Project 2 Report

Detention Basin Design

ENE 421 - Engineering Hydrology
Professor Pokhrel
Due Date: 12/03/2018

Big Dog Consulting
(woof woof)

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Executive Summary

Development of a proposed residential site will result in an increase in impervious area and thus higher runoff generation and peak discharge. The site drains south to a nearby stream which could see adverse effects such as flooding if measures aren't taken to minimize the hydrological effects of the site. This report describes and evaluates the work done by Big Dog Consulting to develop a drainage catchment system and detention pond for the proposed site. The location and length of the catchment system was already provided, so the team only had to determine the pipe diameters that are commercially available. The pipe diameters that were chosen are as follows:

Table 1: Results

Results	
Pipe	Diameter (inch)
A-C	15
B-C	30
C-E	42
D-E	18
E-F	42
Detention Pond	
Outflow pipe size (Inch)	12
Pond Area (acre)	5.6
Pond Height (ft)	1

The detention pond was then created using the Puls method and by abiding by the Ingham County Drain Commissioner's codes and standards. The team concluded that a 5.6 acre, 1 foot deep detention pond would best minimize the effects of the proposed site on the nearby stream. The outflow pipe that was chosen, for the detention basin was a 12 inch diameter pipe. All of the pipes that were chosen are reinforced, precast concrete.

Problem Statement

A proposed residential site is in need of a stormwater drainage and management system. The site consists of seven drainage units, each with differing areas and runoff characteristics. These sites drain into five manholes which will connect to a detention pond. The pond will drain into a stream south of the site through an outlet pipe.

The development of the site will result in increased runoff and thus a higher peak flow when compared to the pre-development stage. The detention pond size and outlet pipe diameter must be designed so that flooding and other adverse effects will be minimized. The post-development peak runoff to the stream must not be greater than the pre-development peak runoff.

Area Map

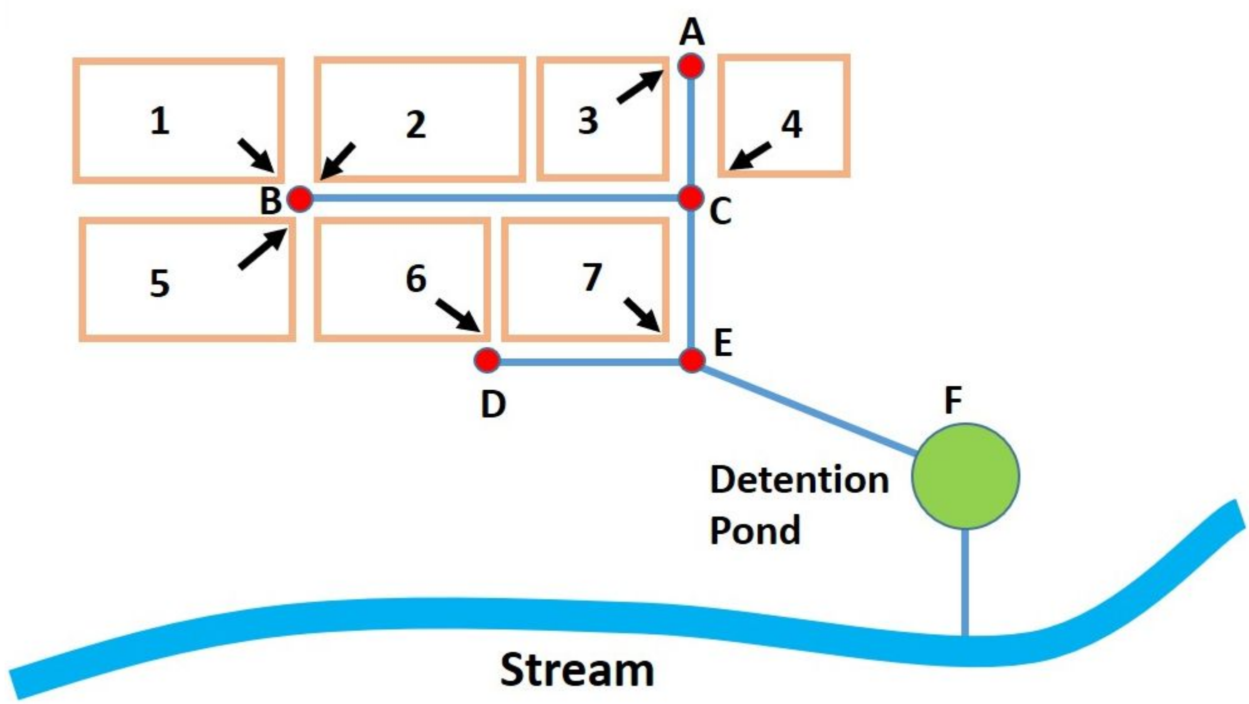

Figure 1: Site Map and Drainage Structure Locations

Other Info

At the site location in Figure 1 and data in Table 2, seven drainage units (1-7) are shown with their different characteristics, which include area, elevation, runoff coefficient, and inlet time. These values were used in Microsoft Excel to calculate the discharge from the watershed using the rational method. Using the discharge from the rational method the team was able to calculate the diameters for each of the pipe segments. Table 3 shows the characteristics of each pipe segment. The equation for the rational method is shown below. The frequency correction factor is given as one.

$$Q = CfCiA$$

Where,
Q= Peak discharge, ft3/s (cfs)
C=Rational method runoff coefficient (dimensionless)
i= Average rainfall intensity (inch/hr)
Cf= Frequency correction factor
A= Area (acres)

Table 2: Catchment Characteristics

Unit #	Area (acres)	Elevation (ft)	Runoff Coefficient	Inlet Time (min)
1	4	115.01	0.9	5
2	3	112.38	0.9	7
3	2	113.67	0.8	10
4	4	113.83	0.7	10
5	5	109.32	0.5	15
6	5	110.07	0.5	15
7	4	107.1	0.5	15

Table 3: Pipeline Characteristics

Pipe Segment	Length (ft)	Ground Slope	Velocity (ft/s)	Time (min)
A-C	520	0.64	5	1.733
B-C	1400	0.81	5	4.667
C-E	550	0.65	5	1.833
D-E	800	0.65	5	2.667
E-F	1100	0.82	5	3.667

The average rainfall intensity is a function of geographic location and design exceedance frequency. To design a hydraulic and hydrologic system, intensity-duration-frequency (IDF) curves are commonly used. For project the group used a 20-year design storm for the system.

$$i = \frac{A}{(tc+B)}$$

Where,
i= Rainfall intensity (in/hr)
tc= time of concentration (min)
A and B= constants that depend on the return period and climate factors

Time of concentration was found for each manhole. The highest tc value is the largest inlet time that goes into each manhole. A table of tc values and their corresponding Intensity values are shown in Table 4. For a 20-year return, the team used the chart IDF curve. Since the frequency of 20-year return was not on the table, the team used interpolation to solve for both constants A and B when x=20. A Figure showing the interpolation of constant B is shown in Figure 2. The constant used for A and B were 209.9 and 27.64.

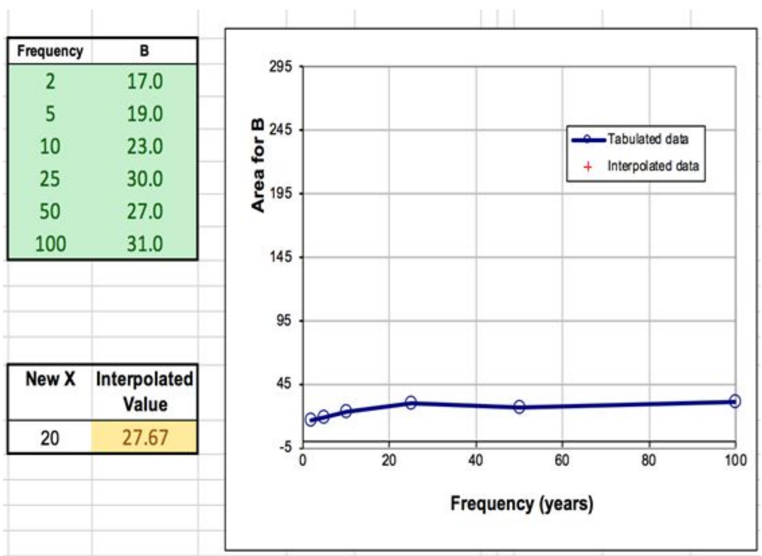

Figure 2: Interpolation of constant B

Table 4: Tc Value for each Manhole

Manhole	Highest tc	tc	i	Peak Q
A	inlet time 3	10.00	5.58	8.92
B	Inlet time 5	15.00	4.92	43.32
C	Inlet 4 + AC+BC	16.40	4.77	62.91
D	inlet time 6	15.00	4.92	12.31
E	Inlet time 7 +CE+DE	19.50	4.45	78.81

After solving for intensity, all the variables were plugged into the rational equation and solved for Peak Q. The resulting Peak Q for each manhole is shown in Table 5.

After solving for peak flow, the team was able to rearrange Manning's equation to solve for the diameter of each pipeline. Manning's equation is shown below:

$$Q = \frac{1.486}{n} A R^{2/3} \sqrt{S}$$

Where,
Q= Discharge in cfs
n= Manning's coefficient
A=Cross-sectional area (ft^2)
R=A/P= Hydraulic Radius (ft) P= Wetted Perimeter (ft)

Table 5: Diameters and Velocity Check

Pipeline	Length (ft)	Ground slope	Velocity (ft/s)	Time (min)	Diameter (ft)	Diameter (in)	Commerically available pipe (in)	Velocity check (Q/A)
A-C	520	0.64	26.36	0.44	0.68	8.19	15	7.27
B-C	1400	0.81	22.85	0.38	1.18	14.17	30	8.83
C-E	550	0.65	13.32	0.22	1.49	17.87	42	7.49
D-E	800	0.65	63.82	1.06	0.77	9.21	18	6.96
E-F	1100	0.82	11.88	0.20	1.51	18.17	42	8.79

4 iterations were performed to solve for the appropriate diameter of each pipe segment, while making sure that the velocities were in-between 3-10 ft/s. The calculated diameters where rounded up to the nearest available commercial size.

Table 6: Commercially Available Pipes

Commercial Pipe Diameters (in)	12	15	18	24	30	36	42	48	60	72

Some other info that was used in the design were some limitations. The first limitation was that any pipe that was selected needed to be commercially available. For reinforced, precast concrete pipes, the available sizes are shown in Table 6 above. The other main limitation that was accounted for was the Ingham County Drain Commissioner requires a max detention/retention discharge rate of 0.15 cfs/acre. The site in question contains 27 acres, meaning that the max discharge rate allowed would be 4.05 cfs.

Method/Procedures

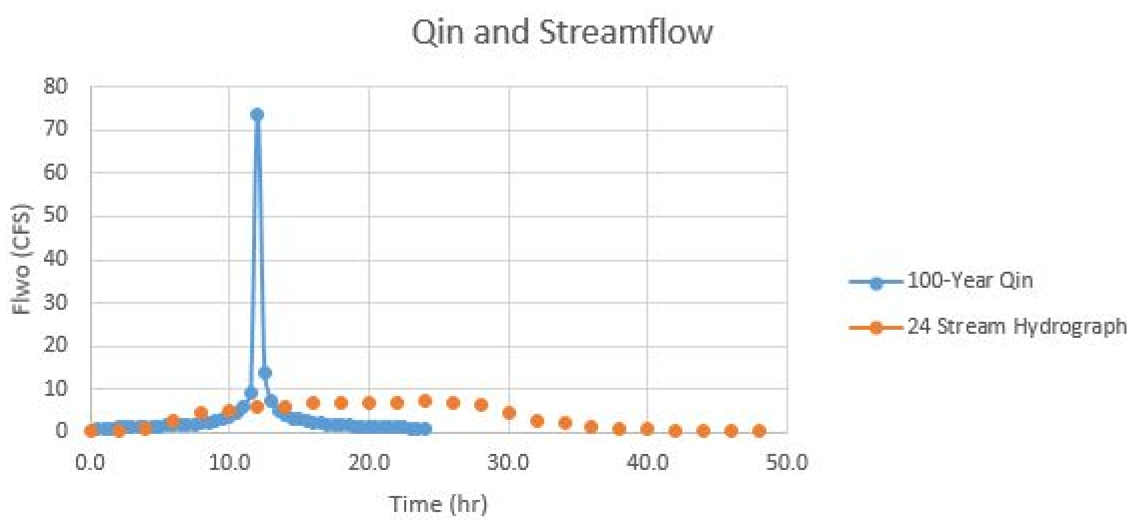

Figure 3: 100 Year Inflow

A detention pond needs to be designed for the site, that can handle a 100 year, 24 hour storm event. This is done by using the data provided from a SCS type 2 rainfall distribution for the East Lansing area. This data is used to get the peak rainfall that needs to be stored in the detention pond. Before a detention pond could be designed, the pre-development conditions needed to be determined and evaluated. Using the 4 hour streamflow data provided and applying the lagging method, a hydrograph was developed to show the existing conditions of the stream. Figure 3 compares the 100 year storm event to the pre-existing streamflow.

The detention pond needed to be designed so that it kept the streamflow either at or below the pre-development peak conditions. The amount of water that needs to be stored in a 100 year event was calculated by using the trapezoidal method on the inflow. This method showed that 163,473.39 cubic feet of water needed to be stored. With this value, an initial pond size of 1 acre was chosen as a starting point.

Using 1 acre as a basin size, multiple heights were selected to get different storage sizes by using the following equation:

$$S = A * h$$

Where S = storage,
A = area of pond, and
h = height of pond.

The outflow (Q_{out}), was also calculated for an initial pipe with a diameter of 1 foot.

$$Q_{out} = CA\sqrt{2gh}$$

Where Q_{out} = the outflow from the pond,
C = is the discharge coefficient = 0.67,
A = the cross-sectional area of the outlet pipe,
g = gravitational acceleration 32.2, and
h = depth of water in the pond.

These two equations were used to start up the Puls method by getting the following equation and applying it to each different pond height:

$$\frac{2S}{\Delta t} + Q_{out}$$

Where Δt was 1800 seconds and
$Q_{out} = O$.

Time Index	Time (min)	Inflow (cfs)	I1+I2	2S1/Δt-O1 (cfs)	2S2/Δt+O2 (cfs)	Outflow (cfs)

Figure 4: Order of Puls Event

The inflow data, from the 100-year rainfall event, was then used to determine the I1 + I2 variable where the previous inflow is added to the next inflow. The Outflow variable in Figure 3, is calculated by using the “Trend” function on the storage and $\frac{2S}{\Delta t} + Q_{out}$ found for each height. The outflow is also dependent on a seperate $\frac{2S}{\Delta t} + Q_{out}$, I1 + I2, and the below equation:

$$\frac{2S}{\Delta t} - Q_{out}$$

This method was run multiple times to create different iterations. Each iteration had different inputs, Those inputs being: the size of the available pipe, the height of the pond, and the area of the pond. These inputs affected the outflow rate and storage of the basin directly. Choosing a proper design also required that all the outflow values for the 24-hr period be below the pre-development conditions and below 4.05 cfs according to the Ingham County Drain Commissioner.

Results

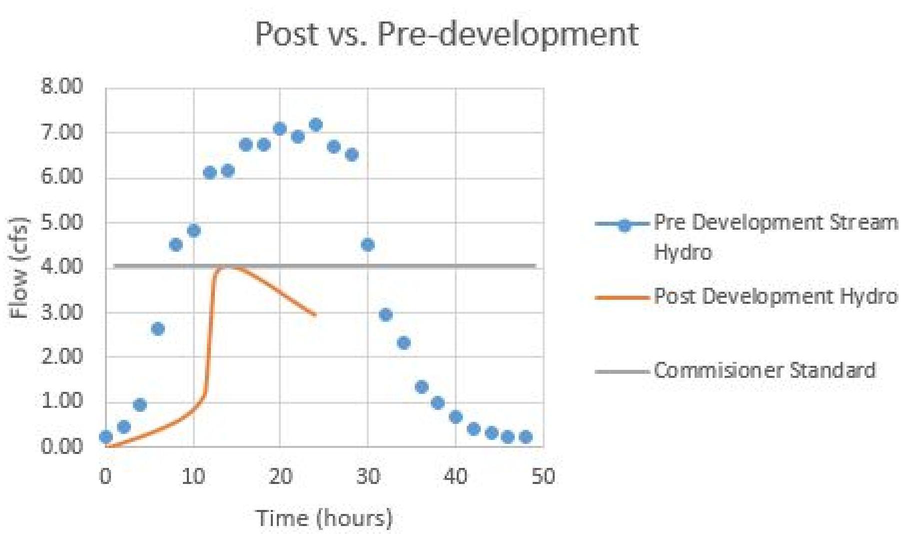

Figure 5: Post vs. Pre-development

Figure 5 shows an iteration of post development versus pre-development. These results show that the values that were chosen, both fit underneath the pre-development curve, and also fit under the 4.05 cfs limit. Other iterations were ran and showed varying results. This was the design that was ultimately chosen due to the outflow remaining under the 4.05 cfs requirement. Figure 6 below also shows the same iteration against the 24 hour storm event.

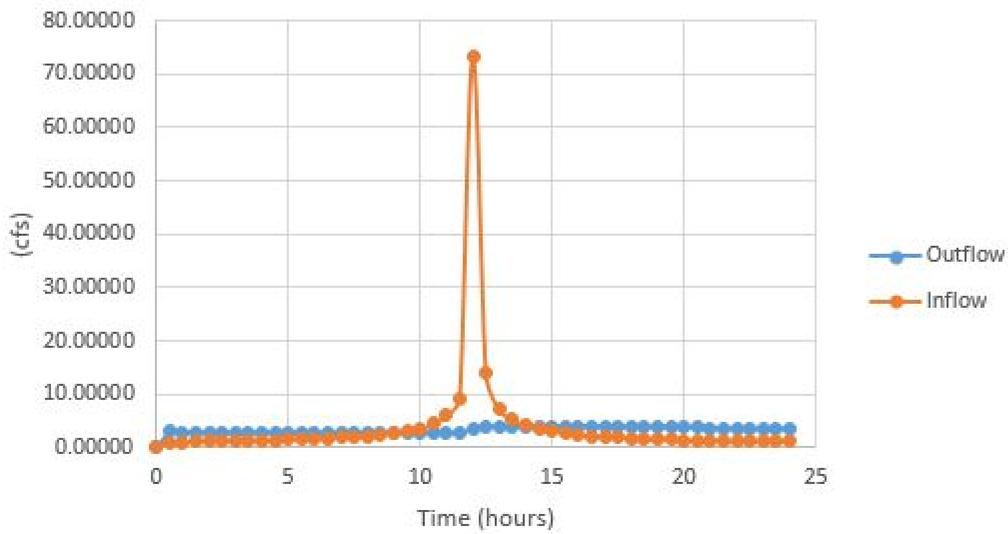

Figure 6: Post Development of Outflow and Inflow

Table 7: Storage Required

Area for Qin	Area for Q out
7132.1796	1571.4831
9393.6024	1776.7919
13742.4924	2077.2116
74452.9968	3366.5043
78627.9312	5611.6604
18961.1604	6936.6170
11133.1584	7185.4682
8349.8688	7263.8940
Total	Total
221793.3900	35789.6305
Storage needed	186003.7595

Table 7 shows the storage needed by the basin to hold the storm event. This is calculated as the area under the inflow curve and above the outflow curve in Figure 6.

Economic Estimation

The cost of the detention pond had to be estimated. For the detention pond, cost data was provided by the EPA. The smallest EPA unit cost will be used due to the detention pond being larger than 1 acre in area and having a shallow depth. . The total detention pond cost was found by multiplying the pond volume by the construction cost per cubic foot.

The cost of the pipes for the catchment system and the detention pond outlet were estimated. The diameters required for the catchment system pipes were found using Manning's equation and were then rounded up to the nearest commercially available pipe size. The pipe unit costs were found using a construction materials cost database. The total pipe costs were found by multiplying the pipe length by the unit cost. A length for the pipe leaving the detention pond was not provided so the team estimated the length to be 500 ft by scaling the diagram provided.

The total cost for the catchment system and outlet pipe came out to $286,271.60. The cost to construction the detention pond was estimated at $121,968.00. The combined cost came out to $408,239.60 excluding construction costs. All cost estimates are listed in Tables 8 & 9.

Table 8: Pipe Cost Estimate

Pipe	**Length (ft)**	**Pipe Unit Cost ($/ft)**	**Pipe Total cost**
A-C	520	20.93	$10,884.00
B-C	1400	57.99	$81,186.00
C-E	550	99.76	$54,868.00
D-E	800	25.71	$20,568.00
E-F	1100	99.76	$109,736.00
Outlet Pipe	500	18.06	$9,030.00
Σ			$286,272.00

Table 9: Detention Pond Cost Estimate

	Vol. (ft^3)	**Construction Cost ($/ft^3)**	**Pond Total Cost**
Detention Pond	243,936	0.50	$121,968.00

Discussion

The location, length, and slope of the pipes to be constructed were given and the diameters required of the catchment system's pipes were calculated using Manning's Equation. The Rational Method was used to solve for the peak flow that each pipe would experience. The diameter required was rounded up to the nearest commercially available size. The pipe diameters chosen are listed below in Table 10.

Table 10: Catchment System Pipe Size Selected

Results	
Pipe	Diameter (inch)
A-C	15
B-C	30
C-E	42
D-E	18
E-F	42

The iterations where the pipe was sized greater than 12 inches had a flow rate over the allowable 4.05 cfs. A 6 inch pipe was found to have the ideal numbers for outflow rate and acreage of the pond. These results for the 6 inch pipe were to have a pond area of 1 acre and a depth of 5 feet. A six inch pipe with a 1 acre pond was not chosen because the pipe size is not commercially available, therefore it would be more expensive to manufacture. The smallest commercially available pipe size is 12 inches. Therefore, a 12 inch pipe was used to run iterations with different pond sizes. Ultimately, a pond size of 5.6 acres in area and 1 foot in depth was chosen. Refer to Table 11 for results. Sod will be placed within 5 days after excavation to keep the pond from eroding. The cost for all the piping came out to be $286,272.00 while the detention pond is $121,968.00. The detention pond is rather inexpensive because it is very shallow, therefore the excavation will be minimal. However, the pond will require maintenance annually to avoid pipe clogging and erosion.

Table 11: Pipe and Pond Size Selected

Pipe Selected:	12 Inch Diameter
Pond Area:	5.6 Acres
Pond Depth:	1 Foot

Conclusions/Recommendations

The proposed residential development site will result in an increased impervious area leading to higher runoff generation and peak flow into the nearby stream. If measures are not taken to accommodate these new hydrological behaviors the stream could see flooding and other adverse effects as a byproduct of the new development. The team was tasked with finding a solution to minimize the effect that the site will have on the stream to the south. The catchment system and outlet pipe locations were already given, so only the pipe diameters had to be calculated. After calculation the team recommends a detention pond size of 5.6 acres in area and 1 foot in depth.

Appendix

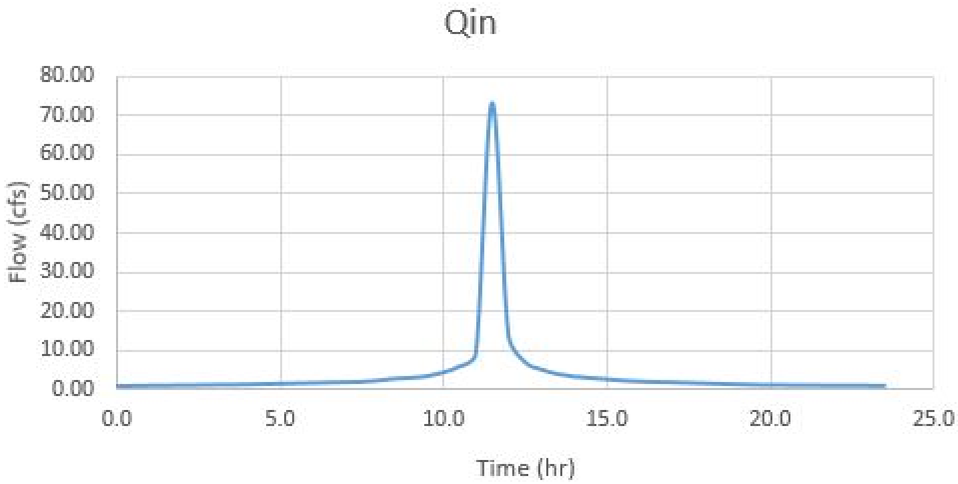

Figure 7: 24 Hour Rain Event

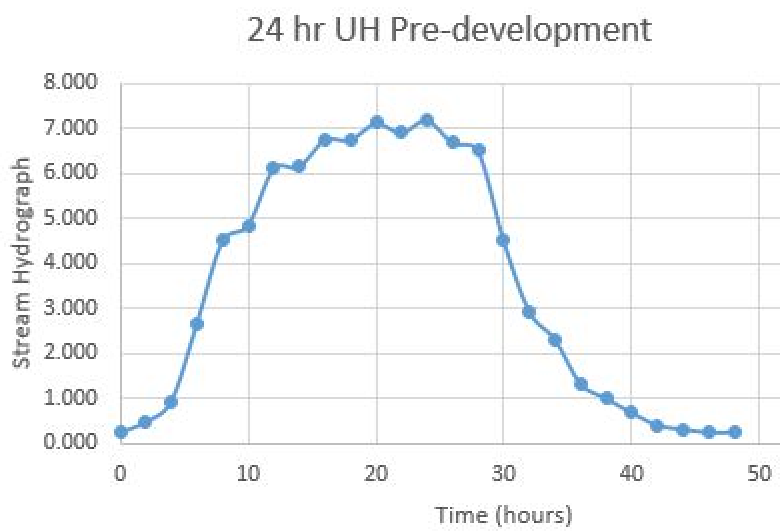

Figure 8: Pre-development Streamflow

Constants A &B for IDF curves for different regions in the US.

Figure 2.7 Map of similar rainfall characteristics (from Steel and McGhee, 1979).

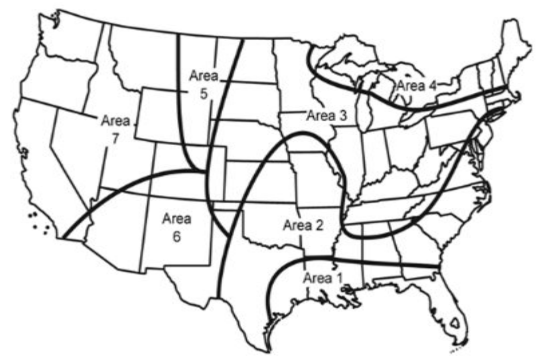

Table 2.8 Intensity–Duration Constants for Various Regions[a]

Frequency (years)		Area 1	Area 2	Area 3	Area 4	Area 5	Area 6	Area 7
2	A =	206	139.75	102	70	70	68	31.9
	B =	30	21	17	13	16	14	11
5	A =	246.9	190.2	131.1	96.9	81.1	74.8	48
	B =	29	25	19	16	13	12	12
10	A =	300	229.9	170	111	111	122	59.8
	B =	36	29	23	16	17	23	13
25	A =	326.8	259.8	229.9	170	129.9	155.1	66.9
	B =	33	32	30	27	17	26	10
50	A =	315	350	250	187	187	159.8	65
	B =	28	38	27	24	25	21	8
100	A =	366.9	374.8	290	220	240.2	209.8	77.2
	B =	33	36	31	28	29	26	10

[a] Conversion of units by the author.
Source: Steel and McGhee (1979).

Figure 9: IDF Curves

Table 12: Puls Method Constants

h (ft)	Qout	S(ft^3)	(2S/Δt)+O (cfs)
0	0.00	0	0.00
1	4.22	243936	275.26
2	5.97	487872	548.05
3	7.31	731808	820.43
4	8.45	975744	1092.61
5	9.44	1219680	1364.64
6	10.34	1463616	1636.58
7	11.17	1707552	1908.45
8	11.94	1951488	2180.26
9	12.67	2195424	2452.03
10	13.35	2439360	2723.75
11	14.01	2683296	2995.45
12	14.63	2927232	3267.11
13	15.23	3171168	3538.75
14	15.80	3415104	3810.36
15	16.36	3659040	4081.96
16	16.89	3902976	4353.53
17	17.41	4146912	4625.09
18	17.92	4390848	4896.64
19	18.41	4634784	5168.17
20	18.89	4878720	5439.69

Table 13: Puls Method Variables

Time Index	Time (min)	Inflow (cfs)	I1+I2	2S1/Δt-O1 (cfs)	2S2/Δt+O2 (cfs)	Outflow (cfs)
1	0	0	0	0.00000	0.00000	0.00000
2	30	0.99	0.99	0.96	0.99	0.02
3	60	1.04	2.03	2.90	2.99	0.05
4	90	1.09	2.13	4.87	5.02	0.08
5	120	1.13	2.22	6.88	7.09	0.11
6	150	1.18	2.32	8.91	9.20	0.14
7	180	1.23	2.42	10.98	11.33	0.17
8	210	1.28	2.51	13.08	13.49	0.21
9	240	1.33	2.61	15.21	15.69	0.24
10	270	1.40	2.73	17.39	17.94	0.28
11	300	1.50	2.90	19.66	20.29	0.31
12	330	1.59	3.09	22.06	22.76	0.35
13	360	1.69	3.29	24.57	25.34	0.39
14	390	1.79	3.48	27.18	28.05	0.43
15	420	1.88	3.67	29.91	30.86	0.47
16	450	1.98	3.87	32.74	33.78	0.52
17	480	2.08	4.06	35.67	36.80	0.56
18	510	2.37	4.45	38.88	40.12	0.62
19	540	2.85	5.22	42.75	44.10	0.68
20	570	3.09	5.94	47.20	48.69	0.75
21	600	3.48	6.57	52.12	53.77	0.82
22	630	4.45	7.92	58.20	60.05	0.92
23	660	5.99	10.44	66.53	68.64	1.05
24	690	9.28	15.27	79.29	81.80	1.25
25	720	73.45	82.73	157.05	162.02	2.49
26	750	13.92	87.36	236.91	244.41	3.75
27	780	7.15	21.07	250.07	257.98	3.96
28	810	5.22	12.37	254.38	262.44	4.03
29	840	4.06	9.28	255.57	263.66	4.04
30	870	3.41	7.47	254.97	263.04	4.04
31	900	3.07	6.48	253.42	261.44	4.01
32	930	2.73	5.80	251.27	259.22	3.98
33	960	2.39	5.12	248.52	256.39	3.93
34	990	2.16	4.55	245.31	253.08	3.88
35	1020	2.04	4.20	241.86	249.51	3.83
36	1050	1.92	3.96	238.28	245.82	3.77
37	1080	1.80	3.72	234.57	242.00	3.71
38	1110	1.68	3.48	230.75	238.05	3.65
39	1140	1.56	3.24	226.81	233.99	3.59
40	1170	1.44	3.00	222.75	229.80	3.53
41	1200	1.32	2.75	218.59	225.51	3.46
42	1230	1.24	2.56	214.36	221.15	3.39
43	1260	1.22	2.46	210.17	216.83	3.33
44	1290	1.20	2.42	206.07	212.59	3.26
45	1320	1.17	2.37	202.04	208.44	3.20
46	1350	1.15	2.32	198.09	204.36	3.14
47	1380	1.12	2.27	194.21	200.36	3.07
48	1410	1.10	2.22	190.41	196.44	3.01
49	1440	1.07	2.17	186.67	192.58	2.95

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