An Analysis of Flow Attenuation Provided by Stream-Buffer Ordinances in Johnson County, Kansas

by

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Abstract

Many communities in the United States have enacted ordinances requiring that areas surrounding natural channels be preserved in a natural state. These areas are commonly referred to as "stream-buffers". One of the goals of the stream-buffer is to preserve dense overbank vegetation. This has the effect of attenuating peak flows during flood events which inundate the channel overbanks. The goal of this study is to use state-of-the-practice hydrologic and hydraulic models to estimate peak-flow attenuation provided by stream-buffers using vertical variation in Manning's n values. Accounting for vertical variation in Manning's n values in overbanks allows for simulation of the roughness of the overbank provided by zones of vegetation. Typical zones include dense grasses and undergrowth at low overbank depths as well as heavily treed zones at higher depths.

An existing 2.6 miles stream reach was evaluated for this study. Hydrologic modeling was completed using HEC-HMS and hydraulic modeling was completed using HEC-RAS. Existing models completed for the Blue River Watershed Study in Johnson County, Kansas, were modified for use in this study. A maximum peak-flow attenuation of 20% was observed for the 2-year and 50-year events over 3,110 feet. The highest maximum peak-flow reductions were observed for events ranging from the 2-year and 100-year events, and a smaller maximum reduction was observed for the 500-year event.

Another goal of this study was to compare the results to stream-buffer ordinances in Johnson County to evaluate if they provide the maximum attenuation of peak flows possible at the case study site. The results showed that maximum attenuation is achieved by the ordinances for events ranging from the 2- to 10-year events.

Acknowledgements

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Most of all I would like to thank my wife Misha, who has endured many an hour of her husband staring at a computer screen instead of working on necessary home projects. I hope she knows that I love her, appreciate her patience, and that I am going to start on painting the house as soon as this paper is accepted.

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Section 1. Introduction

1.1 Hydraulic Geometry of a Natural Stream

The cross section of a stream in its natural state is generally composed of two sections, as shown in Figure 1:

- A main channel which carries baseflow and the majority of rainfall events, and which is considered to be bounded by the stream banks, and
- The overbanks, which are typically located on both sides of the main channel and begin at the stream banks and extend outward. The overbanks carry flows when inflow to the stream is greater than the conveyance capacity of the main channel.

According to Manning's equation, shown in Equation 1, flowrate in an open channel is a function of the area of flow to the 5/3 power and wetted perimeter to the negative 2/3 power.

$$Q = \frac{1.49}{n}\frac{A^{5/3}}{P^{2/3}}\sqrt{s_f} \tag{1}$$

where:

Q = Flowrate (cubic feet per second)

n = Manning's n roughness coefficient

A = Area of Flow (square feet)

P = Wetted Perimeter (feet)

s_f = Friction Slope (feet per feet)

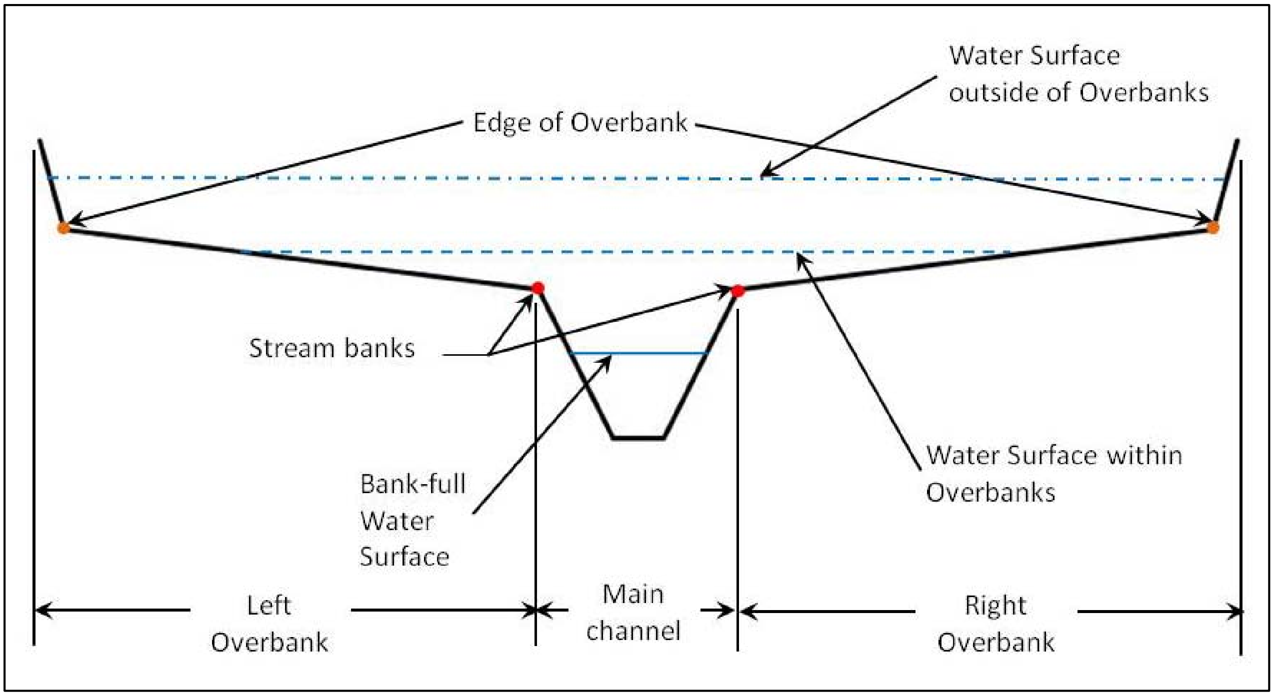

Figure 1. Typical Cross Section of a Natural Stream

Incremental increases in water-surface elevation within the main channel result in increases in the area of flow with relatively small corresponding increases in wetted perimeter. This is due to the steep side slopes of the main channel, as shown in Figure 2. In Manning's equation, area is raised to the 5/3 power and corresponds to an increase in flow, while wetted perimeter is raised to the negative 2/3 power and has the effect of decreasing flow. Because changes in its area are large with corresponding small changes in wetted perimeter, and because the effect of area is raised to a higher exponent than the wetted perimeter in Manning's equation, the main channel is able to provide a large increase in conveyance capacity per unit cross sectional area for an incremental increase in depth.

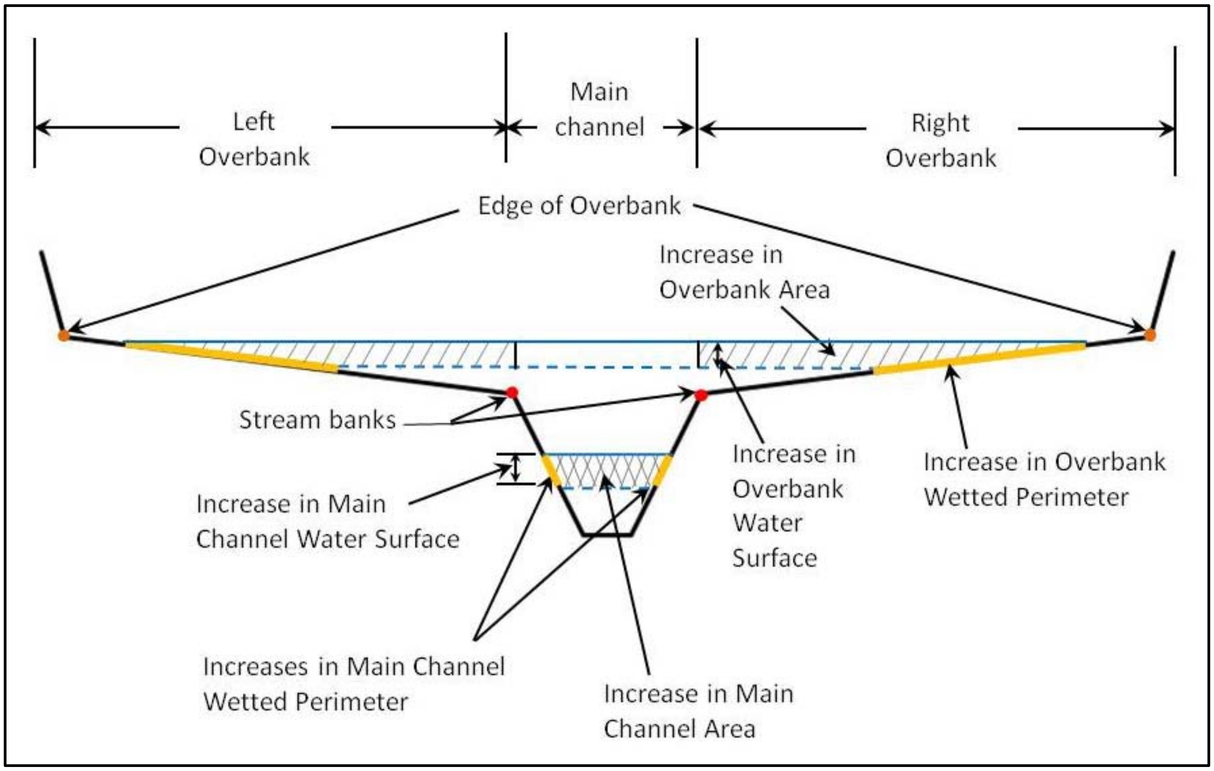

Figure 2. Changes in Water Surface in a Cross Section of a Natural Stream

The hydraulics of the overbanks functions differently than the main channel. Due to the gradual slopes of the overbanks, incremental increases in depth result in increases in area accompanied by relatively large increases in wetted perimeter, as shown in Figure 2. The wetted perimeter, therefore, has a larger effect on the flow in the overbank than it does in the main channel. Because of this, conveyance capacity per unit cross sectional area is smaller in the overbank than it is in the main channel.

1.2 Effect of Manning's n in an Open Channel with Steady Flow

The value of Manning's n, which represents resistance to flow due to friction, also plays an important role in determining flow in Manning's equation. To illustrate, a scenario is assumed in Table 1 wherein an open channel with a slope of 0.005 feet per feet is assumed. In the table, Manning's n values of 0.01 and 0.02 are assumed, and the channel geometries are assumed to be identical in every other respect. In the three columns shown, values for area to the 5/3 power divided by wetted perimeter to the 2/3 power of 1, 2, and 3 feet are assumed. The term area to the 5/3 power divided by wetted perimeter to the 2/3 power is represented by the Greek letter λ for the remainder of the paper, as shown in Equation 2. Using these values as inputs to Manning's equation, flows are calculated and the results populate the table.

$$\lambda = \frac{A^{5/3}}{P^{2/3}} \tag{2}$$

where:

A = Area of Flow (square feet)

P = Wetted Perimeter (feet)

Table 1. Effect of Manning's n Value on Flow

Manning's n Value	**Flowrate (cubic feet per second) for assumed λ (feet to the 8/3 power)**		
	λ = 1	λ = 2	λ = 3
0.01	10.5	16.7	21.9
0.02	5.3	8.4	11.0

As can be seen in Table 1, when the n value is 0.01 and λ is 1 $ft^{8/3}$, a flow of 10.5 cubic feet per second (cfs) is calculated. However, for an n value of 0.02 and λ of 1 $ft^{8/3}$, the Manning's flow is halved to 5.3 cfs. In order to achieve a flowrate of 10.5 cfs for an n value of 0.02, a λ between 2 $ft^{8/3}$ and 3 $ft^{8/3}$ is required. Additional calculations were completed outside of Table 1 which found that the λ required to achieve a 10.5 cfs flow with an n value of 0.02 is 2.83 $ft^{8/3}$.

As previously discussed, an increase in flow area is always accompanied by an increase in wetted perimeter. The difference in flow area between a λ of 1 $ft^{8/3}$ and a λ of 2.83 $ft^{8/3}$ would vary depending on if flow is in the main channel or the overbanks. Because increases in wetted perimeter are relatively small in the main channel for increases in flow area, the difference in flow area for a λ of 1 $ft^{8/3}$ and a λ of 2.83 $ft^{8/3}$ would be small. In the overbank, a larger difference in flow area would be observed as increases in wetted perimeter are larger when flow area increases. Along the length of channel, such an increase would also cause an increase in the volume of stormwater within the overbanks.

To summarize, for an open channel with a certain steady-state overbank flow an Manning's n there is a corresponding λ which is determined by the Manning's n value as well as the channel hydraulic geometry. As overbank Manning's n increases, so to does λ, and along the length of the channel, a large λ corresponds to a large volume of water being stored within the overbank.

1.3 Continuity in an Open Channel

The continuity equation for unsteady, open channel flow can be written as shown in Equation 3.

$$\bar{I} = \frac{\Delta S}{\Delta t} + \bar{O} \tag{3}$$

where:

$\bar{I}$ = Average Inflow over a Time Step (cubic feet per second)

$\bar{O}$ = Average Outflow over a Time Step (cubic feet per second)

ΔS = Change in Storage in Segment of Open Channel (cubic feet)

Δt = Time Step (seconds)

For the scenario presented in Table 1, where the flowrate for two open channels similar in every respect but Manning's n was found, it was shown that the volume stored within open channel overbanks increases with increases in overbank Manning's n. This corresponds to an increase in the ΔS term in Equation 3. For an average inflow, the increase in the ΔS term must correspond to a decrease in the average outflow term for the equation to balance. When outflows are smaller than inflows over a time step, this represents attenuation of flow.

This hydraulic scenario is equivalent to a detention basin. A detention basin receives inflows, stores the inflows within itself, and attenuates these inflows by forcing outflows to be smaller by means of an engineered outlet structure. In the case of open channel overbank flow, outflows are made to be smaller than inflows by high roughness which resists flow. This roughness resistance is represented by high Manning's n values.

This paper utilizes hydrologic modeling software to simulate unsteady flow in a natural channel as a series of detention basins, or reservoir, to quantify overbank attenuation. This hydrologic routing method is known as Modified Puls, and it uses the continuity equation as presented in Equation 3 for each reservoir. In addition, hydraulic modeling software which uses continuity and the energy equation for steady-state open channel flow is used to determine volume stored within the channel for a series of flows. Use of this hydraulic model allows the energy equation to determine changes in hydraulic geometry and velocity caused by increases in Manning's n.

Manning's n values can vary vertically, especially in overbanks where vegetation is dense. If this vegetation includes short, dense undergrowth, surrounded by a dense forest, then Manning's n values may be as high as 0.2 near the ground (Chow 1959). At higher elevations, this n value is expected to decrease, as higher water depths are less affected by friction. This would have the effect of reducing the ability of the overbank to store and attenuate floodwaters.

Research has indicated that when flow area, and thus flood depth, increases past a certain point in the overbank, the overbank conveyance per unit area becomes larger and the ability of the overbank to attenuate inflows is diminished. Bhomik and Stall (1979) state that when flood depths are large in the overbanks, the entire cross section begins to function in the manner of the main channel. Peak flow attenuation due to overbank flow, therefore, occurs for a range of depths.

1.4 Literature Review

Multiple studies have been undertaken to quantify flood attenuation effects of overbank storage in natural channels. These studies have generally found that attenuation of flow hydrographs in open channels caused by overbank storage can be significant depending on channel geometry. These studies have taken the form of either experimental studies or synthetic, computer simulation-based investigations.

Such empirical studies have limited general applicability, as flood wave attenuation is significantly influenced by several factors. These factors include hydrograph properties, channel roughness, longitudinal channel slope, channel width depth, floodplain (overbank) roughness, and floodplain width. Studies such as the one conducted by Wolff and Burges (1994) have evaluated the impact of these factors and found that they all influenced the magnitude of peak flows.

It is much easier and more economical to build a computer model when evaluating the overbank flow attenuation of an existing stream than to conduct an experimental study, as experimental studies require construction of test flumes. Computer-based analyses of peak-flow attenuation typically have taken the form of either:

- Hypothetical scenarios, in which significant factors influencing peak-flow attenuation are varied and general principles are derived from the results, or
- Site-specific studies, in which an existing open channel is evaluated and results specific to the site reported.

Table 2 summarizes a series of the findings of previous studies.

Table 2. Summary of Previous Studies

Title	Author(s)	Year Published	Methodology	Results
Experimental Studies				
Experiments on Flood-Wave Propagation in Compound Channel	Chan-Ji Lai; Chang-Ling Liu; Yeu-Zen Lin	2000	A 115-foot long flume with a compound trapezoidal cross section was constructed. The flume contained a 37-foot long sinuous channel, and flow was loaded at the top of the flume according to specified hydrographs.	Peak flows were attenuated along the channel exponentially.
Computer Model Studies - Hypothetical Scenarios				
An Analysis of the influence of Riparian Vegetation on the Propagation of Flood Waves	B.G. Anderson; I.D. Rutherford; A.W. Wester	2005	A computer simulation was completed using the FLDWAV model analyzing a hypothetical channel. Overbank Manning's n values were varied as a function of cross-sectional geometry and stage. The modeled reach length was 50 kilometers.	Large reductions in peak flow were observed, with a 31% decrease observed for the 50-year event.
An Analysis of the Influence of River Channel Properties on Flood Frequency	C. Gary Wolff; Stephen J. Burges	1994	The computer software DAMBRK was used to analyze a hypothetical channel with two runs completed in which overbank roughness was varied. One run used an overbank Manning's n of 0.035 and the other used 0.120. The modeled reach was 50 miles in length.	Large reductions in peak flow attenuation were observed. Reductions of 20% were observed for the 2- and 10-year events; 10% for the 50-year event, and 14% for the 100-year event.
Effect of Roughened Strips on Flood Propagation on Representative Virtual Cases and Validation	Ahmad-Reza Ghavasieh; Christine Poulard; André Paquier	2005	A hypothetical, 20 kilometer channel was analyzed to find the effect of discrete overbank areas which had high Manning's n values. Two separate computer models were developed for this study, including a 1-dimensional model analysis using the software Mage5 and a 2-dimensional model analysis using the software RUBAR20.	Peak flows were observed to be reduced between 0.1% and 3.65%.

Table 2 (continued). Summary of Previous Studies

Title	Author(s)	Year Published	Methodology	Results
Computer Model Studies - Site Specific Studies				
Hydrological and Statistical Characteristics of Extreme Floods	Timothy Diehl	1990	A computer model of Black Earth Creek in Wisconsin, which is approximately 22 miles long, was developed. A run using a Manning's n value of 0.053 was compared to a run which used 0.1 as the overbank Manning's n value.	Decreases as high as 27% were observed for flood events with recurrence intervals ranging between 4-year and 50-year.
A Watershed Modeling Analysis of Fluvial Geomorphic Influences on Flood Peak Attenuation	Christopher J. Woltemade; Kenneth W. Potter	1994	A computer model of Grant River watershed was developed using the MIKE11 software. Hydraulic modeling of approximately 53 kilometers of Grant River was completed. A series of overbank Manning's n values were used which ranged from 0.03 to 0.057.	Reductions in peak flow were observed as high as 26%.

One experimental study completed in 2000 was “Experiments on Flood-Wave Propagation in Compound Channel” by Lai, et al. For the study a 115 foot-long flume with a compound trapezoidal cross section was constructed. The flume contained a 37-foot long sinuous section to simulate a natural channel, and flow was loaded at the top of the flume according to several specified hydrographs. The study found that peak flows were attenuated exponentially along the experimental flume.

Studies evaluating hypothetical scenarios assumed values for factors affecting peak-flow attenuation. Many such studies evaluated reaches with very large drainage areas (Sholtes and Doyle 2010), leaving a gap in the research for small reaches located in the upper portions of a watershed. The case study analyzed for this paper is a small tributary of Wolf Creek in Johnson County, Kansas. it’s the drainage area for this reach ranges from 0.2 square miles at its start to 2.8 square miles at its confluence with the main channel of Wolf Creek. As a result, the study of this reach fills a gap in knowledge for overbank attenuation in small drainage areas.

Limited research has been completed to relate the effect of vertical variation in Manning’s n overbank values and peak-flow attenuation caused by overbank storage; however, one study, completed by Anderson, et.al. (2005) and described in Table 2, did take into account vertical variation of Manning’s n overbank values. Modeling completed for this paper attempts to duplicate the results found by Anderson, et.al. (2005) using an independent approach.

In addition, this study’s approach is intended to be readily reproducible by a practicing engineer familiar with state-of-the-practice computer modeling. The model used by Anderson, et.al. (2005) was the FLDWAV software, developed by the United States National Weather Service. This model solves the fully dynamic formulation of the St. Venant equations and runs in the

Microsoft DOS environment. Because of this, the FLDWAV model is complex and not generally used by practicing engineers using the Microsoft Windows operating system.

Stream-buffers are generally implemented by practicing engineers who are familiar with the United States Army Corps of Engineers' (USACE) models created by its Hydrologic Engineering Center (HEC), including the Hydrologic Modeling System (HEC-HMS) and the River Analysis System (HEC-RAS). A methodology for using these models to account for vertical variation in Manning's n would allow practicing engineers to complete a detailed analysis of stream-buffers with well-known tools. Desktop modeling of individual reaches with a stream-buffer allows for site-specific conclusions and recommendations to be made.

Section 2. Methodology

2.1 Background

A portion of the Blue River Watershed located in Johnson County, Kansas, was selected to model for this study. The Blue River Watershed Study (BRWS) was completed in 2001, and included hydrologic and hydraulic modeling (CDM 2001). These models were used as a baseline for this study.

The reach selected for this study, shown in Figure 4, is a first-order stream according to the Strahler stream order and is located in the headwaters of the Wolf Creek subwatershed. The drainage area of the study reach ranges from 0.2 square miles at its start to 2.8 square miles at its confluence with the main channel of Wolf Creek. The study reach is approximately 2.6 miles long measured along the thalweg of the channel. The study reach and its drainage area were modeled using the same raw datasets as were used in the 2001 watershed study. In 2001, the drainage area was mostly composed of farmland, with some pasture and rural residential development.

Two scenarios were modeled for this study. The first scenario modeled the study reach without a stream-buffer, and the second model run included a stream-buffer. The stream-buffer was assumed to extend the entire width of the overbanks. This allowed the modeling to find the maximum possible attenuation. After modeling was completed, the results were examined to see if these maximum attenuations could be achieved by existing stream-buffer ordinances in Johnson County, Kansas.

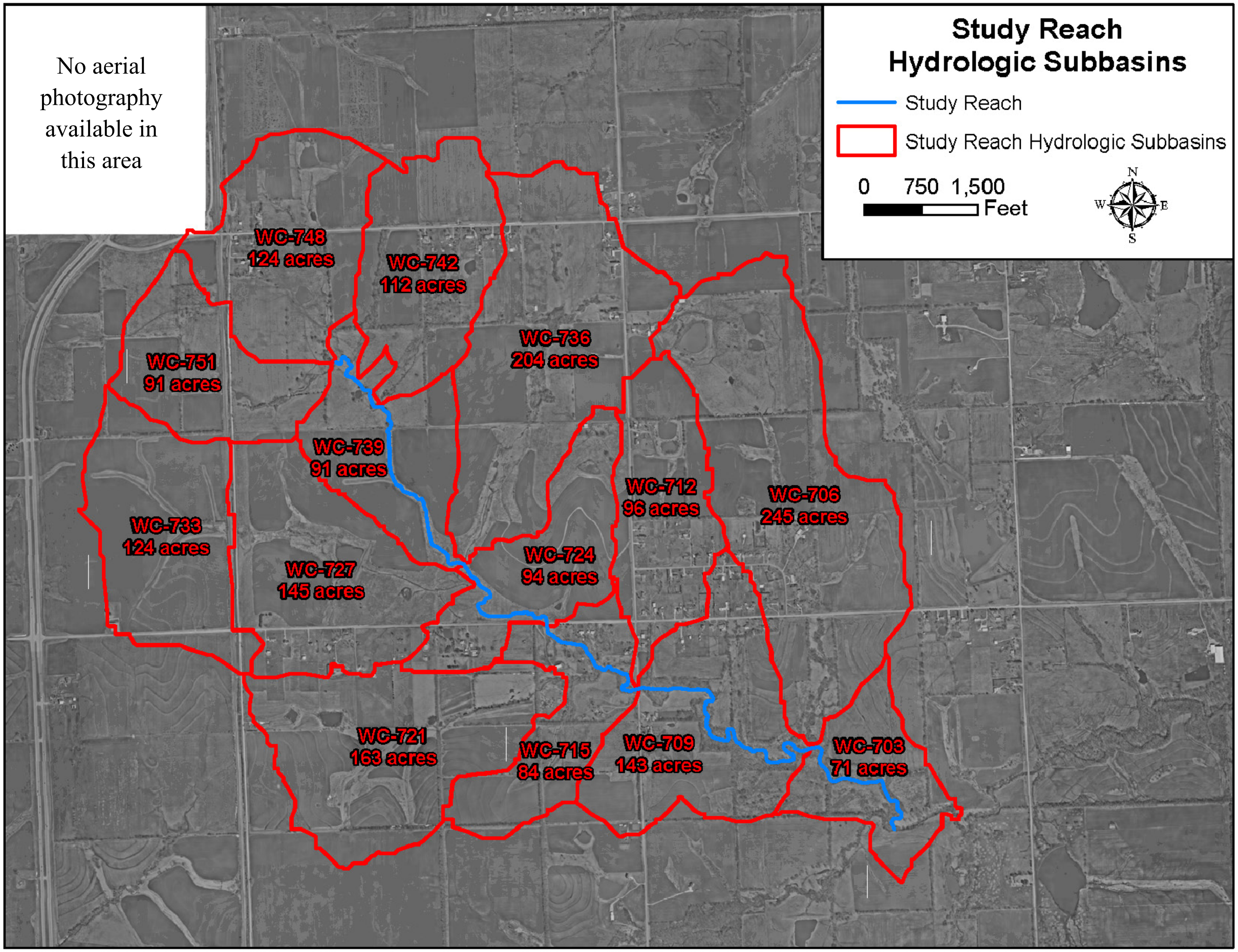

Figure 3. Study Reach and Drainage Area Hydrologic Subbasins

2.2 Hydrologic Modeling

Hydrologic modeling of the drainage area was completed using the HEC-HMS version 3.5. The hydrologic models created for the BRWS were used as a baseline for this study. The BRWS models were completed using the HEC-1 modeling software, which is the predecessor of HEC-HMS. For the purposes of this study, the HEC-1 models were imported to HEC-HMS. The model results were compared, and the peak-flow results for the HEC-1 and HEC-HMS model did not differ significantly, as shown in Table 3.

The BRWS employed the United States Department of Agriculture's Natural Resources Conservation Service Curve Number method to model losses and the SCS unit hydrograph method to model subbasin transform, as outlined in Technical Release 55 (NRCS, 1986). The curve numbers in the BRWS were adjusted to account for antecedent moisture condition (AMC). This adjustment was not done in this study to simplify the modeling process, and an AMC of 2 was used. This generalization resulted in rainfall losses being lower than in the BRWS, however, they are of the same magnitude as the BRWS peak flows, as evidenced by the comparison of peak flows in Table 3.

The BRWS used a 24-hour duration, alternating block frequency storm to simulate rainfall for 2-year, 10-year, 50-year, and 100-year recurrence intervals. A 500-year event was simulated by extrapolating rainfall totals of the more frequent events. Rainfall depths were taken from Kansas Department of Transportation rainfall tables for Johnson County, Kansas (KDOT 1997). Routing was completed using the Modified Puls method. To accomplish this, an iterative process utilizing both the HEC-1 and HEC-RAS models was used. This modeling approach was employed for this study, and is described in detail in Section 2.4.

Table 3. Comparison of HEC-1 and HEC-HMS Peak Flows

HEC-RAS Cross Section	2-year Peak Flow - HEC-1 (cfs)	2-year Peak Flow - HEC-HMS (cfs)	Percent Difference	10-year Peak Flow - HEC-1 (cfs)	10-year Peak Flow - HEC-HMS (cfs)	Percent Difference	50-year Peak Flow - HEC-1 (cfs)	50-year Peak Flow - HEC-HMS (cfs)	Percent Difference
20.054	340	341	0.2%	551	552	0.1%	772	773	0.2%
19.934	514	516	0.4%	835	838	0.3%	1,172	1,176	0.4%
19.669	788	792	0.5%	1,332	1,339	0.5%	1,999	2,011	0.6%
19.355	991	1,000	0.9%	1,677	1,693	0.9%	2,484	2,509	1.0%
19.117	1,051	1,062	1.0%	1,723	1,739	0.9%	2,552	2,579	1.1%
18.919	1,228	1,241	1.1%	2,001	2,023	1.1%	2,930	2,970	1.4%
18.72	1,274	1,288	1.1%	2,074	2,096	1.1%	3,060	3,101	1.3%
18.603	1,357	1,373	1.2%	2,209	2,235	1.2%	3,250	3,295	1.4%
18.152	1,717	1,743	1.5%	2,817	2,856	1.4%	4,000	4,052	1.3%

Table 3 (continued). Comparison of HEC-1 and HEC-HMS Peak Flows

HEC-RAS Cross Section	100-year Peak Flow - HEC-1 (cfs)	100-year Peak Flow - HEC-HMS (cfs)	Percent Difference	500-year Peak Flow - HEC-1 (cfs)	500-year Peak Flow - HEC-HMS (cfs)	Percent Difference
20.054	867	869	0.2%	1,173	1,175	0.2%
19.934	1,319	1,324	0.3%	1,787	1,793	0.3%
19.669	2,270	2,283	0.6%	3,088	3,104	0.5%
19.355	2,822	2,849	0.9%	3,842	3,877	0.9%
19.117	2,924	2,959	1.2%	4,040	4,080	1.0%
18.919	3,348	3,394	1.4%	4,795	4,853	1.2%
18.72	3,500	3,549	1.4%	5,028	5,094	1.3%
18.603	3,714	3,769	1.5%	5,371	5,444	1.4%
18.152	4,509	4,577	1.5%	6,531	6,640	1.7%

2.3 Hydraulic Modeling

Hydraulic modeling of the study reach was accomplished using the HEC-RAS version 4.0. As with the hydrologic modeling, the BRWS hydraulic models were used as a baseline. The BRWS completed hydraulic modeling using HEC-RAS version 2.2. Additional detail was added to the model for this study by adding cross sections to the model, as shown in Figure 5. The average distance between cross sections for the baseline model was 530 feet, and the model used for this study had an average distance of 300 feet. Manning's n values were developed for the new cross sections based on aerial photography and Table 8-4 from the BRWS report, reproduced below as Table 4.

Table 4.
Land Surface Characteristics and Associated Manning n Values

Land Use	Range of Modeled n Values
Grass, urban and maintained	0.025–0.035
Trees and brush	0.035–0.160
Residential areas	0.035–0.15
Agricultural, Pasture	0.025–0.050
Pavement	0.013–0.025
Lake	0.0160–0.033
Concrete-Lined Channel	0.011–0.020
Natural Channel	0.025–0.080

Two hydraulic structures were included in the BRWS HEC-RAS model and retained in the HEC-RAS model for this study. These were the West 183rd Street and Ridgeview Road crossings.

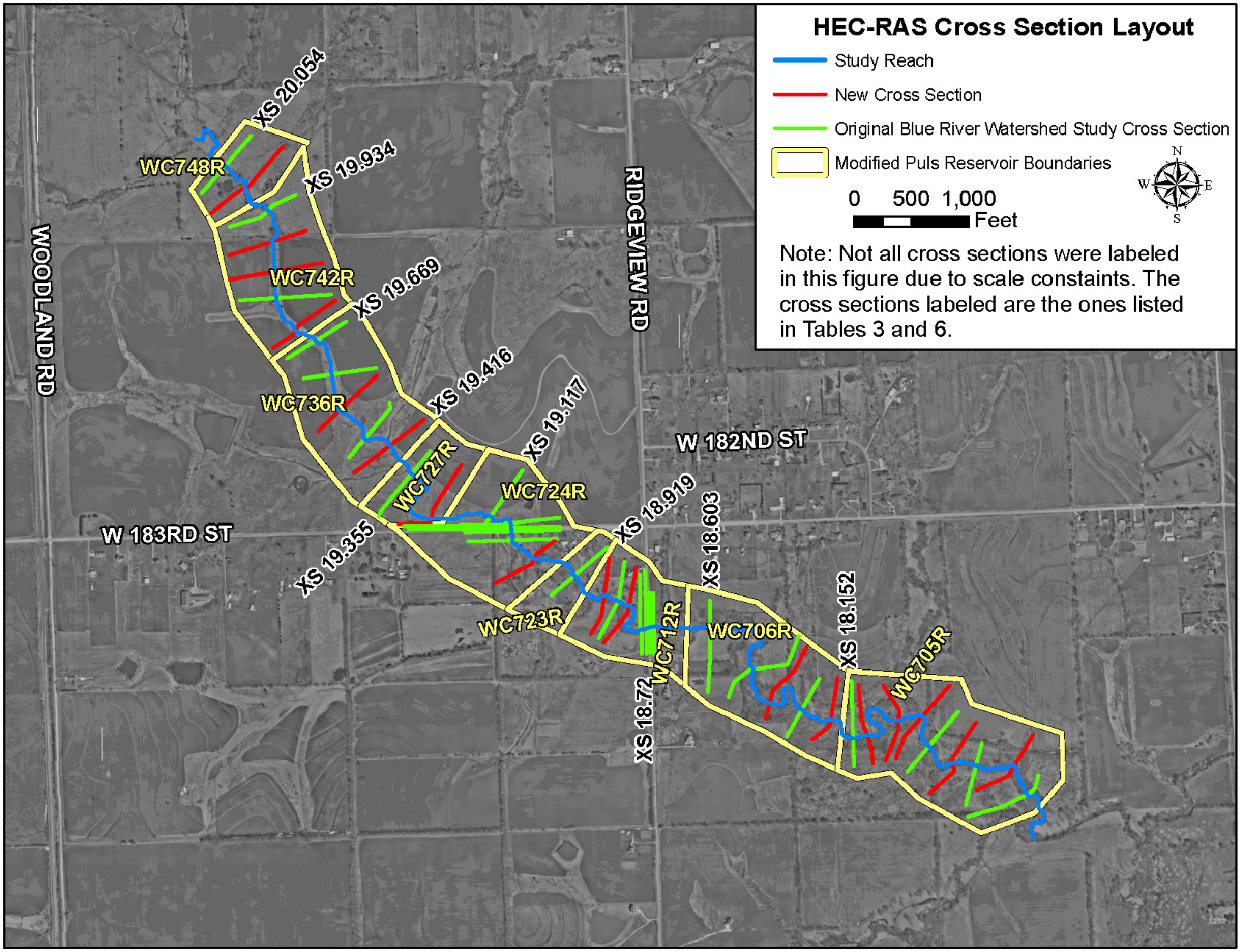

Figure 4. HEC-RAS Cross Sectional Layout and Modified Puls Reservoirs

2.4 Stream-Buffer Simulation Using Modified Puls Routing

The stream buffer was assumed to begin at the stream banks of the cross sections in the HEC-RAS model. The baseline hydrologic modeling was completed using the Modified Puls routing method. The extents of each of the Modified Puls reservoirs are shown in Figure 5. This method was analyzed in depth by Heatherman (2008) and was employed by the BRWS in the following fashion:

1) An initial run was completed in which the Muskingum-Cunge method was used for routing. The 2-year, 10-year, 50-year, 100-year, and 500-year rainfall events were included, as well as one hypothetical rainfall event less than the 2-year and one greater than the 500-year. The event less than the 2-year was developed by multiplying the 2-year event rainfall totals by 0.5, and the hypothetical event larger than the 500 year was developed by multiplying the rainfall totals for the 500 year by 1.5.

2) The "initial run" HEC-1 flows were input to HEC-RAS to find channel volumes. These volumes were then paired with the flows by recurrence interval to produce storage volume-outflow curves for each Modified Puls reservoir.

3) The new volume-outflow curves were then input to the HEC-1 model, and new peak flows obtained. These were then input to the HEC-RAS model to produce new storage volume-outflow volumes.

4) This process was repeated until the storage volume-outflow curves converged, and less than 10% difference between runs was observed. This convergence occurred between the third and fourth run.

A similar process was employed to establish existing conditions with the updated HEC-RAS model. The hydrologic model "initial run" was completed using the Modified Puls method with the final storage volume-outflow curves from the BRWS, and steps 2 through 4 of the BRWS procedure repeated. In addition, Equation 4 below from Heatherman (2008) was used to calculate the number of subreservoirs to be used for each Modified Puls reservoir.

$$N = \frac{SA * Z_r}{Q\left(\frac{\Delta VOL}{\Delta Q}\right)} \tag{4}$$

where:

SA = Surface Area of Steady-state Water Surface (square feet)

Z_r = Change in Water Surface from the Upstream End of Reservoirs to the Downstream End (feet)

Q = Representative Flowrate, Taken as the Average of the Design Event Flowrates (cubic feet per second)

ΔVOL = Change in Volume between Design Events (cubic feet)

ΔQ = Change in Flowrate between Design Events (cubic feet per second)

The number of subreservoirs for each Modified Puls reservoir was calculated for each event and the average of these was input to the model. The number of subreservoirs was also recalculated at each iteration of Step 4 and changed if appropriate.

To simulate the effect of a stream-buffer, a new HEC-RAS model was developed which utilized the "Vertically-Varied Manning's n" option in the Cross Section Editor. This option allows the user to input a curve defining a relationship between Manning's n and water-surface elevation for a portion of the cross section. This was accomplished by first establishing a curve relating

Manning's n and stage for a single segment of the overbank, where a segment is the portion of the cross section between two points. This curve is given in Table 5.

Table 5. Depth – Manning's n Relationship Used

Depth of Water Over Segment Midpoint, ft	Manning's n
0.0	0.20
0.1	0.20
1.1	0.18
2.6	0.10
5.0	0.08
10	0.03

This relationship was based conceptually on the following assumptions for vegetation in the overbank:

- A well established, 3 foot high layer of grasses and dense ground vegetation
- Dense, mature tree growth with average heights of 10 feet

Next, the Manning's n for each segment was calculated based on the depth of water over the segment at its midpoint for a range of water-surface elevations. The Manning's n value for the overbank at each water-surface elevation was then calculated as the length-weighted average of inundated segments. A curve of water-surface elevation versus length-weighted Manning's n was calculated for the left and right overbanks of each cross section and input to the HEC-RAS model. The process for Modified Puls routing was then completed again using this new HEC-RAS geometry to find peak flows for the reach with a stream-buffer.

Section 3. Results

The peak-flow results for existing conditions and with-stream-buffer conditions are given in Table 6. These results show large reductions in peak flow, and as the stream corridor is extended downstream, the modeled reductions in peak flow generally become greater. This indicates that peak-flow attenuation due to overbanks is a cumulative effect. In general, the downstream Modified Puls reservoir, WC705R, did not function consistently with the other reservoirs. Peak flow attenuation for the reservoir was generally not consistent with reservoir WC706R, which is immediately upstream of WC 705R, but varied both higher and lower than WC 705R for the modeled events. This effect was likely due to its position at the downstream boundary of the model. The results for it were not included in Table 6 or the following discussion.

It must be noted that two existing culvert crossings were included in the model. These were located in Modified Puls Reservoirs WC724R and WC712R. These bridges likely cause increased peak-flow attenuation for the 2-year event. The larger events overtopped these structures, causing high weir flows over the decks, and their effect on peak flows is therefore diminished for these events.

Table 6. Comparison of Modeling Results – With and Without Stream Buffer

Modified - Puls Reservoir	Cross Section	2-year Peak Flow Without Stream Buffer, cfs	2-year Peak Flow With Stream Buffer, cfs	Percent Difference from Existing	10-year Peak Flow Without Stream Buffer, cfs	10-year Peak Flow With Stream Buffer, cfs	Percent Difference from Existing	50-year Peak Flow Without Stream Buffer, cfs	50-year Peak Flow With Stream Buffer, cfs	Percent Difference from Existing
WC748R	20.054	341	341	0%	552	552	0%	773	773	0%
WC742R	19.934	501	491	2%	825	804	3%	1,165	1,127	3%
WC736R	19.669	887	844	5%	1,478	1,390	6%	2,095	1,978	6%
WC727R	19.355	1,068	989	7%	1,799	1,631	9%	2,587	2,340	10%
WC724R	19.117	1,115	965	13%	1,826	1,629	11%	2,742	2,391	13%
WC723R	18.919	1,264	1,039	18%	2,027	1,782	12%	3,148	2,639	16%
WC712R	18.72	1,320	1,077	18%	2,122	1,848	13%	3,333	2,733	18%
WC706R	18.603	1,405	1,129	20%	2,245	1,930	14%	3,570	2,865	20%
WC705R	18.152	-	-	-	-	-	-	-	-	-

Table 6 (continued). Comparison of Modeling Results – With and Without Stream Buffer

Modified - Puls Reservoir	Cross Section	100-year Peak Flow Without Stream Buffer, cfs	100-year Peak Flow With Stream Buffer, cfs	Percent Difference from Existing	500-year Peak Flow Without Stream Buffer, cfs	500-year Peak Flow With Stream Buffer, cfs	Percent Difference from Existing
WC748R	20.054	869	869	0%	1,175	1,175	0%
WC742R	19.934	1,313	1,270	3%	1,776	1,731	3%
WC736R	19.669	2,368	2,224	6%	3,220	3,043	6%
WC727R	19.355	2,930	2,625	10%	3,995	3,645	9%
WC724R	19.117	3,082	2,713	12%	3,947	3,788	4%
WC723R	18.919	3,538	3,002	15%	4,398	4,224	4%
WC712R	18.72	3,765	3,123	17%	4,642	4,377	6%
WC706R	18.603	4,031	3,269	19%	4,993	4,607	8%
WC705R	18.152	-	-	-	-	-	-

3.1 Discussion of Results

As discussed in the Section 1, the overbank conveyance capacity per unit area within the overbank is lower than in the main channel, and this effect results in the attenuations in peak flow observed. According to Manning's equation, conveyance capacity per unit area multiplied by the square root of friction slope is equivalent to velocity, as division of both sides of Manning's equation by area results in an equation which determines velocity. Average channel and overbank velocities were taken from HEC-RAS output and are compared for the without-stream-buffer and with-stream-buffer scenarios in Table 7. Changes in hydraulic geometry parameters are also given in Table 7 to illustrate the effect on flow attenuation of the overbank geometry calculated using the energy equation in HEC-RAS. A positive percent increase in Table 7 represents an increase in the reported parameter for the with-stream-buffer scenario compared to the without-stream-buffer scenario, and a negative percent increase in Table 7 represents a decrease.

Table 7. Modeled Peak-Flow Attenuation and Differences in Study Reach Velocities and Hydraulic Properties for Each Modeled Event

Return Interval	Largest Modeled Peak-Flow Attenuation Observed	Percent Difference in Channel Velocity	Percent Difference in Overbank Velocity	Percent Difference in Average Overbank Hydraulic Radius	Percent Difference in Average Overbank Area	Percent Difference in Average Wetted Perimeter	Percent Difference in Average Overbank Manning's n	Average Change in Water-Surface Elevation, feet
2	20%	-7%	-68%	19%	49%	27%	245%	0.3
10	14%	0.1%	-70%	10%	35%	23%	276%	0.5
50	20%	0.1%	-68%	15%	37%	21%	299%	0.6
100	19%	1%	-67%	18%	38%	22%	302%	0.6
500	8%	10%	-61%	23%	31%	14%	291%	0.8

Increases in overbank Manning's n did not appear to correlate to increases in peak-flow attenuation, and therefore they did not account for differences in peak-flow reduction alone. Therefore, explanations for differences in peak-flow attenuation were investigated in the hydraulic geometry determined by HEC-RAS. For the 2-year event, both average channel velocity and average overbank velocities were reduced, and average overbank hydraulic radius was also increased. The increased overbank hydraulic radius indicates that more volume was stored in the overbank for the with-stream-buffer scenario, and as a result attenuation in peak flow was observed. However, the peak-flow attenuation for the 2-year event is likely exaggerated by the inclusion of the two culvert stream crossings in Modified Puls Reservoirs WC724R and WC712R. For both without and with stream-buffer scenarios, the 2-year water surface was below the lowest elevation of the decks of both culvert crossings, and this caused the area immediately upstream of the crossings to act as detention basins, adding to peak-flow attenuation caused by overbank storage. It is likely that without the crossings in the model, the maximum peak-flow reduction would be significantly diminished, and may fall below the maximum 14% peak-flow reduction observed for the 10-year event.

For the 10-year event, average channel velocity is slightly increased. Average overbank velocities were reduced by more than the 2-year event. However, average overbank hydraulic radius did not increase compared to the 2-year event, as the overbank flow area and wetted perimeter had a smaller percent increase than the 2-year event. This indicates that due to the hydraulic geometry of the study reach, the 10-year event was not able to store as significant an amount of floodwater within the overbanks as the 2-year event.

For the 50-year event, average channel velocity was slightly increased. Reduction in average overbank velocity was similar to that observed for the 10-year event. Because a higher increase in average overbank hydraulic radius for the 50-year event was observed compared to the 10-year event, a higher reduction in peak flows was also observed.

For the 100-year event, a small increase in average channel velocity was observed accompanied by a significant reduction in average overbank velocity. The reduction in overbank velocity for the 100-year event, however, was slightly lower than the reduction observed for the 50-year event. This resulted in peak-flow attenuation being slightly lower than the attenuation observed for the 50-year event. This lower peak-flow attenuation was in spite of an increase in average hydraulic radius compared to the 50-year event. This result would suggest that for the study reach, peak-flow attenuation begins to be diminished for the 100-year event. This result was predicted by Bhomik and Stall (1979), as discussed in Section 1.

A drop in peak-flow attenuation was observed for the 500-year event when compared to the 100-year event. This appears to be caused by a large increase in channel velocity and a smaller decrease in overbank velocity compared to the other modeled events. This would suggest that for the study reach, peak-flow attenuation is significantly diminished for events 500-year event. This result was predicted by Bhomik and Stall (1979), as discussed in Section 1.

If stream-buffers located at headwater reaches such as the study reach for this paper throughout the Wolf Creek watershed were used to reduce downstream peak flows, a large reduction in peak flows at downstream reaches would be observed for a wide range of storm events. This could decrease costs to a municipality in multiple ways, including reduced infrastructure costs as drainage structures could be sized smaller, reduction in downstream flooding damages, and reduced need for flood control projects. These benefits would need to be weighed against any economic costs incurred by the ordinance, such as decreased developable land in adjacent property.

The results of this study show large reductions in peak flow as a result of implementation of a stream buffer, however, the land use prior to the stream-buffer of this study must be considered. Existing conditions overbank Manning's n values for the study reach were representative of farmland, and thus were very low – on the order of 0.035, which is a typical channel n value. The magnitude of peak-flow reduction observed is due to the difference between the existing overbank Manning's n values and the much higher hypothetical overbank Manning's n values. Table 7 summarizes the average increase in Manning's n for each modeled event.

Some general principles can be extracted from these results. Development of land located in channel overbanks is extremely varied, and depending on its nature can have a range of effects on overbank flow. Dense, highly-urbanized development can be composed of many obstructions causing loss of overbank conveyance, which, from a hydraulic modeling perspective, is equivalent to a stream-buffer with a high overbank Manning's n. In this case, a stream-buffer may not provide significantly higher peak-flow attenuation.

Development can also take the form of areas sparsely populated by buildings and mostly composed of maintained urban lawns. An overbank composed of this development would generally be modeled with low Manning's n values. In this case, a stream-buffer could produce peak-flow attenuations consistent with those shown by this study. Because of this, this study represents an "upper limit" for peak-flow reduction due to stream-buffers.

3.3 Peak-Flow Attenuation of Johnson County, Kansas Stream-Buffers

The peak-flow reduction results for larger events may not apply in all municipalities. The width of the stream-buffer was not limited for this study and was equal to the HEC-RAS overbank width. In practical applications, stream-buffer ordinances designate a specific width of the stream-buffer, and this will have to be taken into account when modeling specific instances of peak-flow attenuation due to overbank stream-buffers.

Table 8 summarizes stream-buffers ordinance of various cities located within Johnson County, Kansas.

Table 8. Stream-Buffer Ordinances in Johnson County, Kansas

City	Drainage Area	Distance from Stream Bank on Either Side of Stream, feet
Overland Park, Olathe, and Gardner, KS	25 to 40 acres	30
	40 to 160 acres	60
	160 to 5,000 acres	100
	5,000 acres and greater	120
Lenexa, KS	Stream Order 1, Sensitive Stream*	150

* Study reach fell within appropriate parameters for Stream Order 1 designation, and was assumed to fall within that category

The drainage area modeled to the top of the study reach was greater than 215 acres, and the maximum drainage area to the study reach was 1,716 acres. In the cities of Overland Park, Olathe, and Gardner, the stream-buffer would be 100 feet from the stream bank and in the city of Lenexa, it would be 150 feet. Table 9 shows the average overbank top widths observed in the modeling results.

Table 9. Modeled Overbank Top Widths by Event

Event	Left Overbank Average Top Width, feet	Right Overbank Average Top Width, feet	Average of Overbank Top Width, feet*
2-year	84	104	94
10-year	134	143	139
50-year	170	188	179
100-year	170	188	179
500-year	213	221	217

* - Average of Overbank Top Width was calculated as the average of the left and right overbank top widths

The modeled overbank top widths indicated that the stream-buffer ordinances in Overland Park, Olathe, and Gardner would provide the modeled peak-flow attenuation shown in Table 6 at the study reach for the 2-year event. Events larger than the 2-year would not observe the same levels of attenuation as those shown in Table 6.

For the Lenexa stream-buffer ordinance, runoff up to the 10-year event would be contained within the stream-buffer, and the levels of peak-flow attenuation shown in Table 6 for the 2-year and 10-year events would be observed at the study reach. Events larger than the 10-year would not observe the same levels of attenuation as those shown in Table 6.

If the maximum possible attenuation of peak flows for events larger than the 10-year event is one of the goals of a stream-buffer ordinance, these results indicate the current stream-buffer extents are not adequate to provide it. However, the stream buffer would still provide much of the peak flow attenuation observed in the results of this paper for these larger events.

An important factor when designing stream-buffer ordinances is the difference between the local effect on the area which includes the stream-buffer and the effect on areas downstream of a reach with a stream-buffer. In general, peak-flow attenuation due to overbank storage is a benefit to downstream areas. At the same time, the local effect is that of lower stream flowrates accompanied by higher water-surface elevations for subcritical flow. Figure 6 below is a profile plot showing the differences in water-surface elevations along the study channel for the 100-year event. An average increase of 0.4 feet and a maximum increase of 1.1 feet was observed on the study reach with the stream-buffer. Changes in flow caused by stream-buffers can lower or raise water surfaces, and this effect should be taken into account when considering implementation of a stream-buffer, especially for larger events which extend beyond the boundary of the stream corridor.

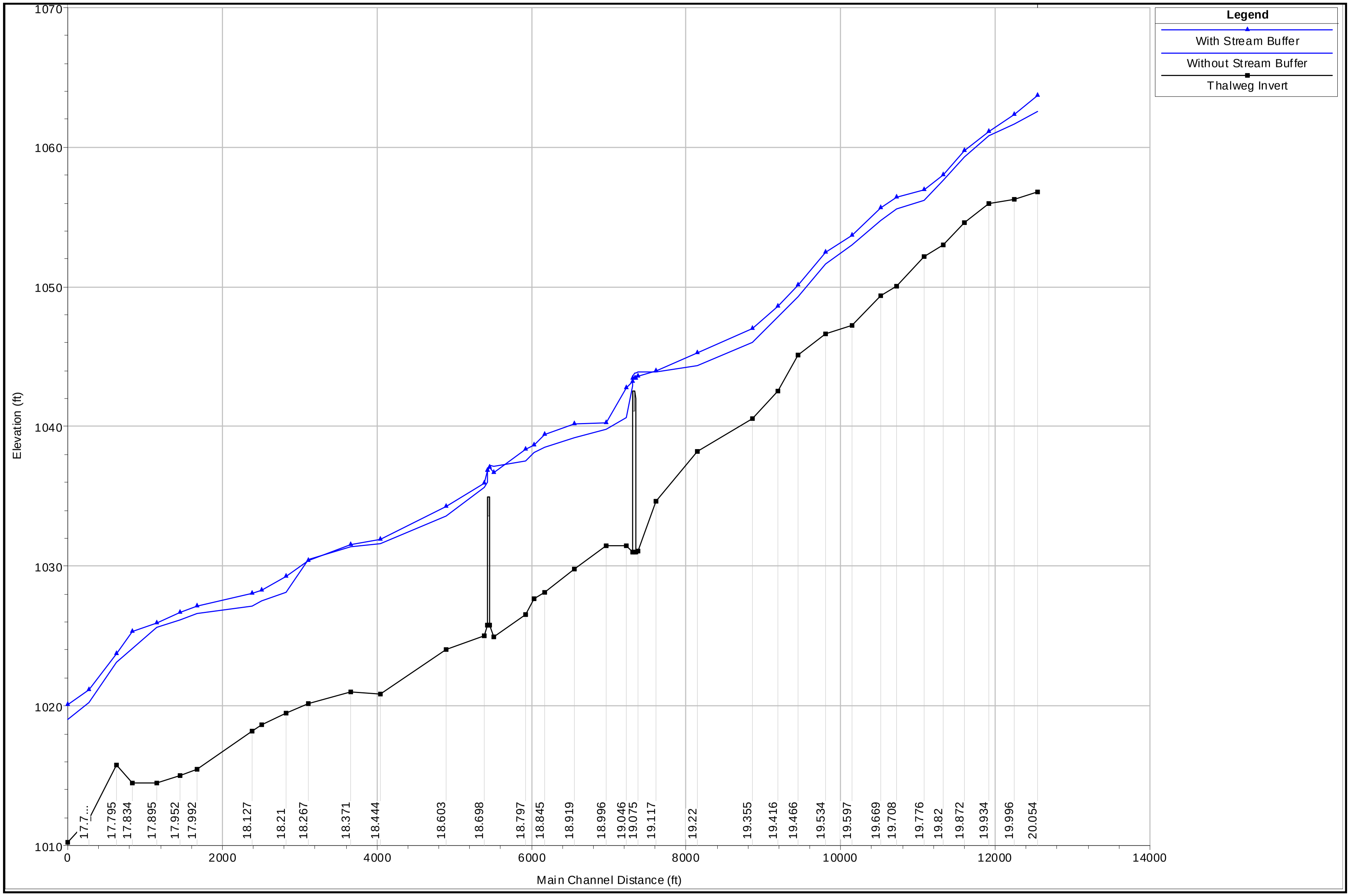

Figure 5. 100-year Event Water Surface Profiles for With and Without Stream-Buffer Conditions

3.4 Recommendations for Future Study

The reductions of peak flow observed in this study were a result of simulating stream buffers. The n values used for the buffers were vertically varied according to the curve specified in Table 5. This curve has not been experimentally verified, and future studies may need to be conducted to find vegetation planting schemes which could produce large overbank peak flow attenuation . A relationship between Manning's n, depth of flow, percent of overbank with vegetation, and longitudinal flow velocity has been developed (Fathi-Moghadam, et al., 2011). This paper was published after the modeling portion of this study was completed, and thus was not incorporated. Future studies could attempt to replicate this study using the relationship developed by Fahti-Moghadam.

Such studies could also make recommendations to help create a more accurate curve based on vegetation type. These studies may also wish to investigate cost-effective and easy-to-maintain planting schemes for various regions to produce desired attenuation of peak flows.

Wolteman and Potter (1994) indicate that for large watersheds with established riparian corridors, peak flows become a function of the volume of a rainfall event more than the intensity of a rainfall event. This study used an alternating block rainfall event in which a high intensity occurs at the midpoint of the event. Future studies could investigate whether the same peak-flow reductions found by this study are also true of high-volume rainfall events.

If a municipality were to determine design flows for infrastructure based on a with-stream-buffer condition using the methodology outlined in this paper, it would be important to take into account the seasonality of Manning's n values. The values assumed for this study are valid only for the growing season, which extends from approximately May through September for the study site. Lower Manning's n values and overbank peak-flow attenuation are expected for the non-growing season, and infrastructure would be undersized for a rainfall event occurring at this time for equivalent rainfalls. At the same time, the rainfall depths associated with design events are expected to be lower for the non-growing season. It is therefore not immediately obvious whether peak flows would be higher for design events occurring during the growing or non-growing season. Future research could establish design event rainfall depths for the non-growing season, and these depths could be modeled using the methodology outlined in this paper with appropriately lower Manning's n values. The higher of the peak flows for the growing season and non-growing season analyses would then be used as the infrastructure design flows.

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