the St. Venant equation and utilizes
continuity and momentum equations (Brunner, 2016). Although on a much smaller scale in
comparison to damn-break or levee-breach studies, due to large amounts of impervious surface
within the study area’s contributing area and a subsequent decrease in time to peak flow, it can
be assumed that a sudden jump in WSE occurs within the basin and therefore unsteady flow
modeling can be used (Parolari et al., 2018). Furthermore, while most studies use HEC-RAS to
estimate WSE based on observed or modeled discharge rates, Aricò et al., 2009 showed that
modeled peak flow rates using monitored unsteady WSE data were similar to observed peak flow
rates for various events after calibrating for Manning’s channel roughness values. Therefore,
alongside explanation of uniform channel flow computations, further discussion involving HEC-
RAS in terms of unsteady flow modeling and solution to the continuity and momentum equations
will be completed due to relevance in section 2.4.1.

Within the HEC-RAS software, engineers are able to use the Hydraulic Design Functions
tool to design channels in terms of channel geometry and sediment transport capacity, as well as
to determine WSE or discharge of individual cross-sections by solving for an individual variable that is needed to parametrize the Manning equation. The Manning equation can be written as followed:

\[ Q = \frac{1.486}{n}AR^{\frac{2}{3}}S^{\frac{1}{2}} \]  

(14)

where \( Q \) is discharge, \( n \) is Manning’s n value, \( A \) is cross sectional area, \( R \) is hydraulic radius, and \( S \) is energy slope. Additionally, a range of \( n \) values, which are related to channel roughness, are able to be applied across a single cross section in order to solve for the needed parameter (Brunner, 2016).

1-dimensional unsteady flow is calculated by the combination of continuity and momentum equations, which are simplified and solved in HEC-RAS using a finite difference method. The continuity equation states that for a control volume, the rate of mass entering must equal the rate of mass exiting plus mass stored within the control volume. For a volume within a channel the continuity equation can be written as follows:

\[ \frac{\partial A}{\partial t} + \frac{\partial S}{\partial t} + \frac{\partial Q}{\partial x} - q_1 = 0 \]  

(15)

where \( t \) is time, \( Q \) is flow, \( A \) is cross-section area, \( S \) is storage from non-conveying portions of the cross-section, \( q_1 \) is lateral flow per unity distance, and \( x \) is the distance along the channel (Brunner, 2016). Similarly, the momentum equation states that the for a control volume, the momentum entering the volume and the additional external forces acting on the volume must equal to the momentum leaving the system. For a volume within a channel, these external forces consist of hydrostatic pressure, gravity, and friction and are included in the momentum equation as followed:

\[ \frac{\partial Q}{\partial t} + \frac{\partial (VQ)}{\partial x} + gA \left( \frac{\partial z}{\partial x} + S_f \right) = 0 \]  

(16)
where \( Q, A, t, x \) are as previously defined, \( z \) is water surface elevation, \( V \) is velocity, \( g \) is acceleration of gravity, and \( S_f \) is friction slope (Brunner, 2016). Continuity and momentum equations are then solved via an implicit finite-difference method, which simplifies these partial differential equations into linear algebraic equations, through an iterative method called the Skyline Matrix solver (Brunner, 2016).

Various upstream and downstream boundary conditions can be utilized such as a stage hydrograph, flow hydrograph, rating curve, or normal depth in order to begin the solving process and force the model at measured timesteps. This can result in the model becoming unstable if the difference between a computed value and a known boundary condition, such as water surface elevation, is greater than the numerical solution tolerance. To troubleshoot this, the Manning’s \( n \), channel geometry, or timestep must be altered to help the model converge. In this study, detailed stage hydrograph data both upstream and downstream was used to constrain the model alongside a sensitivity analyses of Manning’s \( n \) values for channel roughness in order to solve for discharge at each timestep.

### 2.3. Data Collection

An Onset HOBO U30 weather station was installed immediately adjacent to the stormwater basin to measure rainfall, pressure, temperature, wind speed/direction, relative humidity, and solar radiation at five-minute intervals. Nine Onset HOBO U20L-04 water level loggers were installed at the outlet, both inlets, throughout the channel, and in one shallow groundwater well in order to collect absolute pressure and water temperature data every five minutes (Figure 2). Loggers were read-out and manual water depths were measured every two months to account for error with logger measurement. The data were then adjusted to account for atmospheric pressure using the weather station’s barometric pressure gauge in order to obtain
water depths and subsequently added to the previously obtained datum to determine water surface elevations (WSE).

A detailed survey using a Topcon rotating laser system was carried out in order to measure channel geometry and datum at each logger location as well as dimensions of the sediment forebay beneath the northeast inlet in 2018. Channel geometry of Reach 2 was also previously measured in 2017 and was used to compare successive years in terms of channel erosion. Additionally, a drone flight was carried out to obtain imagery and was later post-processed in Pix4D and ArcMap to develop a three-dimensional point cloud and Digital Elevation Model (DEM) estimation, respectively. Along with the above data collection, periodic inspections were also conducted in order to visually monitor the basin in terms of sediment build up within the outlet and other areas within the basin.

2.4. Methods

24-hour rainfall totals obtained by the Rabanus weather station were compared with the North Dakota Agricultural Weather Network (NDAWN) 24-hour rainfall totals, which is located about 5.6 kilometers northeast of the study site, to determine accuracy and exceedance probabilities for 24-hour storms (Figure 3). Between May 1, 2018 and September 30, 2018, a total of 40.87 cm and 47.69 cm of rainfall were recorded at the Rabanus and NDAWN weather stations, respectively (Figure 4). In regards to precipitation frequency, the August 26 storm with a 24-hour rainfall total of 5.21 cm was equal to the one-year 24-hour storm and also included a 6-hour rainfall amount of 4.47 cm, falling between the 6-hour one-year and two-year frequency event of 4.04 and 4.83 cm, respectively, based on the National Oceanic and Atmospheric Administration’s (NOAA) Precipitation Frequency Data Server. All other recorded storms fell below the 6- and 24-hour rainfall frequency amounts. Since antecedent conditions play an
important role in the variation of runoff and infiltration response to rainfall, individual storms (>0.0254 cm (0.01 in)) were identified visually and individually separated by matching post-storm to pre-storm in-channel water level or when post-storm in-channel water level became stable (Figure 5). Since in-channel water level is a general indicator of the shallow water-table elevation within the basin, it is likely that addition from soil water to in-channel water via accumulated infiltration and interflow has ceased once pre- and post-storm in-channel water levels become similar or once post-storm in-channel water level becomes stable, which indicates an increase in water-table elevation. Therefore, with this rainfall separation technique, it is assumed that pre-storm soil water amounts are similar for each individual storm to satisfy later modeling.

Figure 3: Exceedance probability of 24-hour rainfall amounts NDAWN and Rabanus weather stations. 50% of storms were greater than 0.1524 cm (0.06 in) and 0.23114 cm (0.091 in) for Rabanus and NDAWN, respectively.
Figure 4: Cumulative rainfall (cm) between April and October 2018 for Rabanus and NDAWN weather stations.
Figure 5: Individually separated storms between May 1, 2018 and September 30, 2018 based on matching before storm water levels to post storm water levels, or when post storm water levels become stable. Blue is rainfall intensity (in./5 min.), green is start of storm period, and red is end of storm period.
2.4.1. Comparison of Pre- and Post-Retrofit Channel Conveyance for Small Storms

Since no monitoring program was in place prior to fruition of “The Fargo Project” there is no benchmark for comparing pre- and post-retrofit performance for various storm sizes and intensities. It can be assumed that pre- and post-retrofit basins behave similarly for large storms (> 1-year event) due to unchanged inlet and outlet dimensions/sizes as well as a generally unaltered soil media within the basin. These large storms result in complete flooding of the basin (Figure 6). As a result, it is likely that the majority of hydrological benefits occur during smaller and more frequent storms due to increases in channel roughness and therefore an increased number of events able to pond within the basin, leading to an increase in evaporation and infiltration.

![Figure 6: An example of basin flooding following a large storm event around July 9, 2019. View is towards the northwest.](image)

HEC-RAS models were used to compare pre- and post-retrofit channel conveyance of small storms greater than 0.0254 cm (0.01 in) and less than 0.254 cm (0.1 in). Due to contributing areas being heavily urbanized and therefore impervious, very small storms still generate a flashy, yet variable, response to in-channel water level at various monitoring locations with respect to flooding. Rainfall events greater than 0.254 cm are seen to mostly flood all monitoring points within the channel (Figures 11 & 12). For post-retrofit modeling, individual models were created for each reach (Figure 7). Each models’ geometry consisted of multiple
surveyed cross sections as well as estimates of Manning’s roughness coefficient (n). Manning’s n values were obtained from Sturm et al., 2010 and ranged from 0.04 to 0.08 for the majority of Reach 1 and 2, consistent with an excavated channel with a clean bottom and side-brush, while Manning’s n values of the lower most portion of Reach 1 and 2 ranged from 0.08 to 0.14, consistent with an excavated channel with dense and tall in-channel brush, in order to perform a sensitivity analysis of possible discharge rates. WSE data were used for both upper and lower unsteady flow boundary conditions in each reach over the entire storm duration. An exception was made to storms where Reach 2 logger failed to provide accurate data and therefore a normal depth friction slope equal to the slope of Reach 2 channel bottom (0.003 ft/ft) was used as a lower boundary condition. Post-processing of modeled discharge occurred in Microsoft Excel in order to separate baseflow from stormflow. Additionally, due to Reach 2 being below a sediment forebay, reverse level pool routing was conducted in order to better estimate magnitude and timing of modeled flow into the sediment forebay assuming a level surface and no infiltration or evaporation takes place. This was done by creating a stage-volume relationship of the sediment forebay using surveyed data and by use of the equation:

$$ I(t) = Q(t) + \frac{S(t + \Delta t) - S(t - \Delta t)}{2\Delta t} \quad (17) $$

Where \( I \) is inflow into the sediment forebay (m³/s), \( Q \) is estimated discharge immediately beneath the sediment forebay (m³/s), \( S \) is the volume of water stored in the forebay (m²), and \( t \) is the timestep (seconds) (D’oria et al., 2012).
Pre-retrofit modeling consisted of a combined model of all three reaches (Reach 1, Reach 2, and Outlet (Figure 8) with channel geometry obtained from a 2007 DEM. Manning’s channel n values for each cross section were set to include the concrete lining (0.017) and grass portion (0.03) of the channel based on Sturm et al., 2010, and reach slopes were determined from the 2007 DEM to be 0.001ft/ft and 0.005ft/ft for Reach 1 and Reach 2, respectively. The model also contained an outlet culvert with dimensions, roughness, and slope data obtained from Houston Engineering Inc., 2007 study and is assumed to flow under free-flow conditions within the
model. Since HEC-RAS is a hydraulic model and incapable of modeling loss within the channel or floodplain, it is assumed that negligible losses occur within the channel; this assumption is likely to hold true within the channel due to the relatively shallow depth to water table and underlying clay layer. A hydraulic design function model for uniform flow was run for various cross-sections within Reach 1 and 2 to determine the minimum discharge rate before overtopping occurs for the concrete lining and entire channel. The minimum concrete lining discharge values were estimated to be 0.08 m$^3$/s (3ft$^3$/s) for Reach 1 and 0.25 m$^3$/s (9ft$^3$/s) for Reach 2, while minimum channel conveyance before flooding for Reach 1 and Reach 2 were estimated at 0.28 m$^3$/s (10 ft$^3$/s) and 0.57 m$^3$/s (20ft$^3$/s), respectively. Modeled post-retrofit channel flows for storms less than 0.254 cm. and greater than 0.0254 cm. were then compared to channel capacity of respective pre-retrofit channels to assess flooding frequency within the pre-retrofit basin.
Figure 8: HEC-RAS model of pre-retrofit Rabanus concrete channel with outlet culvert. Red lines show locations of HEC-RAS cross section lines where data was populated (cross section geometry, Manning’s N value, bank stations, and cross section width) via a 2007 DEM, blue lines show channel centerlines, and the gray block at the top center is the outlet culvert. Each cross section within Reach 1 and Reach 2 was run within HEC-RAS’s uniform hydraulic design function model.

2.4.2. Comparison of Pre- and Post-Retrofit DEM’s

Since point clouds created from structure-from-motion techniques such as Pix4D are estimates of a digital surface model (DSM), ArcMap was used to estimate both minimum and average DEM’s. This was done by using ArcMap’s “LAS dataset to raster” tool in order to make two separate DEM estimates of the minimum and average elevation within a 1-meter by 1-meter area. Furthermore, structure-from-motion techniques to estimate a DEM become increasingly inaccurate with dense vegetation and ponded water, therefore channel interpolation was conducted within HEC-RAS. HEC-RAS can interpolate channel surfaces if the user inputs
georeferenced river centerlines, bank stations, and surveyed channel cross-sections within the “View/Edit geometric data” window and can be exported as a raster surface within the “RAS Mapper” window. A raster surface of the sediment forebay was also created using data from the manual survey and the “Natural Neighbor (3D Analyst)” tool in ArcMap. The channel surface, sediment forebay, and estimated minimum and average DEM’s were then combined using the “Mosaic to New Raster” tool to build more accurate minimum and average DEM’s (Figure 9).
Figure 9: Minimum (top) and average (bottom) DEM estimations obtained from drone imagery and processed within Pix4D and ArcMap. DEM units are in meters.

ArcMap’s “Raster Calculator” tool was then used to subtract the 2007 pre-retrofit DEM from the estimated minimum and average DEM’s created from structure from motion techniques. The “Storage Capacity” tool, located within ArcMap’s “Spatial Analyst
Supplemental Tool” toolbox, was then used to develop a stage-volume relationship of the pre- and post-retrofit DEM’s.

2.4.3. Event-Scale Ponded Volume Estimation of Post-Retrofit Basin

A model within ArcMap’s ModelBuilder was created in order to calculate the total volume of water within the detention basin at a one-hour timestep across each individual storm for the average DEM estimate (Figure 10). First, pre-processing of raw WSE data was done for each selected storm in Microsoft Excel by creating a file consisting of “Timestep”, “Logger”, “WSE”, “X Coordinate”, and “Y Coordinate” columns. “Excel to Table” tool in ArcMap was then used to create a point shapefile of logger locations with an attribute table consisting of WSE data at an hourly timestep for each logger. The “Iterate Feature Selection” iterator was used to create a layer of each successive timestep to run through the created model. As each timestep layer is selected through the iterator, a smooth WSE surface is interpolated using the “Spline” tool, the minimum/average DEM is subtracted from the recently created WSE surface and cell values less-than-or-equal-to zero are set to ‘NULL’ using the “Raster Calculator” tool, and the sum of cell values (volume) and total area of cell values greater than zero are calculated using the “Zonal Statistics as Table” tool. The sum (volume) and area is then appended to a table after each timestep run iteration.

Neither stormwater management plans from North Dakota or Minnesota offer a definition of ponding time for detention basins, alternatively from the New Jersey Department of Environmental Protection (NJDEP), ponding time is defined as the time between peak ponded volume and when 10% of peak ponded volume remains within the basin (Marcoon et al., 2004). With this definition it is assumed that initial inputs of water from impervious contributing areas and subsequent growth of ponded water within the detention basin are not conductive for
sediment fallout, and that the ponded surface eventually re-mobilizes sediments as it becomes smaller and is able to drain. From the New Jersey Stormwater Best Management Practices Manual, the removal of sediment via ponding within a detention basin linearly increases from 40% TSS during 12-hours of ponding time to 60% TSS during 24-hours of ponding time. Although the specific TSS fallout percentage based on ponding time cannot be directly assumed and applied to a Fargo, ND study area, the general relationship of increasing TSS fallout with increasing ponding time can be assumed. Because the retrofit basins studied by Marcoon et al., 2004 did not have a low flow channel with permanent water as is present in this study, ponding time in this study will be defined as the time between peak ponded volume and when the ponded volume is back within the confines of the earthen-channel and sediment forebay. In order to simplify the aforementioned model, it is assumed that ponding terminates when the ponded area meets that of the total area of the channel and sediment forebay, which was estimated to be 1593.92 m².
Figure 10: Model created within ArcMap’s ModelBuilder used to estimate ponded volume at hourly timesteps for each storm event.
2.4.4. Estimation of Evaporation and Infiltration

Due to the absence of flow data for Inlet 1, Inlet 2, and the Outlet during the monitoring period, a true event-scale water balance would only be possible through watershed modeling techniques. Although, these techniques are still rough estimates of various components of the water balance unless models are calibrated with in-situ discharge measurements of inflows and outflows to the basin. These measurements are difficult to obtain manually due to the infrequent and flashy nature of urban storms and streams and tools for automated monitoring are expensive. Therefore, an estimation of event-scale infiltration and evaporation amounts were calculated using Hydus-1D for various storm sizes. The typical surface water balance equation is:

$$\Delta S = P + R + G_{in} - O - ET - I$$  \hspace{1cm} (18)

where $\Delta S$ is storage volume, $P$ is precipitation, $R$ is inflow, $G_{in}$ is groundwater becoming surface water, $O$ is outflow, $ET$ is evapotranspiration, and $I$ is infiltration (Lewis et al., 2003).

Alternatively, for a detention basin the equation can be written as:

$$V_{i+1} = V_i + V_{in} - V_{out} + V_p - V_e - V_{inf}$$  \hspace{1cm} (19)

where $V_{i+1}$ is the volume of water in the detention pond at time step end, $V_i$ is initial water volume at beginning of timestep, $V_{in}$ is inflow volume during the timestep, $V_{out}$ is outflow volume during timestep, $V_p$ is precipitation falling on the ponded surface during the timestep, $V_e$ is evaporation from ponded surface during timestep, and $V_{inf}$ is infiltration from the surface area beneath the ponded volume during the timestep (Kannan et al., 2014). With this equation, transpiration is assumed negligible, while rainfall falling on un-ponded areas of the detention basin are also negligible or assumed to exit the system through infiltration or evapotranspiration. Therefore, if the volume of water ponded within the detention basin is known for each timestep, a volume flux compiled of inflow, outflow, precipitation, evaporation, and infiltration is known:
\[ V_{i+1} - V_i = V_{in} - V_{out} + V_p - V_e - V_{inf} \]  

where variables are the same as the previous equation. While \( V_p \) and \( V_e \) are both dependent on the ponded surface area within the detention basin for each timestep and can be estimated using monitored weather data, \( V_{inf} \) is dependent on the ponded surface area, depth and time of ponding, soil properties, and depth to water table within the detention basin.

Although soil survey research is ongoing within the study site, surface and shallow well inspection has shown that the majority of material within the basin is composed of silty clay, though lenses of sand can be seen towards the northern end of the basin. Layering of these respective soil types also vary at different locations and are shown to be underlain intermittently by an effectively impermeable clay layer. Alternatively, since Hydrus-1D depends on many soil hydraulic parameters in order to solve the Richard’s equation, soil data from the United States Department of Agriculture’s (USDA) Web Soil Survey were obtained (Table 1). These data were then used to predict water retention parameters through Hydrus-1D’s built in Rosetta model that utilizes pedotransfer functions and neural network analysis (Schaap et al, 1998).
Table 1: Soil properties obtained from the USDA’s Web Soil Survey and utilized within Hydrus-1D’s Rosetta model to obtain parameters to satisfy the van Genuchten equation.

<table>
<thead>
<tr>
<th>Soil Layer</th>
<th>Sand (%)</th>
<th>Silt (%)</th>
<th>Clay (%)</th>
<th>Bulk Density (g/cm^3)</th>
<th>Water Content (33kPa)</th>
<th>Water Content (1500 kPa)</th>
<th>Saturated Hydraulic Conductivity (cm/hr)</th>
</tr>
</thead>
<tbody>
<tr>
<td>A: 0-20 cm Silty Clay</td>
<td>5.3</td>
<td>44.7</td>
<td>50</td>
<td>1.20</td>
<td>12.2</td>
<td>10.5</td>
<td>0.33125</td>
</tr>
<tr>
<td>AB: 20-50 cm Silty Clay</td>
<td>5.3</td>
<td>44.7</td>
<td>50</td>
<td>1.35</td>
<td>12.1</td>
<td>10.5</td>
<td>0.33125</td>
</tr>
<tr>
<td>Bkssg: 50-104 cm Silty Clay</td>
<td>5.3</td>
<td>44.7</td>
<td>50</td>
<td>1.35</td>
<td>12.1</td>
<td>10.5</td>
<td>0.33125</td>
</tr>
<tr>
<td>Cs: 104-152 cm Silty Clay</td>
<td>5.3</td>
<td>44.7</td>
<td>50</td>
<td>1.35</td>
<td>11.8</td>
<td>9.7</td>
<td>0.33125</td>
</tr>
</tbody>
</table>

Hydrus-1D models were run for individual storms for the length of ponding in order to estimate infiltration and evaporation from the ponded subsurface and surface, respectively. Average hourly ponded depths were obtained from the previously calculated hourly ponded volume of an individual storm event when the ponded volume’s area was greater than the area of the channel and sediment forebay (1593.92 m^2). It is assumed that no infiltration occurs within the channel or sediment forebay due to the persistent occurrence of water within, therefore infiltration only takes place once banks are overtopped. Additionally, due to obvious limitations of Hydrus-1D modeling infiltration of a two-dimensional area, estimates of infiltration during an individual storm’s ponding time can be drastically different if modeled infiltration is able to begin once banks are overtopped compared to when ponded volume’s area is greatest. If modeled infiltration begins once banks are overtopped then the greatest soil water capacity and infiltration rate occurs while the ponded volume is relatively small, and as the ponded area increases, both soil water capacity and infiltration rate decrease, thereby limiting total infiltration.
when ponding is greatest. Conversely, if infiltration modeling begins once peak volume occurs, accuracy of infiltration estimation increases when ponded volume is greatest, but since ponding time is decreased within the model, cumulative infiltration can be underestimated for some storms. Therefore, Hydrus-1D modeling of infiltration for both situations was completed for individual storms to determine a total infiltration range during ponding time. Also, despite the fact that ponded depth varies due to the sloping topography of the detention basin, variable ponded head inputs have been shown to preform similarly to an average ponded head for a Green and Ampt infiltration model, a derivative of the Richard’s equation (Lee, 2011; Warrick et al., 2005). Average hourly ponded depth were input as a variable pressure head upper boundary condition while a deep drainage lower boundary condition was set alongside an initial ground water depth based on monitored data and a soil profile water amount equal to the field capacity.

Meteorological data including solar radiation, maximum and minimum temperature, humidity, and wind velocity at hourly timesteps were also input to estimate evaporation via the Penman-Montheith equation within Hydrus-1D.

Evaporation and infiltration depths for each hourly timestep were then multiplied by the total ponded area and ponded area without the channel and sediment forebay at each respective timestep to estimate evaporation and infiltration volumes.
3. RESULTS

3.1. Low-Cost Monitoring Techniques

Although standalone water level data does not allow advanced analysis of water quantity within the basin, it can show how often channel banks are overtopped at various locations and timing of water level response with respect to rainfall initiation. Channel overtopping results in increased opportunity for infiltration and evapotranspiration within the floodplain compared to being confined within the channel. Figure 11 shows WSE data for Inlet 1, Reach 1_1, Reach 1_2 and Figure 12 shows Reach 2_1, and Reach 2_2 locations within Reach 2. The elevations of both the left and right channel banks at each location are shown as well. For runoff from 25 individual storms at the Inlet 1 location, 28% of the storms stayed within the channel, 12% overtopped one bank, and 60% overtopped both channel banks. At the Reach 1_1 location, 16% stayed within the channel, 12% overtopped one bank, and 72% overtopped both banks. At the Reach 1_2 location only 8% stayed within the channel, 12% overtopped one bank, and 84% of the storms overtopped both banks. At location Reach 2_1 within Reach 2, 24% of storms stayed within the channel, 4% overtopped one bank, and 72% overtopped both banks. At the Reach 2_2 location, 28% of the storms stayed within the channel and 72% overtopped both channel banks (Table 2).

Table 2: Percentage of 25 individual storms that stayed within the channel, overtopped one bank, or overtopped both banks at various logger locations within Reach 1 and Reach 2 of the Rabanus post-retrofit basin.

<table>
<thead>
<tr>
<th>Location</th>
<th>No Banks Overtopped</th>
<th>One Bank Overtopped</th>
<th>Both Banks Overtopped</th>
</tr>
</thead>
<tbody>
<tr>
<td>Inlet 1</td>
<td>28%</td>
<td>12%</td>
<td>60%</td>
</tr>
<tr>
<td>Reach 1_1</td>
<td>16%</td>
<td>12%</td>
<td>72%</td>
</tr>
<tr>
<td>Reach 1_2</td>
<td>8%</td>
<td>12%</td>
<td>84%</td>
</tr>
<tr>
<td>Reach 2_1</td>
<td>24%</td>
<td>4%</td>
<td>72%</td>
</tr>
<tr>
<td>Reach 2_2</td>
<td>28%</td>
<td>0%</td>
<td>72%</td>
</tr>
</tbody>
</table>
Figure 11: WSE’s (blue) for Inlet 1 (a), Reach 1_1 (b), and Reach 1_2 (c) locations between May 1, 2018 and September 30, 2018. Also plotted are left (red) and right (green) bank station elevations to determine when banks are overtopped, as well as rainfall amount (yellow).
Figure 12: WSE’s (blue) for Reach2_1 (a) and Reach 2_2 (b) locations between May 1, 2018 and September 30, 2018. Also plotted are left (red) and right (green) bank station elevations to determine when banks are overtopped, as well as rainfall amount (yellow).
Reach 2 channels surveys taken in 2017 and 2018 show both deepening and widening at most surveyed cross section locations (Figure 13). This can especially be seen towards the middle of Reach 2 (cross sections 3, 4, and 5) where rock-riffles are located and designed to induce meandering and channel widening.

Figure 13: Surveyed cross-sections of Reach 2 from 2017 (orange) and 2018 (blue) showing a general increase in width and depth of the channel allowing for greater conveyance within the channel. There is also lateral movement seen in successive years, especially for cross-section 1 and 5.
3.2. Comparison of Pre- and Post-Retrofit Basin Behavior and Performance

Of the eight storms collected during the monitoring period that were greater than 0.0254 cm. (0.01 in.) and less than 0.254 cm. (0.1 in.), only two storms reached a modeled peak flow rate that exceeded the estimated pre-retrofit channel hydraulic design capacity flow rate of 0.28 m$^3$/s (10ft$^3$/s) and 0.57 m$^3$/s (20ft$^3$/s) for Reach 1 and Reach 2, respectively. In both cases the minimum channel roughness was used for the post-retrofit hydraulic model; no storms exceeded the minimum pre-retrofit channel design capacity when the maximum channel roughness parameters were utilized within the post-retrofit flow model. Figure 14 shows post-retrofit modeled Inlet 1 and Inlet 2 peak flows for both minimum and maximum channel roughness conditions compared to the respective conveyance capacity estimates of the concrete lining and entire channel of pre-retrofit Reach 1 and 2. For Reach 1, 62.5% and 75% of storms stay within the concrete lining for minimum and maximum channel roughness values, respectively, whereas 50% and 75% of the storms stay within the concrete lining of Reach 2 for minimum and maximum channel roughness values. For both minimum and maximum channel roughness values, 25% of storms exceeded the flow capacity of the concrete lining but stayed within the channel of Reach 1, while 37.5% and 12.5% of storms exceeded the concrete lined capacity but stayed within the channel of Reach 2 for minimum and maximum roughness values, respectively. For both Reach 1 and Reach 2 only one storm exceeded respective estimates of channel conveyance capacity.
Figure 14: Post-retrofit modeled peak flow rate of small storms (>0.0254 cm and <0.254 cm) storm for minimum (blue) and maximum (orange) channel roughness values compared with pre-retrofit estimated concrete lining (gray) and channel (yellow) conveyance capacity for Reach 1 (a) and Reach 2 (b).

Comparisons can be made between pre- and post-retrofit channel behavior in terms of flood frequency over the eight storms. Post-retrofit channel flooding for Reach 1 at Inlet 1, Reach 1_1, Reach 1_2, and for Reach 2 at Reach 2_1 and Reach 2_2, based on figures 11 and 12, show sporadic discharge rates for various size storms (Figure 15 & 16). For Reach 1, at least one bank is overtopped in 75% of storms and both banks are overtopped in 50% of storms. Only two of the eight storms stayed within the channel of Reach 1. Whereas for Reach 2, only two storms were able to overtop at least one bank and six of the eight storms stayed within the confines of the channel.
Figure 15: Post-retrofit channel behavior for small storms (<0.254 cm. and >0.0254 cm.) for Reach 1 at Inlet 1 (a), Reach 1_1 (b), and Reach 1_2 (c) locations. Peak water surface elevation (WSE) (blue) is compared to left (orange) and right (gray) bank elevations for each storm in order to determine how often channel flooding occurs.
Figure 16: Post-retrofit channel behavior for small storms (<0.254 cm. and >0.0254 cm.) for Reach 2 at Reach 2_1 (a), Reach 2_2 (b) locations. Peak WSE (blue) is compared to left (orange) and right (gray) bank elevations for each storm in order to determine how often channel flooding occurs.

Both the minimum and average post-retrofit DEM estimates show a significant decrease in ground elevation within the channel and sediment forebay, as well as immediately adjacent to the Reach 1 channel (Figure 17). There is a significant increase in elevation at various locations throughout the basin related to large built structures that are intended for community use.
Figure 17: Minimum (a) and average (b) estimated post-retrofit DEM’s with subtraction of 2007 pre-retrofit DEM. Red and blue indicate an increase and decrease, respectively, in ground elevation compared to pre-retrofit DEM. Units are in meters.
A stage-storage curve was created for the post-retrofit minimum and maximum DEM’s and pre-retrofit DEM (Figure 18). A difference in WSE occurs for similar volumes between the pre-retrofit and post-retrofit average DEM for WSE’s greater than about 272.4 m. For instance, at a volume of 50,000 m$^3$ the WSE for the pre- and post-retrofit is 273.62 and 273.74 m, respectively. The minimum post-retrofit DEM initially has a lower WSE for similar volumes to the pre-retrofit DEM but begins to converge at volumes greater than 20,000 m$^3$.

![Figure 18: Stage-storage relationship of minimum and average post-retrofit DEM’s and the pre-retrofit DEM. The average post-retrofit DEM shows a slight increase in WSE compared to the pre-retrofit DEM with similar volumes.](image)

### 3.3. Event-Scale Estimation of Ponded Volume, Infiltration, and Evaporation

As mentioned previously, infiltration was modeled via Hydrus-1D using two separate scenarios to account for the one-dimensional limitations of the model; Scenario 1 and Scenario 2 represent initiation of infiltration once channel banks are overtopped and once peak ponded volume has been reached, respectively (Figures 19-22). A 0.127 cm storm had a ponding time of 5 hours, total cumulative infiltration of 12.56 m$^3$, and total evaporation of 0.16 m$^3$, accounting
for 2.92% and 0.037% of the maximum ponded volume, respectively (Figure 19). A 1.55 cm storm had a ponding time of 18 hours, total infiltration amount ranging from 1081.73 m$^3$ and 1745.76 m$^3$, and total evaporation of 80.94 m$^3$, making up a loss of 8.53% to 8.55% of the maximum ponded volume (Figure 20). A 2.90 cm storm had a total ponding time of 17 hours, cumulative infiltration between 1049.58 m$^3$ and 1295.17 m$^3$, and evaporation of 25.71 m$^3$, which is a total loss of 9.53% to 11.71% compared to maximum ponded volume (Figure 21). Lastly, a storm of 5.21 cm (equivalent to the 1-year 24-hour storm) had a total ponding time of 36 hours, total infiltration ranging from 2709.02 m$^3$ and 3871.30 m$^3$, and total evaporation of 94.57 m$^3$, equaling a total loss between 5.80% and 8.20% compared to the maximum ponded volume (Figure 22).

Table 3: Rainfall, estimated ponding time, maximum ponded volume, infiltration, and evaporation for various size/intensity storms.

<table>
<thead>
<tr>
<th></th>
<th>Storm 1</th>
<th>Storm 2</th>
<th>Storm 3</th>
<th>Storm 4</th>
</tr>
</thead>
<tbody>
<tr>
<td>Rainfall (cm)</td>
<td>0.127</td>
<td>1.55</td>
<td>2.90</td>
<td>5.21</td>
</tr>
<tr>
<td>Ponding Time (hrs)</td>
<td>5</td>
<td>18</td>
<td>17</td>
<td>36</td>
</tr>
<tr>
<td>Max Ponded Volume (m$^3$)</td>
<td>429.57</td>
<td>13625.21</td>
<td>11282.72</td>
<td>48374.67</td>
</tr>
<tr>
<td>Infiltration (scenario 1) (m$^3$)</td>
<td>12.56</td>
<td>1745.76</td>
<td>1049.58</td>
<td>2709.02</td>
</tr>
<tr>
<td>Infiltration (scenario 2) (m$^3$)</td>
<td>12.56</td>
<td>1081.73</td>
<td>1295.17</td>
<td>3871.30</td>
</tr>
<tr>
<td>Evaporation (m$^3$)</td>
<td>2.92</td>
<td>80.94</td>
<td>25.71</td>
<td>94.77</td>
</tr>
</tbody>
</table>
Figure 19: 7/22/18 storm (24-hr rainfall total of 0.127 cm) ponded volume in black (a), and infiltration/evaporation during ponding time where blue (Scenario 1) indicates infiltration modeling initiation at instance of bank overtopping and gray (Scenario 2) indicates infiltration modeling initiation once peak ponded volume occurs; though for this storm bank overtopping and peak ponding time occurred simultaneously (b). Total ponding time was 5 hours, total infiltration was 12.56 m$^3$, and total evaporation was 0.16 m$^3$. 
Figure 20: 9/02/18 storm (24-hour rainfall amount of 1.55 cm) ponded volume in black (a) and infiltration/evaporation during ponding where blue (Scenario 1) indicates infiltration modeling initiation at instance of bank overtopping and gray (Scenario 2) indicates infiltration modeling initiation once peak ponded volume occurs (b). Total ponding time was equal to 18 hours, while total infiltration was between 1081.73 m$^3$ and 1757 m$^3$, and total evaporation was 80.94 m$^3$. 
Figure 21: 7/19/18 storm (24-hour rainfall amount of 2.90 cm) ponded volume in black (a) and infiltration/evaporation during ponding where blue (Scenario 1) indicates infiltration modeling initiation at instance of bank overtopping and gray (Scenario 2) indicates infiltration modeling initiation once peak ponded volume occurs (b). Total ponding time was equal to 17 hours, while total infiltration was between 1049.58 m³ and 1295.17 m³, and total evaporation was 25.71 m³.
Figure 22: 8/26/18 storm (24-hour rainfall amount of 5.21 cm) ponded volume in black (a) and infiltration/evaporation during ponding where blue (Scenario 1) indicates infiltration modeling initiation at instance of bank overtopping and gray (Scenario 2) indicates infiltration modeling initiation once peak ponded volume occurs (b). Total ponding time was equal to 36 hours, while total infiltration was between 2709.02 m$^3$ and 3871.30 m$^3$, and total evaporation was 94.57 m$^3$.

The exceedance probability of ponding time for storms between May 1, 2018 and September 30, 2018 can be seen in Figure 23. It was determined that 50% of storms had a ponding time equal to or greater than 9-hours whereas 40% of the storms had a ponding time equal to or greater than 12-hours. Additionally, the maximum ponding time that occurred was 36 hours and was the result of a 5.21 cm rainfall, equivalent to the 1-year 24-hour return period storm that occurred 8/26/18, whereas the minimum ponding time was 2 hours.
Figure 23: Exceedance probability of ponding time for storms between May 1, 2018 and September 30, 2018. 50% of storms had a ponding time greater than or equal to 9-hour, 40% had a ponding time greater than or equal to 12 hours. The longest ponding time was 36 hours.

Evidence of sediment deposition was seen at the outlet throughout the majority of the 2018 study and consisted mostly of course gravel and sand material to a depth of about 15 cm. Comparatively, the outlet structure was free of any sediment in May following a period of snow melt (Figure 24).
Figure 24: Outlet structure in May (left) followed by a period of extended snow melt and September (right) after multiple large and intense storms. Deposition of sediment occurs to a depth of about 15 cm.
4. DISCUSSION

As previously mentioned, it is necessary that retrofit detention basins perform equivalent to the pre-retrofit basin with regards to both upstream and downstream flood mitigation. An increased WSE for similar volumes could have an adverse effect for both upstream contributing areas and downstream receiving systems by decreasing drainage efficiency and increasing stress and pressure, respectively (Marcoon et al., 2004). While environmental protection of receiving streams is important, municipalities mainly focus stormwater resources on protection of infrastructure, therefore a seemingly successful retrofit basin with thriving habitat and increased infiltration/evapotranspiration can be deemed unsuccessful if it decreases drainage efficiency of the system. An increase in WSE for post-retrofit basins relative to pre-retrofit basins with similar ponded volume decreases drainage efficiency of the upstream contributing area and increases pressure and stress on the downstream drainage system, ultimately increasing the risk of flooding for large storms.

It is obvious through aerial photographs that the Rabanus drainage basin has changed significantly between pre- and post-retrofit with the addition of various community structures and the excavation of an earthen-channel and sediment forebay. This has ultimately caused a small decrease in the volume capacity of the detention basin in comparison to the pre-retrofit basin (Figure 18). While the methods to obtain a post-retrofit DEM in order to determine the aforementioned result are based on structure-for-motion techniques rather than more accurate methods such as Lidar, this finding can be backed up by comparing peak in-situ WSE data and XPSWMM derived WSE model results from a previous study conducted by Houston Engineering Inc. for the City of Fargo. Although only design storms consisting of the 2-, 5-, and 10-year 6-hour rainfall events were modeled within the study, similar size storms to the 2-year 6-
hour design storm were seen during the monitoring period. The 2-year 6-hour rainfall event (4.52 cm) was modeled within XPSWMM using a constant rainfall rate and the subsequent peak WSE of the pre-retrofit basin was 273.34 m (Houston Engineering Inc., 2007). Comparatively through monitored data, a 3.73 cm storm lasting 9-hours reached an average maximum WSE of 273.27 m, and the 1-year 24-hour storm of 5.207 cm that included an intensity between the 1- and 2-year 6-hour rainfall event reached an average maximum WSE of 273.77 m. Although it is difficult to compare rainfall events with different amounts, intensities, and durations, it is obvious that a smaller, less-intense storm provided a relatively similar peak WSE for the post-retrofit basin compared to a larger, more-intense storm with the pre-retrofit basin. Similarly, the 1-year 24-hour storm achieved a much higher peak WSE in the post-retrofit basin compared to the WSE of the 2-year 6-hour storm in the pre-retrofit basin despite a total rainfall difference of only 0.69 cm.

While a higher WSE could be contributed to an overall decrease in storage capacity of the Rabanus detention basin, it’s likely not the sole reason due to the relatively small difference in pre- and post-retrofit WSE’s for similar volumes (Figure 18). An increase in impervious area both upstream and downstream could increase WSE within the basin due to an increase in runoff, though there is no evidence of a substantial increase in impervious surfaces within the area. Additionally, clogging of the basin outlet could decrease drainage efficiency both to and from the detention basin leading to an increased WSE. Evidence of outlet sediment deposition was easily visible and first noted on 6/20/2018 to be about 15 cm in depth, and although the depth of sediment fluctuated throughout the season it persisted for its entirety. Another reason for the large difference in WSE could be attributed to differences in storm intensity monitored compared to what was used in the XPSWMM model. The 2-year 6-hour storm was modeled as a constant
rate rainfall event (0.753 cm/hr) whereas the 8/26/2018 24-hour 5.207 cm rainfall event had a 3.04 cm/hr intensity for a 30-minute period. This intense period of rainfall likely greatly overwhelmed the outlet drainage efficiency in comparison to the constant rate rainfall that was modeled, and subsequently resulted in a much higher peak WSE.

Since this study did not include any pre-retrofit monitoring program it is difficult to truly assess the effectiveness of the post-retrofit detention basin in comparison to studies that utilize a before-after or side-by-side approach. Therefore, only assumptions can be made based on modeled results with regards to channel conveyance and basin volume capacity. Ponding time, infiltration, and evaporation comparison between pre- and post-retrofit basins for individual storms would have to be completed using more extensive modeling of the entire drainage area and built conveyance systems that are both upstream and downstream of the pre-retrofit detention basin. Although this comparison would be interesting, it is beyond the scope and time-constraints of this study. Rather than focusing on flood mitigation, research involving detention basins is evolving to include more water quality studies, though water balance/hydrological studies on detention basins are still rare. Therefore, this study is important for future case studies in estimating components of the water balance and ponding time of detention basins through utilization of monitored data rather than solely on modeling techniques.

It was hypothesized that since there was a general increase in channel roughness of the post-retrofit channel compared to the pre-retrofit concrete channel, that an increase in flooding would occur for both channels during small, more frequent storms, relative to the pre-retrofit channel, and therefore increase the potential for infiltration and evapotranspiration. WSE data accumulated between May 1, 2018 and September 30, 2018 with subsequent hydraulic modeling of pre- and post-retrofit channels show that Reach 1 floods more often after the retrofit was
completed, whereas Reach 2 behaves similar in the pre- and post-retrofit basin for small storms. While this study only takes into account data collected in 2018, it is entirely possible that at the beginning stages of the “The Fargo Project” and initial introduction of earthen channels in 2015, Reach 2 behaved similar to Reach 1 and flooding occurred more often after retrofitting. Though since 2015, the initial post-retrofit Reach 2 channel has been undercut and down cut due to the large runoff producing impervious area above Inlet 2, ultimately increasing channel capacity. This can be seen from manual surveys of various locations along the Reach 2 channel taken in successive years (Figure 13). Although only one channel survey has been conducted for Reach 1, due to the relatively small impervious contributing area above Inlet 1, it is likely erosion has not occurred to the same magnitude as in Reach 2 and has not impacted channel conveyance. Moreover, the total water volume to reach 1 is lower due to land use differences between contributing areas to Reach 1 and Reach 2; where Reach 2 contributing area consists mostly of impervious area and Reach 1 contributing area consists of an abundance of green-space and various private detention ponds.

Additionally, alongside erosion at Reach 2, WSE’s at Reach 1_2 logger location for larger storms increase before WSE’s at Reach 1_1 logger location increase, indicating that flow from Reach 2 is able to split and flow through both the confluence and up Reach 1, creating a backwater effect at the middle of the basin. While this is not necessarily surprising due to the relatively large and impervious contributing area above Reach 2 in comparison to Reach 1, the densely established channel wetland vegetation located in the middle of the basin and at the end of Reach 2 likely dissipates a lot of the energy provided by the Reach 2 contributing area. As a result, ponding is able to occur instead of being short-circuited directly through the basin and to the outlet.
One of the goals for a SCM is to promote infiltration or evapotranspiration in order to reduce the surface water quantity reaching natural waterbodies. Although this is not the main objective for traditional retention/detention basins, which mainly reduce flooding upstream and attempt to reduce erosion downstream, retrofitting can increase potential for infiltration and evapotranspiration due to the increased channel flooding and ponding times within the basin.

There is also a push for an increase in ponding time for retrofit detention basins which ultimately leads to a greater amount of sediment deposition. Although many of these retrofit designs include a variation to the outlet structure, such as an automatic valve, an increase in channel and floodplain roughness could have a similar but less significant impact on ponding time.

The majority of rainfall events in a given year are relatively small, though due to impervious areas, excessive runoff occurs at erosional rates (Urban Drainage and Flood Control District, 2011). Therefore many detention basins are designed to capture and treat a water quality volume (WQV) equal to the 90th percentile storm; which for the 2018 monitoring period in this study, the 90th percentile storm was about 1.651 cm. Literature regarding ponding/drainage time for WQV treatment range between 24- and 48-hours in order to successfully decrease sediment loading (Urban Drainage and Flood Control District, 2011; Becciu et al., 2015) though 12-hours is also seen as ample ponding time for settlement of most particulates (Blick et al., 2011). The actual definition of ponding time is rarely seen in literature and therefore can be interpreted in multiple ways when trying to monitor and estimate; these interpretations can include the time between initiation of ponding within the basin to complete emptying of the basin, time between peak ponded volume to complete emptying of the basin, time between peak ponded volume to when 10% of peak ponded volume is left, or any other combination. The choice of ponding time method is relatively subjective but can offer insight into total time of attenuation or potential
sediment fallout of individual storm events if considering the beginning of ponding time to be once ponding initiates or when maximum ponded volume is reached, respectively. Within this study, ponding time was considered to be the time between the maximum ponded volume and when the area of ponded volume was less than or equal to the area of the earthen-channel and sediment forebay. This method is similar to that used by the NJDEP, which focuses on a ponding time-sediment fallout relationship and considers initiation of ponding time to be between the maximum ponded volume and when 10% of maximum ponded volume is left, and therefore potentially underestimates total attenuation time of water within the basin. The average time difference between ponded initiation (when WSE’s begin to rise) and peak ponded volume for all individual storms during 2018 monitoring was about 2-hours, therefore making the utilized ponding time method suitable for estimating ponding time in terms of both attenuation and sediment fallout.

The ponding model developed for this study utilizes monitored WSE data and an estimated DEM to estimate ponding time, volume, and surface area of individual storms, this is in contrast with other studies that model stage discharge relationships of various outlets to obtain the same information (Kannan et al., 2014; Parolari et al., 2018). The model used here assumes areas within the basin are inundated if DEM elevations are less than the model created ponded surface elevation at each timestep, and therefore underestimates and overestimates ponded area and volume during regression and transgression of ponding within the basin, respectively. During transgression, raised areas such as small natural levees or man-made walking paths hold back flooding until WSE’s increase to overcome these barriers, while the same structures disconnect the floodplain and channel, allowing for individual ponded areas throughout the basin that are discontinuous from the main ponded area during regression. The multiple disconnected
ponded areas throughout the basin described by the latter process can also have a residence time within the basin that significantly exceeds the main ponded area, resulting in an increase in ponding time and evapotranspiration after a storm event. Unfortunately, the model built and used in this study fails to account for both cases and ultimately simplifies the basin and neglects disconnected areas.

With this simplification of ponded area and volume, results show that 40% of storms had a ponding time greater or equal to 12-hours, which is generally viewed as an ample amount of time for attenuation and sediment fallout of most particulates within a detention basin. It can be assumed that as ponding time increases during individual storm events, the amount of sediment fallout also increases, which for about 10% of the 25 individual storm events, at least 24-hours of ponding time was seen.

While it is likely that sediment fallout occurred as a result of ponding within the basin, sediment deposition was also seen accumulating at the outlet throughout 2018. This deposition of outlet sediment could be the result of channel erosion of Reach 2 as discussed previously, though visual inspection of sediment showed a courser gravel and sand material compared to the silty clay material that makes up the majority of the channel and basin. This courser material appears similar to sediment used during winter months on roadways where it is subsequently entrained by spring snowmelt and rainfalls throughout spring and summer seasons, and finally deposited within the basin. Additionally, in-situ field investigations show that the accumulation of outlet sediment occurring during summer months is eventually entrained during high flows from spring snowmelt and washed further downstream of the basin (Figure 24). Therefore, there seems to be a cyclical process of sediment entrainment and movement during periods of spring snowmelt in contributing areas of the basin that are deposited in both upstream culverts/contributing areas of
the basin, as well as the basin itself. Sediment deposited in upstream culvers/contributing areas of the basin are eventually entrained via large flows caused by intense rainfall events and deposited within the basin, followed by another period of sediment entrainment within the basin that is deposited at the basin outlet. While deposited outlet sediment is not representative of the silty-clay material that makes up the majority of the basin, this finer material could easily be entrained via smaller energy flows and out of the basin without any evidence of outlet deposition. More research would be required to specifically compare this sediment with winter road gravel and understand the total time it requires to move through the stormwater system.

Research involving water balance and hydrologic implications of detention basins is fairly limited in comparison to studies that focus on water quality and drainage efficiency. Studies that have incorporated a traditional water balance method assume infiltration and evaporation occur at a constant rate dependent on ponded area or assume these factors are negligible in comparison to inflow and outflow volumes of the basin. These studies also utilize modeling techniques or expensive monitoring equipment to determine ponded volume. This study incorporates novel methods to estimate ponded volume through utilization of low-cost water level monitors and determines infiltration and evaporation of the ponded volume via Richard’s and Penman-Monteith equations, respectively.

Due to the lack of hydrologic focused studies on detention or retrofit detention basin and absence of pre-retrofit monitoring program, it is difficult to compare infiltration and evaporation estimates of the post-retrofit basin to pre-retrofit rates. Although it is clear that infiltration amounts are dependent on basin ponding time during a storm event as well as the initial ground water depth. For small storms, water is not able to infiltrate entirely due to the short ponding time and relatively small pressure head, therefore only limited infiltration occurs before the basin
is emptied. While large storms have an increased ponding time and pressure head within the basin and therefore have greater potential for infiltration, storms with more intensity but less total rainfall have greater infiltration than larger, less-intense storms. This is due to the sudden increase in ponded volume caused by an intense storm compared to a steady increase in ponded volume during a larger but less intense storm. This difference in ponded volume is the result of both the channel conveyance capacity and size of the basin outlet structure. Intense storms overwhelm the capacity of both the basin channels and outlet quickly following storm initiation, resulting in rapid flooding, while the basin channels and outlet are able to convey most of the runoff produced by larger, less intense storms, leading to a gradual increase in flooding.

Evaporation was dependent on timing of solar radiation and humidity in conjunction with the peak ponded volume area during a storm event. If peak volume occurs during a period of low solar radiation or high humidity there will be limited evaporation compared to if the peak volume occurs during early afternoon hours with maximum solar radiation and low humidity. Even with perfect timing and conditions, evaporation was seen to be relatively insignificant compared to the infiltration component during ponding time of various storm events.

Though this study uses more accurate modeling methods to determine infiltration and evaporation than previous detention basin studies, many assumptions are made regarding soil type/characteristics, initial groundwater level, and soil water movement within the basin using the Hydrus-1D model. As mentioned previously, soils within the basin are surprisingly complex with various sized, oriented, and shaped sand lenses dispersed throughout a mainly silty-clay majority, and an intermittent impermeable clay layer underlying it. The use of the USDA’s Web Soil Survey and subsequent simplification of complex soil layering and zoning to an effectively homogeneous silty-clay profile drastically under-estimates infiltration rates in areas that sand-
lenses occur, though this mostly affects smaller storms that have limited ponding time and pressure head. Additionally, ongoing construction has occurred since the retrofit completion in 2015, likely causing soil compaction and a decrease in infiltration rates that are not modeled. The clay confining layer has been seen to sit relatively deep beneath the surface and below the groundwater level, therefore having no implications on infiltration except for long-term deep drainage into underlying aquifers. It should also be mentioned that the initial groundwater elevation used within Hydrus-1D was based on a single shallow groundwater well monitored and assumed as an average depth to groundwater throughout the basin. Since this monitoring well was situated relatively close to the Reach 2 channel it is likely the depth to water table in that area is more shallow compared to areas that are located on the outer edge of the basin, ultimately reducing the potential soil water capacity and total infiltration of the profile within the model. While this study models vertical water infiltration and movement within the soil profile during ponding time it neglects interflow and evapotranspiration during and after ponding time. Due to the clay impervious layer underlying the basin, very limited deep percolation occurs and therefore previously infiltrated soil water is limited to evapotranspiration or interflow to the basin channel and subsequent exit through the basin outlet. While evapotranspiration can be assumed negligible due to limited evaporation during ponding time, it is likely significantly increased once storms have passed, assuming favorable meteorological conditions. The addition of native vegetation both within channels and throughout the floodplain also likely increases potential evapotranspiration compared to pre-retrofit vegetation.

Future research must include more field work for detailed soil mapping/testing and additional installation/monitoring of shallow wells that are incorporated into a three-dimensional infiltration model. Accounting for soil moisture, soil water movement, and evapotranspiration
between storm events would also be very important to determine the fate of infiltrated water from previous storms.
5. CONCLUSIONS

While there is a push towards more holistic type SCM’s that include components to increase infiltration, evapotranspiration, and water quality, many communities rely on structures that only offer flood control and attenuation, and therefore solutions to alter the urban hydrologic regime to a more natural one are negated. Although these flood control structures are likely necessary in many areas even when more holistic methods are introduced throughout an urban watershed due to limitations of soil infiltration and evapotranspiration for large and intense storm events, many of them are in the form of detention basins with low-flow concrete channels and mowed grass. These previously established detention basins offer an opportunity for renovation to a more natural system that includes an earthen channel and native vegetation, or other structural renovations like forebays and outlet modification. Additionally, retrofitting of detention basins within highly developed urban areas using aforementioned techniques offers a more cost-effective solution in comparison with other watershed-wide techniques.

Studies involving holistic and outlet structure retrofit detention basins showed a reduction in TSS and other chemicals in comparison with traditional detention basin. Studies that focused on water quantity showed that outlet modification allows for more control of detention basin ponding during storms, which increases attenuation, evaporation, and TSS fallout, though increases risk of basin failure through overtopping. More advanced outlet structures that are controlled remotely have the ability to take weather forecasts into account, and therefore potentially decrease basin failure risk. While most studies involve a before-after or side-by-side approach to compare retrofit to non-retrofitted basins in terms of water quality, no studies compare pre- and post-retrofit basin channel performance with respect to flooding during small storms. Likewise, studies that take a hydrologic approach either assume infiltration and
evaporation are negligible in comparison to inflow to and outflow from the basin, or are considered to be constant for the duration of a storm event. Techniques developed via HEC-RAS, ArcMap, and Hydrus-1D based on monitored water level, weather, drone imagery, and survey data provide novel methods to monitor detention basin performance in terms of channel conveyance, basin capacity, ponding time, and infiltration and evaporation estimation.

WSE data monitored between May 1, 2018 and September 30, 2018 and used within a HEC-RAS model to compare pre- and post-retrofit channel performance ultimately showed that the post-retrofit Reach 1 channel floods more often than the pre-retrofit channel and both pre- and post-retrofit Reach 2 channels behave relatively similar. While roughness of the post-retrofit earthen channel has overall increased in comparison to the concrete pre-retrofit channel, extensive erosion has occurred within Reach 2 due to the very impervious urban contributing area upstream, resulting in an increased conveyance capacity. Total pre- and post-retrofit basin volume capacity has also stayed relatively similar, with post-retrofit capacity only slightly lower despite having many newly constructed features such as walking paths, a sediment forebay, viewing area, and natural playground.

Estimation of total infiltration and evaporation over various storm sizes and intensities resulted in a total abstraction that ranged from 2.9% to 11.7% of the maximum ponded volume for respective storms. Greatest infiltration occurred during shorter, more intense storms due to the large pressure head produced by rapid ponding within the basin, whereas larger, less intense storms had a gradual increase in ponding and pressure head, resulting in less infiltration. Ultimately, infiltration was controlled by ponding volume and area during individual storms, as well as initial depth to groundwater. Small storms were unable to infiltrate and saturate the entire soil column due to short ponding times. While initial infiltration is estimated, final fate of
infiltrated water was not accounted for and either resulted in evapotranspiration or interflow to basin channels and possible outflow. Estimation of ponding time for each individual storm showed that 40% of storms between May 1, 2018 and September 30, 2018 had a ponding time greater than 12-hours, while only 10% of storms had a ponding time of at least 24-hours. It is likely that the outlet structure is oversized and therefore does not provide adequate ponding to allow for the maximum amount of sediment fallout. Retrofitting this outlet structure to include a dual size opening could result in extended ponding time for the majority of storms that occur within a season, while also allowing larger infrequent storms to be efficiently attenuated and conveyed downstream.

Many results of this study incorporate a relation to sediment erosion, deposition, and entrainment. Deposited sediment was seen at the outlet over most of the monitoring period and was likely the result of cyclical periods of sediment movement and deposition further downstream from contributing areas/culverts, into the basin, within the outlet structure, and finally out of the basin. While most of the deposited sediment at the outlet structure consisted of coarser gravel and sand material, finer silty-clay material that makes up the majority of the basin was also a likely source for sediment entrainment during various storms and was able to exit the system. This is especially likely in areas of the basin where sparse vegetation is grown, causing loose soil to easily be eroded and entrained during storm events.

With deposition of sediment at the outlet to a depth of around 15 cm, it is possible that conveyance efficiency of the basin was affected due to a decrease in outlet diameter. It is difficult to assume if this would have occurred in the pre-retrofit basin because of the maintained grass throughout the basin and concrete channel, resulting in a decreased potential for erosion compared to the post-retrofit basin. Further research to determine the source of accumulated
outlet sediment and TSS would have to be conducted in order to determine if the post-retrofit basin soil or the area upstream of the basin is the main contributor.

As previously mentioned, it is difficult to assess performance of the post-retrofit basin without pre-retrofit monitoring data, though techniques used in this study could be incorporated into other studies to produce comparisons. Management techniques to reduce loose soil and channel erosion should be emphasized during basin retrofits that introduce native vegetation and earthen channels.
6. REFERENCES


Federal Water Pollution Control Act, 2002.


APPENDIX A. PRE-RETROFIT REACH 1 CONVEYANCE CAPACITY AT VARIOUS CROSS-SECTIONS USING HEC-RAS HYDRAULIC DESIGN-UNIFORM FLOW FUNCTION

Figure A1: Pre-retrofit Reach 1 conveyance capacity at reach station 500.
Figure A2: Pre-retrofit Reach 1 conveyance capacity at reach station 498.
Figure A3: Pre-retrofit Reach 1 conveyance capacity at reach station 496.
Figure A4: Pre-retrofit Reach 1 conveyance capacity at reach station 494.
Figure A5: Pre-retrofit Reach 1 conveyance capacity at reach station 492.
Figure A6: Pre-retrofit Reach 1 conveyance capacity at reach station 489.
Figure A7: Pre-retrofit Reach 1 conveyance capacity at reach station 488.
Figure A8: Pre-retrofit Reach 1 conveyance capacity at reach station 486.
Figure A9: Pre-retrofit Reach 1 conveyance capacity at reach station 485.
Figure A10: Pre-retrofit Reach 1 conveyance capacity at reach station 484.
Figure A11: Pre-retrofit Reach 1 conveyance capacity at reach station 483.
Figure A12: Pre-retrofit Reach 1 conveyance capacity at reach station 482.
Figure A13: Pre-retrofit Reach 1 conveyance capacity at reach station 481.
Figure A14: Pre-retrofit Reach 1 conveyance capacity at reach station 479.
Figure A15: Pre-retrofit Reach 1 conveyance capacity at reach station 478.
APPENDIX B. PRE-RETROFIT REACH 2 CONVEYANCE CAPACITY AT VARIOUS CROSS-SECTIONS USING HEC-RAS HYDRAULIC DESIGN-UNIFORM FLOW FUNCTION

Figure B1: Pre-retrofit Reach 2 conveyance capacity at reach station 798.
Figure B2: Pre-retrofit Reach 2 conveyance capacity at reach station 797.5.
Figure B3: Pre-retrofit Reach 2 conveyance capacity at reach station 797.
Figure B4: Pre-retrofit Reach 2 conveyance capacity at reach station 790.
Figure B5: Pre-retrofit Reach 2 conveyance capacity at reach station 765.
Figure C1: Post-retrofit Reach 1 cross-section geometry and Manning’s N minimum channel roughness values for reach station 770.7072.

Figure C2: Post-retrofit Reach 1 cross-section geometry and Manning’s N minimum channel roughness values for reach station 587.3918.
Figure C3: Post-retrofit Reach 1 cross-section geometry and Manning’s N minimum channel roughness values for reach station 816.987.
APPENDIX D. CROSS-SECTION GEOMETRY AND MANNING’S N VALUES USED IN HEC-RAS USTEADY FLOW MODEL FOR POST-RETROFIT REACH 1

MAXIMUM CHANNEL ROUGHNESS

Figure D1: Post-retrofit Reach 1 cross-section geometry and Manning’s N maximum channel roughness values for reach station 770.7072.

Figure D2: Post-retrofit Reach 1 cross-section geometry and Manning’s N maximum channel roughness values for reach station 587.3918.
Figure D3: Post-retrofit Reach 1 cross-section geometry and Manning’s N maximum channel roughness values for reach station 816.978.
APPENDIX E. CROSS-SECTION GEOMETRY AND MANNING’S N VALUES USED IN HEC-RAS UNSTEADY FLOW MODEL FOR POST-RETROFIT REACH 2

MINIMUM CHANNEL ROUGHNESS

Figure E1: Post-retrofit Reach 2 cross-section geometry and Manning’s N minimum channel roughness values for reach station 375.7501.

Figure E2: Post-retrofit Reach 2 cross-section geometry and Manning’s N minimum channel roughness values for reach station 349.2607.
Figure E3: Post-retrofit Reach 2 cross-section geometry and Manning’s N minimum channel roughness values for reach station 335.7305.

Figure E4: Post-retrofit Reach 2 cross-section geometry and Manning’s N minimum channel roughness values for reach station 299.2859.
Figure E5: Post-retrofit Reach 2 cross-section geometry and Manning’s N minimum channel roughness values for reach station 292.1528.

Figure E6: Post-retrofit Reach 2 cross-section geometry and Manning’s N minimum channel roughness values for reach station 253.0302.
Figure E7: Post-retrofit Reach 2 cross-section geometry and Manning’s N minimum channel roughness values for reach station 223.4644.

Figure E8: Post-retrofit Reach 2 cross-section geometry and Manning’s N minimum channel roughness values for reach station 208.5352.
Figure F1: Post-retrofit Reach 2 cross-section geometry and Manning’s N maximum channel roughness values for reach station 375.7501.

Figure F2: Post-retrofit Reach 2 cross-section geometry and Manning’s N maximum channel roughness values for reach station 349.2607.
Figure F3: Post-retrofit Reach 2 cross-section geometry and Manning’s N maximum channel roughness values for reach station 335.7305.

Figure F4: Post-retrofit Reach 2 cross-section geometry and Manning’s N maximum channel roughness values for reach station 299.2859.
Figure F5: Post-retrofit Reach 2 cross-section geometry and Manning’s N maximum channel roughness values for reach station 292.1528.

Figure F6: Post-retrofit Reach 2 cross-section geometry and Manning’s N maximum channel roughness values for reach station 253.0302.
Figure F7: Post-retrofit Reach 2 cross-section geometry and Manning’s N maximum channel roughness values for reach station 223.4644.

Figure F8: Post-retrofit Reach 2 cross-section geometry and Manning’s N maximum channel roughness values for reach station 208.5352.