University of Minnesota: Twin Cities

CE 4501 Hydrologic Design

Project 5: Modeling Using HEC-HMS

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Cover Letter

Dear Client,

Thank you for hiring our firm for your hydrological analysis. After extensive modeling of your planned work site near the city of Normal, IL, we have concluded that you only need to excavate 4.6% of the total site area for pond-reservoir construction. Of this 4.6%, 2.02% should be used for West Pond, 0.46% for Center Pond and 2.12% for East Pond. That is, our firm recommends constructing each pond such that the maximum surface areas of West, Central and East Pond are 0.497, 0.113, and 0.520 acres, respectively.

Also, it is recommended that West, Central and East Ponds have outlet pipe diameters of 15in, 18in and 24in, respectively. With this combination of pond areas and outlet pipe diameters, the peak outlet flow of the site during a 100 year storm after development will not exceed that of predevelopment, as dictated by Normal, IL city ordinance. This can be seen in figure 2 of the attached report. Also, for safety reasons, the maximum surface elevation of any pond will not exceed 9ft above the base. This can be seen as well in the attached report in figure 4.

While we here at Fo' Drizzle Hydrologic Consulting do all we can to ensure the quality of our products, there are unavoidable imperfections and restrictions in our results that must be noted. For example, while the NOAA is known for providing credible, accurate precipitation data, very limited samples of precipitations depths were pulled from the site in order to model the 25 year and 100 year storm conditions. This limited quantity of data was used in order to facilitate the use of our modeling program, *Hydrologic Engineering Center - Hydrologic Modeling System* (HEC-HMS). While this error is significant, it provides a decent approximation of storm conditions.

Thank you again for contracting our firm for your hydrologic consulting needs. Please consider our services for your future project hydrological planning needs.

-Chris Logston

Contents

0 Introduction

In this project, the client is developing a watershed in proximity to the city of Normal, IL. They have contracted our firm to determine the minimal area than can be allotted to storage pond surface in such a way that any one pond's surface elevation does not exceed 1 ft below its berm, and that the post development watershed peak outflow doesn't exceed that of predevelopment during a 100 year storm. The former condition is set to minimize excavation costs, while the latter is set by the local city ordinance.

The client has provided the predevelopment and post development topographical maps and schematics shown in figures A.1.7, A.1.8, A.1.10 and A.1.10. The client has also provided the pre and post development land-use data shown in tables A.1.5 and A.1.5, respectively. As shown in these maps, there will be only thee ponds in the post development scenario.

The client has allowed our firm to choose the diameter of each pond's outlet pipe in order to assist in minimizing pond area while still meeting the elevation and flow conditions. Each pipe diameter can be either 10, 12, 15, 18 or 24 inches.

In order to facilitate the calculation of storage and flows in both the pre and post development watershed, two corresponding models were created using the Hydrologic Engineering Center - Hydrologic Modeling System (HEC-HMS) program, provided for free by the U.S. Army Corps of Engineers (USACE). Using this model, the maximum surface area and outlet pipe diameter of each lake is adjusted until total pond area is minimized and the conditions of surface elevation and peak flow are met.

Given the total possible combinations between three lakes, 5 outlet pipe diameters, and an infinite possibility of percentage distribution $p_{pond,j}$, there were many trials that could be carried out. Optimization attempts were ran until a target percentage of $p_{tota} = 4.6\%$ was achieved that satisfied the conditions that the post development peak flow resulting from a 100 year storm not exceed that of predevelopment, and that no pond surface elevation exceeded 9 ft above the base.

Storm conditions were created using a frequency storm and an SCS storm for the 25 year and 100 year events, respectively, with precipitation data harvested from the NOAA Hydrometeorological Design Studies Center website.

All data was processed using LibreOffice Calc^{TM}, part of the LibreOffice Suite^{TM}.

1 Methods

1.1 SCS Values

The client has provided extensive land-use data for conditions both before and after development. The soil conservation service (SCS) method is used to compute the the following parameters which are input for each subbasin in the HEC-HMS model.

1.1.1 Weighted CN Equation

The first 5 columns of tables A.1.5 and A.1.5 show the client-provided pre/post development land use. For each sector/area, a weighted curve number (CN_w) is derived by multiplying the researched curve number associated with each i land-use type, and multiplying it by the portion by which that type of land used composes the entire sector/area. These researched land-use curve numbers can be found in table A.1.5.

$$CN_w = \frac{\sum_{i=1}^{n} CN_i A_{Ti}}{\sum_{i=1}^{n} A_{Ti}} \tag{1}$$

1.1.2 Maximum Soil Retention Equation

The potential maximum soil moisture retention (S) for each sector/area is then calculated from the curve number as follows:

$$S_i = \frac{1000}{CN_w} - 10 \tag{2}$$

1.1.3 Initial Abstraction Equation

The initial abstraction (I_a) for each sector/area is then calculated from S as follows:

$$I_a = 0.2S \tag{3}$$

1.1.4 Lag Time Equation

Each sector/area's hydraulic length (HL) and average slope (Y) is provided in columns 7 and 8 of tables A.1.5 and A.1.5. These values are used to calculate each sector/area's lag time (T_p) as follows:

$$T_p = \frac{HL^{0.8}(S+1)^{0.7}}{1900\sqrt{Y}} \tag{4}$$

Equation 4 is an empirical formula which produces a value of T_p in minutes. In order to be compatible with HEC-HMS, this formula is converted to yield values in hours as follows:

$$T_p = \left(\frac{HL^{0.8}(S+1)^{0.7}}{1900\sqrt{Y}}\right)\left(\frac{60\text{min}}{\text{hr}}\right) \tag{5}$$

Inputted Values for T_p can be found in the last column of tables A.1.5 and A.1.5.

1.1.5 Determining Impervious Portion

HEC-HMS also considers the percentage of each sector/area that is impervious. This is derived from researched imperious percentages in a process similar to deriving curve numbers.

$$\%\mathrm{imp}_T = \frac{\sum_{i=1}^{n} \mathrm{imp}_i A_{T\,i}}{\sum_{i=1}^{n} A_{T\,i}} \tag{6}$$

Inputted Values for $\%\mathrm{imp}_T$ can be found in column 11 of tables A.1.5 and A.1.5.

1.2 Muskingum Routing

The Muskingum Method is used to route the flow through each of the 4 channels in both pre and post development. The pre and post development channel lengths are shown in column 2 of tables A.1.5 and A.1.5 respectively.

1.2.1 Travel Time

In this method, a time to travel of a given pulse (K) is calculated for each channel. Assuming an average flow velocity of $V = 2\frac{ft}{s}$, K is calculated for each channel and converted into HEC-HMS compatible units of hours as follows:

$$K = \left(\frac{L}{V}\right)\left(\frac{1\mathrm{hr}}{3600s}\right) = \left(\frac{L}{2\frac{ft}{s}}\right)\left(\frac{1\mathrm{hr}}{3600s}\right) \tag{7}$$

The values of K can be found in column 3 of tables A.1.5and A.1.5.

1.2.2 Stability Criteria

With known values of K, and an assumed appropriate reach paramater of $x = 0.15$, each channel is broken into N subreaches so that the following Muskingum Method stability criteria are met.

$$\frac{1}{2(1-x)} \leq \frac{K}{N\Delta t} \leq \frac{1}{2x} \tag{8}$$

Where Δt is the resolution of the routing data desired from HEC-HMS.

1.2.3 Choosing Number of Subreaches

In order to minimize processing time, the lowest N is chosen that satisfies equation 8.

Each channel's N value and associated criteria are show in columns 4-7 of tables A.1.5and A.1.5.

1.3 Acquisition of Rainfall Data

Rainfall depth data is acquired from the Hydrometeorological Design Studies Center of the National Oceanic and Atmospheric Administration National Weather Service Database. After specifying the location Normal, IL (site 11-6200), the precipitation depth in English units is queried. For the 25 year storm, the corresponding annual maximum precipitation depths for storms of 5 min., 15 min. and 60 min durations are retrieved. For the 100 year storm, the maximum precipitation depth of a 24 hour duration storm is retrieved.

These obtained depths are shown in table A.1.5.

1.4 Elevation-Area Tables

In this project, HEC-HMS is set to use elevation-area tables as reservoir parameters. These parameters are determined by deciding on a target percentage of which total pond area composes the total watershed area p_{total}, and then deciding how this allotment is distributed across all three ponds ($p_{pond,j}$).

1.4.1 Area as a Function of Height

In order to develop these tables, the client-provided pond-slope information of 3(H):1(V) is used to derive a function of area as a function of height.

$$A(H) = \pi(r_{max} - 3(H_{max} - H))^2 \tag{9}$$

Where:
H is the surface height above the pond base;
H_{max} is the height between the pond base and berm;
r_{max} is the maximum surface radius

H_{max} is given to be 10ft. The maximum surface radius is the radius of the pond at H_{max}. This radius is to be determined by deciding what percentage of the total watershed area will be consumed in allotting land for this maximum area.

$$r_{max} = \sqrt{\frac{A_{max}}{\pi}} \tag{10}$$

For a given pond j, the maximum surface area $A_{max,j}$ is determined by multiplying the total watershed area $A_{watershed}$ by the percentage of area it consumes $p_{total,j}$.

$$A_{max,j} = p_{total,j} A_{watershed} \tag{11}$$

The percentage of the area allotted to all ponds that is composed of any one pond is the ratio of that pond's area to total area alloted for ponds.

$$p_{pond,j} = \frac{p_{total,j}}{p_{pond}} = \frac{A_j}{A_{pond}} \tag{12}$$

The percentage of the total watershed area that is composed of any one pond is the ratio between the two areas.

$$p_{total,j} = \frac{A_j}{A_{watershed}} \tag{13}$$

In the minimum, the client has requested that the sum of all maximum pond areas not exceed 10%.

$$p_{total} = \sum_{j=1}^{n} p_{total,j} \leq 10\% \tag{14}$$

In order to minimize excavation costs, the percentage by which all maximum pond areas compose the total watershed area is to be minimized by adjusting pipe outlet sizes and the distribution of p_{total} across all j ponds. This is the optimal scenario.

$$\underset{\varnothing_{pipe}, p_{pond,j}}{\text{MIN}} \quad [p_{total}] \tag{15}$$

The adjustment of outlet pipe diameter $\varnothing$ is discussed in section 1.5.

The area vs. height for each lake for the optimized scenario is shown in columns 2, 4 and 6 of table A.1.5

1.4.2 Elevation Differences Between Ponds

The client has also provided that the bottom elevation of the central pond is 4 ft higher than the east pond. This difference comes into play in calculating the Δz used for the discharge according to the pipe model as discussed in section 1.5.4. However, is just so happens that the orifice discharge Q_o is the controlling discharge for outlets throughout both the 25 yr and 100 yr storm over the post developed subbasin. Therefore, this difference in elevation between ponds is ignored.

1.5 Elevation-Discharge Tables

In this project, HEC-HMS is also set to use elevation-discharge tables as reservoir parameters. This parameter is adjusted by determining an optimal outlet pipe diameters $\varnothing_{pipe}$ for each pond.

1.5.1 Effect of Riser Height

There are several models for discharge as a function of height that apply to the outlet of each pond. However, the client has indicated that the inlet of each pond's discharge pipe will be placed 5 ft above each pond bottom. Therefore, none of these models come into play until the surface elevation is at least as high the inlet of the outlet pipe. This can be see in table A.1.5 as all discharge values up to $H = 5ft$ are zero.

1.5.2 Orifice Discharge Model

The orifice equation describes the rate of flow of liquid through an orifice.

$$Q_o = C_o A_{cs} \sqrt{2g(H - S_0)} \tag{16}$$

Where:
C_o is the coefficient of discharge (0.65 for a sharp tube);
A_{cs} is the pipe cross sectional area, equated;
g is the acceleration of gravity (32.2 $\frac{ft}{s^2}$);
H is the surface elevation of the pond;
S_0 is the height of the outlet.

The cross sectional area of the pipe is a function of diameter ($\varnothing$):

$$A_{cs} = \pi \left(\frac{\varnothing}{2} \right)^2 \tag{17}$$

1.5.3 Weir Discharge Model

The weir equation describes the rate of flow of liquid over a crest.

$$Q_w = C_w(\pi \varnothing)(H - S_0)^{\frac{3}{2}} \tag{18}$$

Where:
C_w is the constant associated with the geometry of the berm (given by client to be 3.3)

1.5.4 Pipe Discharge Model

The pipe equation describes the rate of flow of liquid as dictated by Bernoulli's equation for energy balance.

$$Q_P = A_{cs} \sqrt{\frac{2g\Delta z}{1 + k_e + k_p}} \tag{19}$$

Where:
Δz is the elevation head from the pond water surface to the downstream end of the outlet pipe;
k_e is the inlet's entrance loss (given by client to be 0.7);
k_p is the friction loss coefficient, calculated using equation 21.

As shown in figure A.1.6, Δz is the height between the surface and the outlet pipe outlet. Given a slope of 3%, and a pipe length of 100 ft, the pipe outlet is 3 ft below the pond base.

$$\Delta Z = H + 3' \tag{20}$$

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The friction loss coefficient is a function of pipe length, Manning's roughness n, and pipe interior cross section area.

$$k_p = \frac{2gn^2L}{1.49^2 \left(\frac{\varnothing}{4} \right)^{\frac{4}{3}}} \tag{21}$$

L is given by the client to be 100 ft, and n is researched to be 0.014 for the client-demanded concrete pipe.

1.5.5 Controlling Q

In order to arrive at the most conservative estimate, the actual Q used is whichever is least consider all three models at a specific height.

$$Q_{control}(H) = \text{MIN}[Q_o(H), Q_w(H), Q_P(H)] \tag{22}$$

1.6 Use of HEC-HMS

HEC-HMS allows the user to model flow through a watershed by implementing the inter-flow characteristics of relevant subbasins, channels and reservoirs. The flow being modeled is that which results from storms of specific properties over the watershed.

For the predevelopment model, each sector is modeled as a subbasin, while the inter-sector flow is modeled with a series of channels and junctions. The runoff of each subbasin during a storm is calculated using the soil conservation service (SCS) method, which requires the input parameters of subbasin area A_T, initial abstraction I_a, weighted curve number CN_w, percent impervious $\%_{imp}$ and lag time T_P. The runoff flow through each channel is modeled using the Muskingum routing method, which requires input parameters of pulse time to travel K and reach shape parameter x.

In the post development model, the SCS and Muskingum methods are again used for the subbasins and channels respectively. However, some inter-subbasin flow is intercepted by a set of ponds. The contribution to total post-development flow from these ponds is modeled by each's outflow curve. This outflow curve is interpolated from input elevation-storage tables and elevation-discharge tables.

Two storms of recurrence intervals 25 years and 100 years are modeled to induce subasin runoff, which in turn induced trans-channel flow and pond storage/discharge. For the 25 year storm, precipitation data is researched from the NOAA Hydrometeorological Design Studies Center, and input as a frequency storm. For the 100 year storm, the precipitation data is researched from the same source and input input as an SCS method storm.

The model then produces a set of data, which includes peak outflow for both pre and post development conditions. As shown in figures A.1.11 and A.1.12, this is the peak outflow from junction B for predevelopment and the terminal junction for post. This data is used to verify that $Q_{p,pre} \geq Q_{p,post}$.

For the post development conditions, HEC-HMS also produces data for elevation vs. time for each pond, a sample of which is shown in figure 4 for the 100 yr storm. This data is used to verify that the surface elevation of any one pond does not exceed 1 ft below its berm, or 9 ft above its base.

The target value of total area allocated to pond surface A_{pond} as well as the distribution of that allocation across all three lakes is used to produce the elevation vs. area tables input for each of the post development ponds. As long as the conditions of $H \leq 9ft$ and $Q_{peak,pre,100} \geq Q_{peak,post,100}$ are met, this target value is met. In the optimization process, the distribution of A_{pond} as well as outlet pipe diameters are adjusted until both conditions are satisfied.

2 Sample Calculations

2.1 SCS Method

2.1.1 Curve Number Calculation

Using the given land-use values in table A.1.5, the wighted curve number for predevelopment sector IV is calculated using equation 1 as follows:

$$CN_w = \frac{A_{woods}CN_{woods} + A_{meadow}CN_{meadow} + A_{brush}CN_{brush} + A_{paved}CN_{paved}}{A_{woods} + A_{meadow} + A_{brush} + A_{paved}}$$

$$CN_w = \frac{(1.289)(70) + (1.235)(71) + (1.194)(77) + (0.061)(98)}{1.289 + 1.235 + 1.194 + 0.061} = 73.0$$

2.1.2 S Number

The potential maximum soil moisture retention for predevelopment sector IV is calculated using equation 2 as follows:

$$S = \frac{1000}{73.0} - 10 = 3.7004$$

2.1.3 Initial Abtraction

The initial abstraction for predevelopment sector IV is calculated using equation 3 as follows:

$$I_a = 0.2(3.7004) = 0.7401$$

2.1.4 Lag Time

Using given values of average subbasin slope and hydraulic length given in table A.1.5, the lag time (in minutes) for predevelopment sector IV is calculated using equation 4 as follows:

$$T_p = \left[\frac{(450)^{0.8}(3.7004 + 1)^{0.7}}{1900\sqrt{2.5}}\right]\left(\frac{60\text{min}}{\text{hr}}\right) = 7.9\text{min}$$

2.2 Muskingum Routing

2.2.1 Time to Travel

With an assumed water flow velocity of $V = 2\frac{ft}{s}$, the time to travel for predevelopment channel 1 is calculated using equation 7 as follows:

$$K = \left(\frac{630}{2}\right)\left(\frac{1\text{hr}}{3600s}\right) = 0.0875\text{hr}$$

2.2.2 Stability Criteria

With a safely assumed Muskingum reach parameter of $x = 0.15\frac{ft}{ft}$, and a resolution of $\Delta t = 1min = \frac{1}{60}hr$, the number of subreaches needed to satisfy the stability criteria is determined using equation 8 as follows:

$$\frac{1}{2(1-0.15)} \leq \frac{0.0875}{N(\frac{1}{60}\text{hr})} \leq \frac{1}{2(0.15)} \to 0.588 \leq \frac{5.25}{N} \leq 3.33$$

$$N = 1 \to 0.588 \leq 5.25 \nleq 3.33$$

$$N = 2 \to 0.588 \leq 2.625 \leq 3.33$$

Therefore, 2 subreaches are used for routing flow through predevelopment channel 1.

2.3 Storage Relationship

2.3.1 Pond Surface Area

The total watershed area is calculated by summing the given areas of all sectors. This value should be the same for both pre and post development.

$$A_{watershed,predevelopment} = A_{T,I} + A_{T,II} + A_{T,III} + A_{T,IV} + A_{T,V}$$

$$\to A_{watershed,predevelopment} = 5.159 + 8.528 + 3.074 + 3.779 + 4.029 = 24.570 \text{ ac.}$$

$$A_{watershed,postdevelopment} = A_{T,ctr.pnd} + A_{T,w.ctrl} + A_{T,s.out} + A_{T,w.rwy.} + A_{T,east} + A_{T,w.pd.e}$$

$$\to A_{watershed,postdevelopment} = 4.401 + 4.134 + 2.679 + 3.110 + 8.768 + 1.48 = 24.570 \text{ ac.}$$

With a target percentage of the total watershed area dedicated to ponds of $p_{total} = 4.6\%$, the total pond area is calculated as follows:

$$A_{pond} = p_{total}(A_{watershed}) = \left(\frac{4.6}{100}\right)(24.570ac.) = 1.130 \text{ ac.} \tag{23}$$

The distribution of total pond area amongst the three ponds is then adjusted in the optimization process. For the optimal scenario:

$$p_{pond,west} = 44\ \%$$
$$p_{pond,central} = 10\ \%$$
$$p_{pond,east} = 46\ \%$$

the maximum surface areas of each lake are calculated using equation 11 as follows:

$$A_{max,west} = \left(\frac{44}{100}\right)(1.130) = 0.497 \text{ ac.}$$

$$A_{max,central} = \left(\frac{10}{100}\right)(1.130) = 0.113 \text{ ac.}$$

$$A_{max,east} = \left(\frac{46}{100}\right)(1.130) = 0.520 \text{ ac.}$$

2.3.2 Percentage of Total Watershed Area

The sum of the pond maximum surfaces areas over the total watershed is confirmed to be the target percentage of $p_{total} = 4.6\%$ as follows:

$$\frac{A_{max,west} + A_{max,central} + A_{max,east}}{A_{watershed}} = \frac{0.497 + 0.113 + 0.520}{25.570} = 0.046 = 4.6\%$$

2.3.3 Orifice Discharge Model

The diameter of each ponds' outlet pipe is adjusted in the optimization process. For the optimal scenario:

$$\begin{aligned} \varnothing_{pond,west} &= 15 \text{ in} \\ \varnothing_{pond,central} &= 18 \text{ in} \\ \varnothing_{pond,east} &= 24 \text{ in} \end{aligned}$$

In order to calculate the orifice discharge from west pond with a surface elevation of $H = 5.5$ft, the interior cross sectional area is calculated using equation 17 as follows:

$$A_{cs} = \pi \left(\frac{15\text{in}\frac{1\text{ft}}{12\text{in}}}{2}\right)^2 = 1.227 \text{ ft}^2$$

The orifice model discharge is then calculated using equation 16 as follows:

$$Q_o = (0.65)(1.227)\sqrt{2(32.2)(5.5 - 5.0)} = 4.526\frac{ft^3}{s}$$

2.3.4 Weir Discharge Model

The weir discharge of the west pond outlet pipe at an elevation of $H = 5.5$ ft is calculated using equation 18

$$Q_w = (3.3)\left(\pi 15\text{in}\frac{1\text{ft}}{12\text{in}}\right)(5.5 - 5.0)^{\frac{3}{2}} = 4.579\frac{ft^3}{s}$$

2.3.5 Pipe Discharge Model

In order to calculate the pipe discharge from west pond with a surface elevation of $H = 5.5$ft, the friction loss coefficient is calculated using equation 21 as follows:

$$k_p = \frac{2(32.2)(0.014)^2(100)}{1.49^2\left(\frac{15\text{in}\frac{1\text{ft}}{12\text{in}}}{4}\right)^{\frac{4}{3}}} = 2.681$$

The pipe model discharge is then calculated using equation 19 as follws:

$$Q_P = 1.227\sqrt{\frac{2(32.2)(5.5+3)}{1+0.7+2.681}} = 13.715\frac{ft^3}{s}$$

2.3.6 Control Discharge

The west pond discharge to actually be input into HEC-HMS for a height of $H = 5.5$ ft and outlet diameter of $\varnothing = 15$ in is determined by choosing the model of least discharge as shown in equation 22.

$$Q_{control}(5.5) = \text{MIN}[4.526\frac{ft^3}{s}, 4.579\frac{ft^3}{s}, 13.715\frac{ft^3}{s}] = 4.526\frac{ft^3}{s}$$

This value can be seen in the west pond discharge column at $H = 5.5$ ft in table A.1.5.

3 Results

3.1 Plots

3.1.1 Outlet Hydrographs

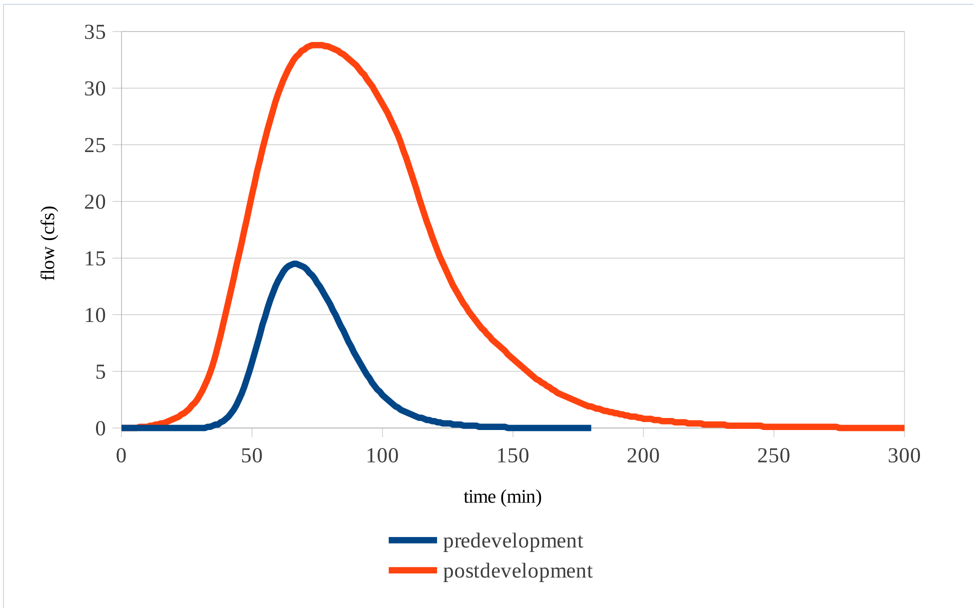

Figure 1: Outlet Hydrograph of pre/post development watershed during 25 year storm

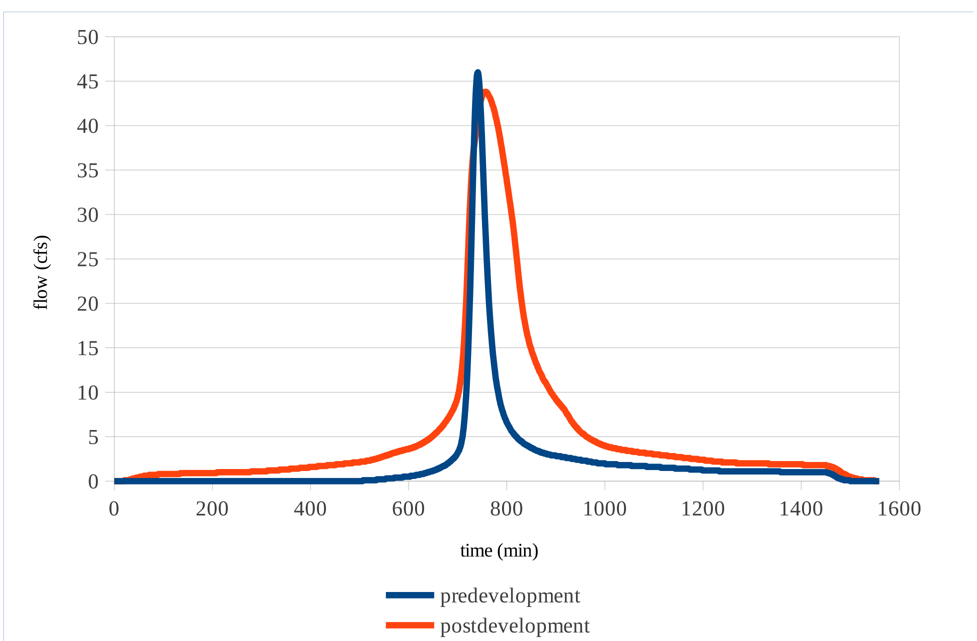

Figure 2: Outlet Hydrograph of pre/post development watershed during 100 year storm

3.1.2 Pond Surface Elevations

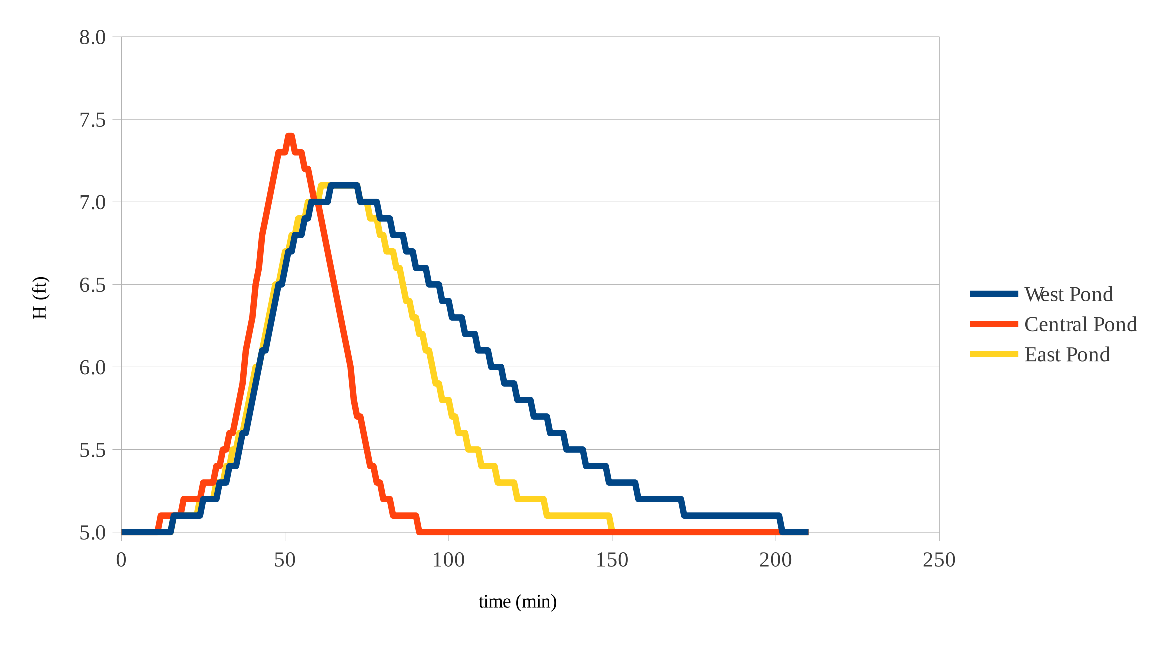

Figure 3: Ponds' surface elevation during 25 year storm

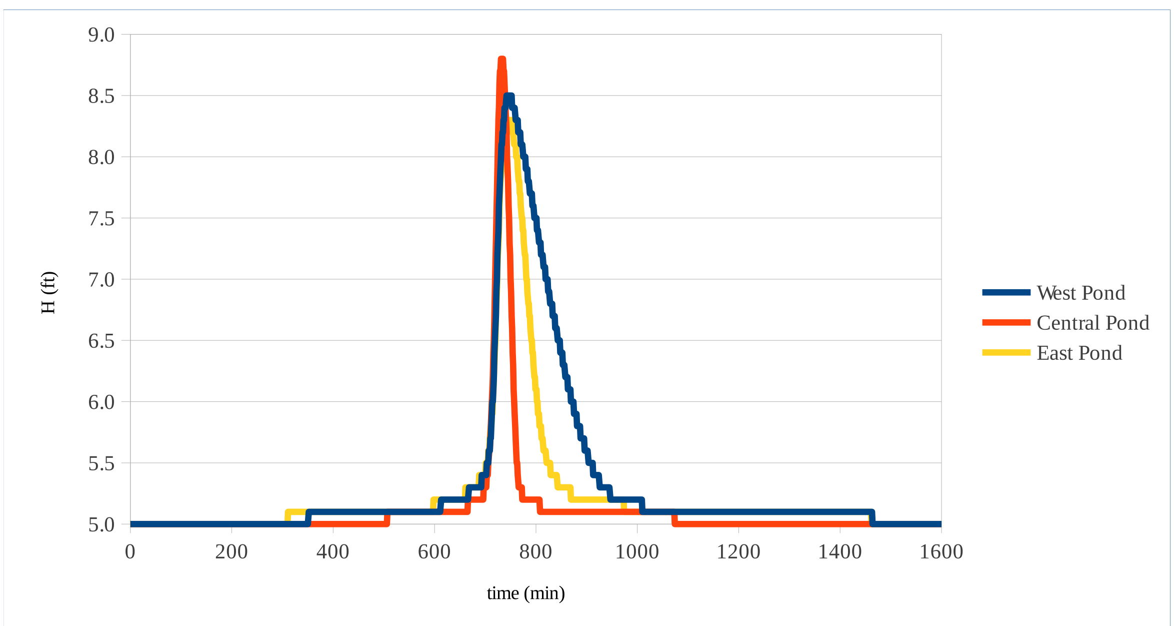

Figure 4: Ponds' surface during 100 year storm

3.2 Tables

3.2.1 Final Design Values

The percent of the entire watershed that is allocated to pond area was minimized to 4.6 %. This was achieved with the values lake area values and pipe diameters shown in table 1.

	West Pond	Center Pond	East Pond	sum
A_{max} (ac.)	0.497	0.113	0.520	1.130
p_{total}	2.02%	0.46%	2.12%	4.60%
p_{pond}	44%	10%	46%	100.00%
$\varnothing_{pipe}$ (in)	15	18	24	

Table 1: Final Design Values

3.2.2 Storm Values

	25 yr, pre	25 yr, post	100 yr, pre	100 yr, post
$Q_{peak}(\frac{ft^3}{s})$	14.5	3.8	46	43.8
T_p (min)	67	75	741	756

Table 2: Peak flow and time to peak for both storms pre and post development

3.2.3 Pond Areas

As shown in the table below, the total area allotted to ponds is far less than 10 % of the total watershed area. In fact, it is only 3.9%.

	Area (ac)
A_{west}	0.452
A_{center}	0.113
A_{east}	0.565
A_{pond}	1.130
$A_{watershed}$	24.572
4.6% of $A_{watershed}$	1.130
10% of $A_{watershed}$	2.457

3.2.4 Pond Design

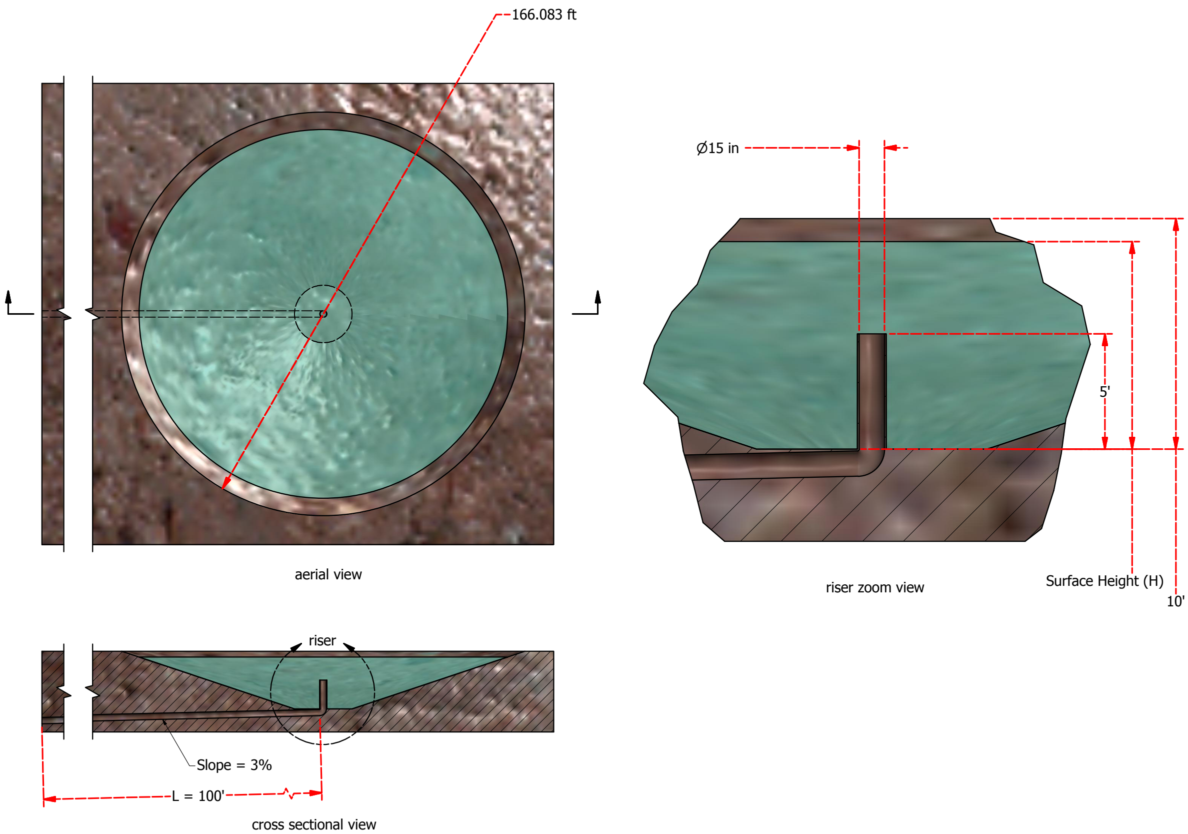

Figure 5: West pond schematic

4 Discussion

The design criteria of having no more than 10 % of the total watershed area be allotted to maximum pond surfaces was met and surpassed. In fact, a percentage of $p_{total} = 4.6\%$ was achieved after several optimization trials. This target percentage was realized by adjusting the distribution of total pond area amongst the three ponds as well as their outlet pipe diameters.

During the optimization process, a target percentage for p_{total} was set. With this value known, an area for total area allotted to combined maximum surface areas for all three ponds (A_{pond}) was obtained. At this point, the optimization process truly began as the the distribution of A_{pond} amongst all three ponds was adjusted so that all conditions were met.

The first condition was that all pond surface elevations remain within 9 ft above their bases ($H_max \leq 9$ ft). The second was that the predevelopment peak outflow during a 100-year storm not be exceeded by the postdevelopment peak outflow during the same event $Q_{p,100,pre} \geq Q_{p,100,post}$.

Increasing the maximum surface area A_{max} of a pond reduced the tendency of its surface elevation to exceed the maximum height, as the storage increased. The drawback was that this increased A_{max} required a larger percentage consumed of the total pond allotment budget $p_{pond,j}$. Given the constant a constant p_{pond} set by the target percentage initially set, the increase in A_{max} of any one pond removed that afforded to the other two ponds.

Increasing a pond's outlet pipe diameter also reduced its overflowing tendencies, but this also increased trans-channel flow, resulting in an increased outlet flow. This negatively affects the $Q_{p,post}$ during the 100 year storm by causing it to increase.

By adjusting the distribution of A_{pond} across West, Center and East Pond as well as their respective outlet pipe diameters until both conditions are met, an adequate scenario is determined for the target percentage.

This process is carried out for increasingly low target values of p_{total} until the time-marginal value of the time spent working on the project is surpassed by the time-marginal value of non-project related activities.

Sources of error include the time-resolution (Δt) used for the routing method as well imprecise data obtained from the NOAA website. A high Δt is liable for error as a more precise, higher value of Q_p could occur between any two sampling points, the distance between of which is set by Δt. Although this former error can be mitigated by increasing the time-resolution, HEC-HMS along accepts a Δt as low as 1min. Therefore, this is the functional minimum.

5 References

Discharge Equations and Curve Numbers:
Hydrology and Floodplain Analysis, 5^{th} *ed.* by Bedient, et al.

Storm Precipitation Data:
Hydrometeorological Design Studies Center
National Oceanic and Atmospheric Administration (NOAA) National Weather Service Database
http://hdsc.nws.noaa.gov/hdsc/pfds

Watershed Model:
Hydrologic Engineering Center - Hydrologic Modeling System (HEC-HMS)
U.S. Army Corps of Engineers (USACE)

All data was processed using LibreOffice Calc^{TM}, part of the LibreOffice Suite^{TM}.

A Appendix

A.1 Model Inputs

A.1.1 Subbasin Parameters

	woods	meadow	brush	res. $\frac{1}{4}$ ac. lts	res.$\frac{1}{2}$ ac. lts	park, grass.$> \frac{3}{4}$	paved
CN	70	71	77	83	80	74	98
% imp.	0	0	0	38	25	0	100

Table A.1.5: Researched subbasin land-use curve numbers and impervious percentages

Sector	A_{woods} (ac.)	A_{meadow} (ac.)	A_{brush} (ac.)	A_{paved} (ac.)	A_T (ac.)	HL (ft)	Y %	CN_w	I_a (in.)	%imp$_T$ %	T_p (min)
I	0.000	3.621	1.538	0.000	5.159	880	1.9	72.8	0.7477	0.0000	15.4
II	1.020	5.502	2.008	0.000	8.528	1440	2.8	72.3	0.7659	0.0000	19.1
III	0.143	1.130	1.801	0.000	3.074	650	3.2	74.5	0.6857	0.0000	09.0
IV	1.289	1.235	1.194	0.061	3.779	450	2.5	73.0	0.7401	1.6142	07.9
V	1.701	0.554	1.774	0.000	4.029	1260	1.5	73.2	0.7315	0.0000	22.9

Table A.1.5: Pre-development subbasin parameters

Area	$A_{rs.\frac{1}{4}}$ (ac.)	$A_{rs.\frac{1}{2}}$ (ac.)	$A_{pk,grs.>\frac{3}{4}}$ (ac.)	A_{paved} (ac.)	A_T (ac.)	HL (ft.)	Y %	CN_w	I_a (in.)	%imp$_T$ %	T_p (min)
Ctr. Pnd.	2.817	0.000	0.000	1.584	4.401	860	1.1	88.4	0.2625	60.3149	12.1
W. Ctrl.	3.463	0.000	0.000	0.671	4.134	590	1.2	85.4	0.3410	48.0634	9.5
S. Out	0.000	2.618	0.000	0.061	2.679	650	0.9	80.4	0.4873	26.7077	14.1
W. Rwy.	2.320	0.000	0.000	0.790	3.110	670	1.8	86.8	0.3039	53.7492	8.2
East	7.343	0.000	0.000	1.425	8.768	775	1.3	85.4	0.3409	48.0764	11.4
W. Pd. E.	0.000	0.000	1.425	0.055	1.480	358	1.0	74.9	0.6705	3.7162	9.8

Table A.1.5: Post-development subbasin parameters

A.1.2 Reach Parameters

Channel	L (ft)	K (hr)	N	$\frac{1}{2(1-x)} <$	$\frac{K}{N\Delta t}$	$< \frac{1}{2x}$
1	630	0.0875	2		2.625	
2	1450	0.2014	4	0.588	3.021	3.333
3	480	0.0667	2		2.000	
4	210	0.0292	1		1.750	

Table A.1.5: Pre-development routing parameters

Channel	L (ft)	K (hr)	N	$\frac{1}{2(1-x)} <$	$\frac{K}{N\Delta t}$	$< \frac{1}{2x}$
1	750	0.1042	2		3.125	
2	1850	0.2569	5	0.588	3.083	3.333
3	1400	0.1944	4		2.917	
4	880	0.1222	3		2.444	

Table A.1.5: Post-development routing parameters

A.1.3 Rain Fall Depths

time	depth (in)	
	25 yr	100 yr
5 min	0.738	
15 min	1.39	
60 min	2.55	
24 hr		6.58

Table A.1.5: Inputted rainfall depths of 25 and 100 year storm

A.1.4 Elevation, Area and Discharge Tables

	West Pond		Center Pond		East Pond	
H	Area	Discharge	Area	Discharge	Area	Discharge
(ft)	(ac.)	$(\frac{ft^3}{s})$	(ac.)	$(\frac{ft^3}{s})$	(ac.)	$(\frac{ft^3}{s})$
0.0	0.203	0.000	0.007	0.000	0.218	0.000
0.5	0.215	0.000	0.009	0.000	0.230	0.000
1.0	0.227	0.000	0.011	0.000	0.242	0.000
1.5	0.239	0.000	0.014	0.000	0.255	0.000
2.0	0.251	0.000	0.018	0.000	0.268	0.000
2.5	0.264	0.000	0.021	0.000	0.281	0.000
3.0	0.278	0.000	0.025	0.000	0.295	0.000
3.5	0.291	0.000	0.029	0.000	0.309	0.000
4.0	0.305	0.000	0.034	0.000	0.323	0.000
4.5	0.319	0.000	0.038	0.000	0.338	0.000
5.0	0.334	0.000	0.044	0.000	0.353	0.000
5.5	0.349	4.526	0.049	5.498	0.368	7.331
6.0	0.364	6.401	0.055	9.218	0.384	16.387
6.5	0.380	7.840	0.061	11.289	0.400	20.070
7.0	0.395	9.053	0.067	13.036	0.416	23.175
7.5	0.412	10.121	0.074	14.575	0.432	25.911
8.0	0.428	11.087	0.081	15.966	0.449	28.384
8.5	0.445	11.976	0.089	17.245	0.467	30.658
9.0	0.462	12.803	0.097	18.436	0.484	32.775
9.5	0.480	13.579	0.105	19.554	0.502	34.763
10.0	0.497	14.314	0.113	20.612	0.520	36.643
$p_{pond,j}$	44		10		46	
$p_{total,j}$	2.024		0.46		2.116	
A_j (ac.)	0.497		0.113		0.520	
$\varnothing_{pipe}$ (in)	15		18		24	

Table A.1.5: Elevation, area and flow Data for all lakes in optimized scenario ($p_{total} = 4.6\%$)

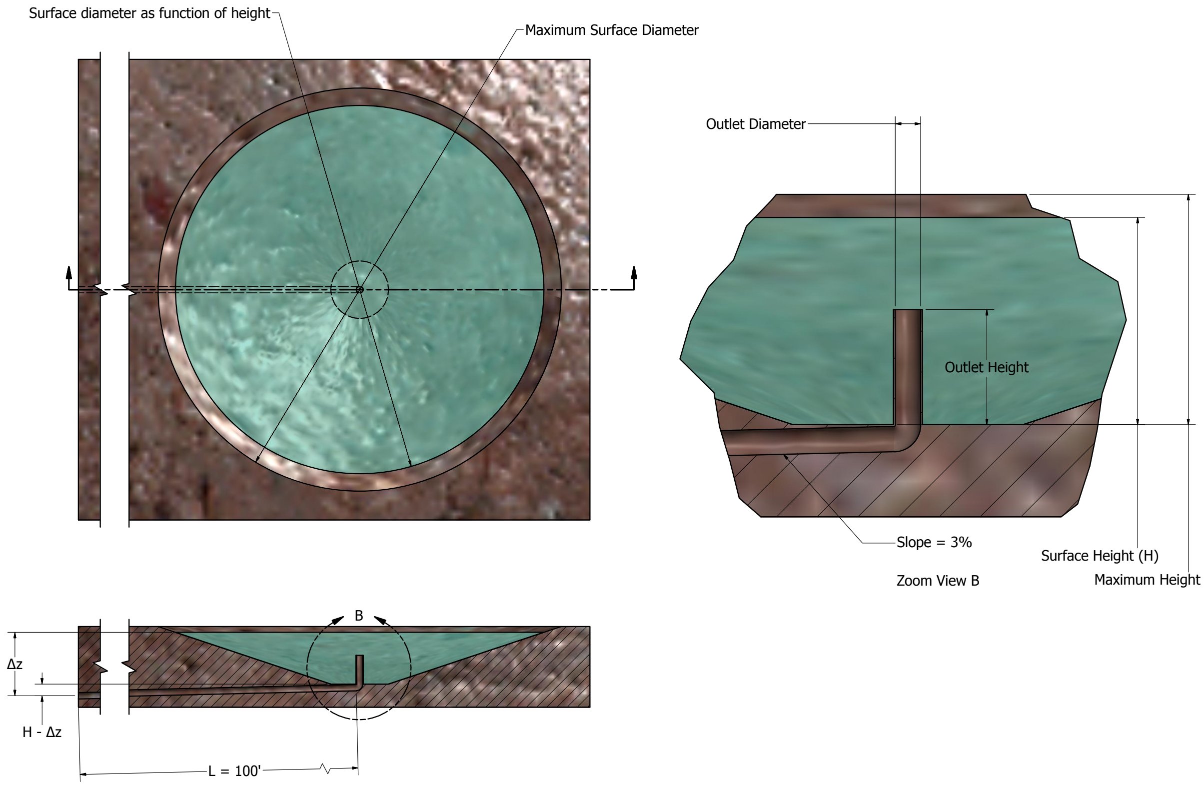

Figure A.1.6: Schematic of sample pond

A.1.5 Pre and Post Development Maps

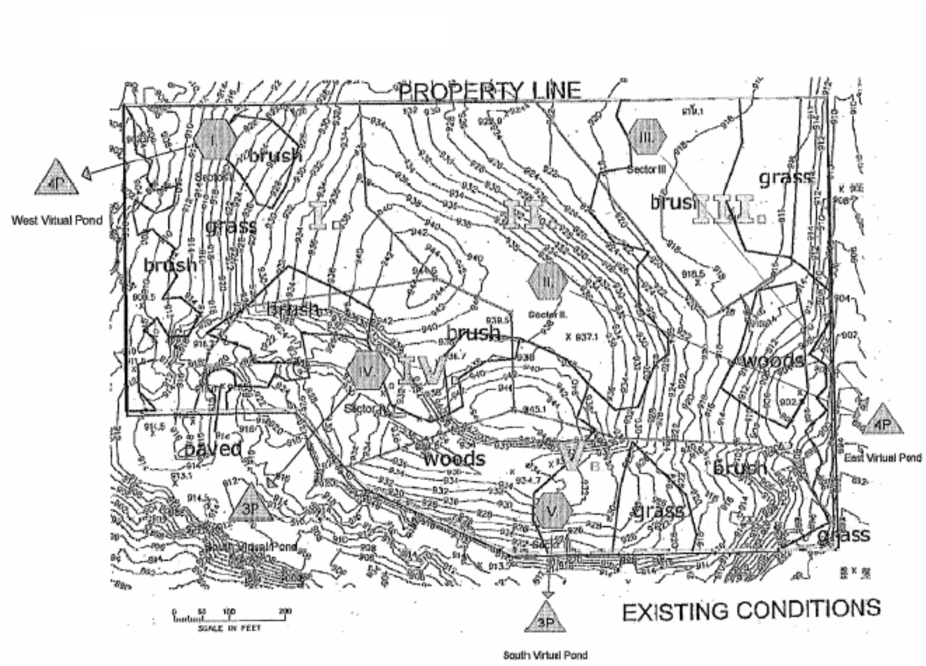

Figure A.1.7: Topographical map of existing, predevelopment conditions

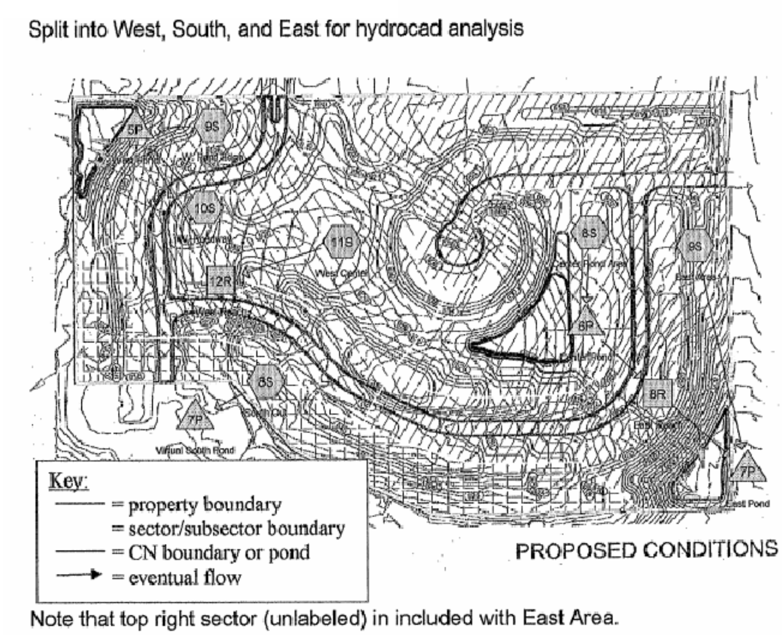

Figure A.1.8: Topographical map of proposed, postdevelopment conditions

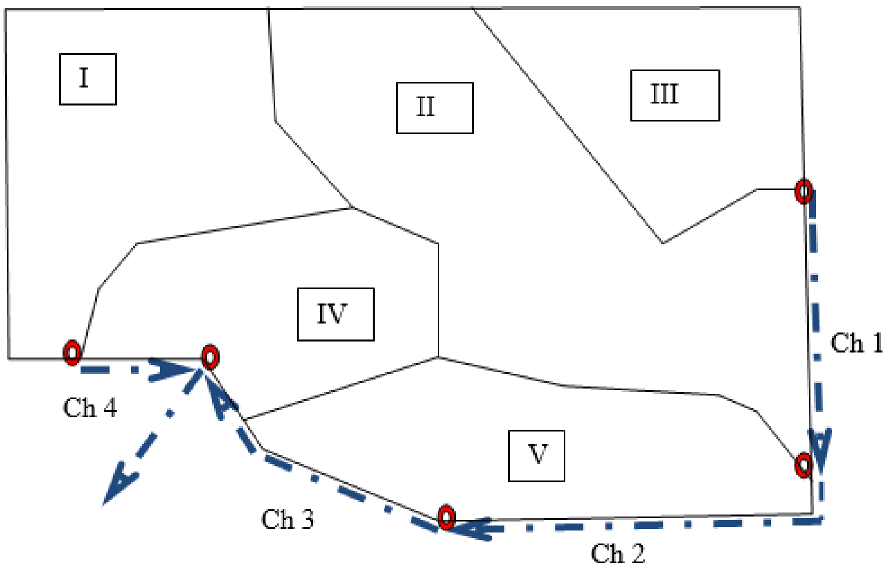

Figure A.1.9: Schematic of existing, predevelopment conditions

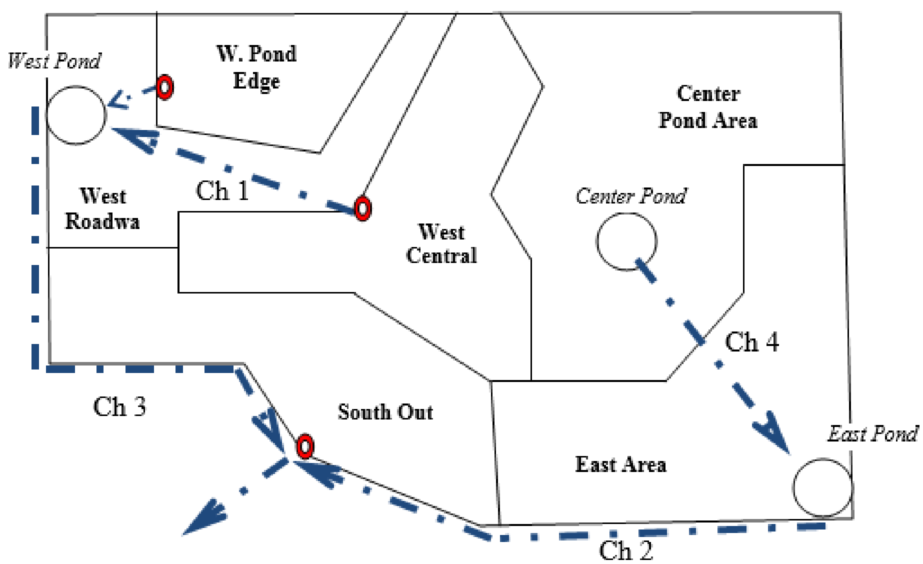

Figure A.1.10: Schematic of proposed, postdevelopment conditions

A.1.6 HEC-HMS Screenshots

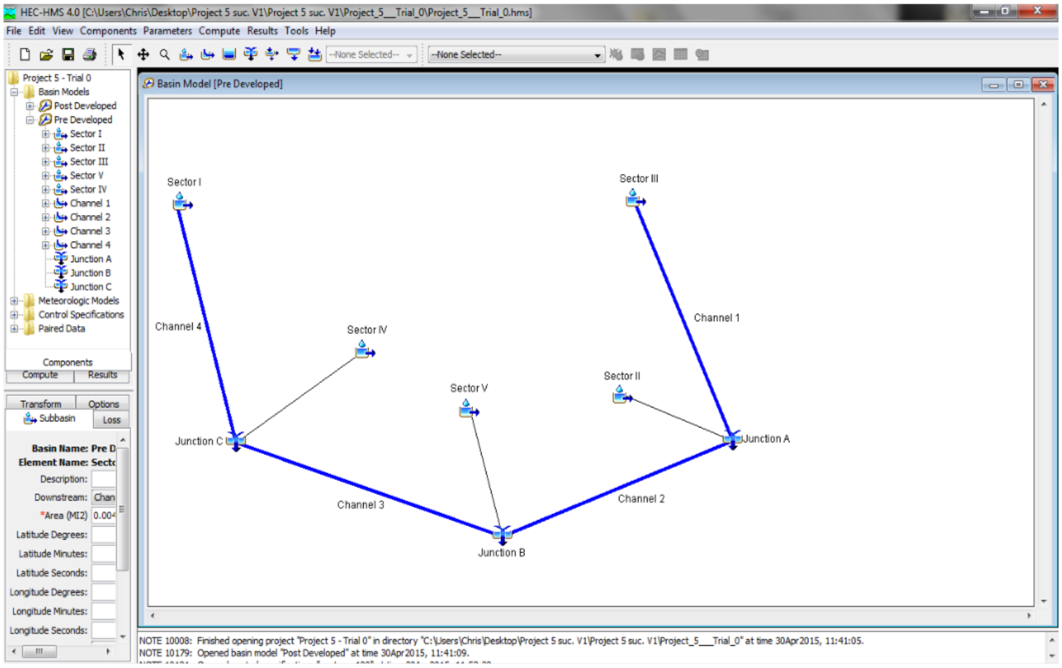

Figure A.1.11: Screen shot of HEC-HMS model of existing, predevelopment conditions

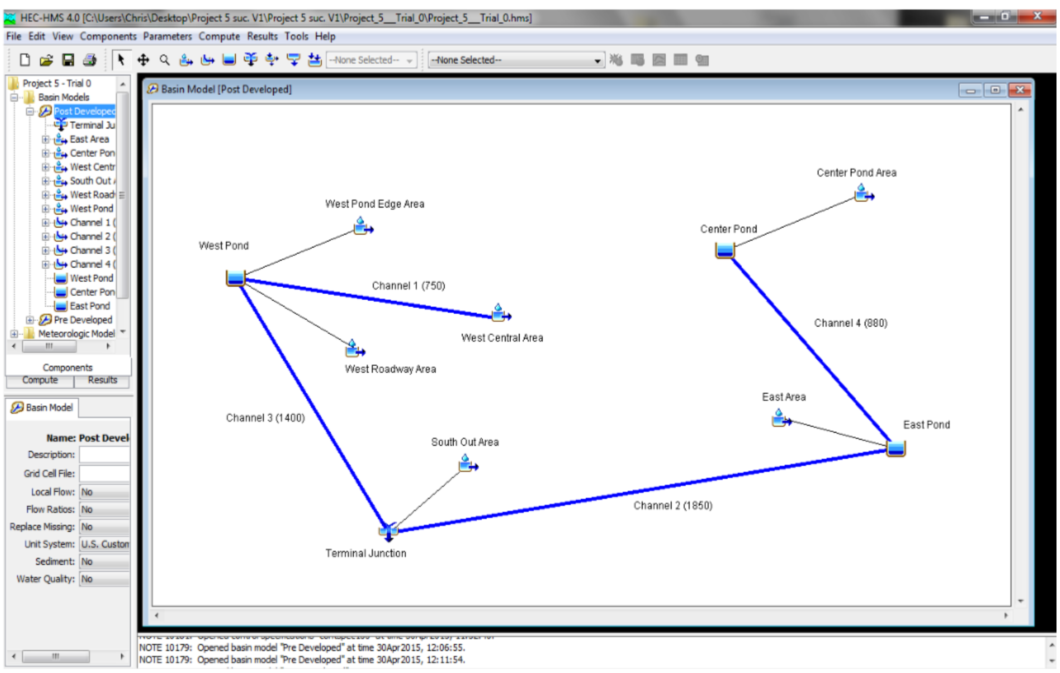

Figure A.1.12: Screen shot of HEC-HMS model of proposed, postdevelopment conditions