Spring 2012

Monitoring and Modeling of the Hydraulic and Sediment Processes at In-Stream Restoration Structures in the Vicinity of a Bridge Crossing

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MONITORING AND MODELING OF THE HYDRAULIC AND SEDIMENT PROCESSES AT IN-STREAM RESTORATION STRUCTURES IN THE VICINITY OF A BRIDGE CROSSING

by

Brian P. Schultz

A Thesis

Presented to the Faculty of
Bucknell University
In Partial Fulfillment of the Requirements for the Degree of
Master of Science in Civil Engineering

Approved: Jessica T. Newlin
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May, 2012

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Acknowledgments

I would first like to thank my friends and classmates that volunteered their time and effort helping me collect data at the White Deer Creek study site. These volunteers include: Ben Stodart, Brian Charland, Tim Horlacher, Jordan Makansi, Nick Noss, Sean Reese, and Zak E. Among these volunteers, I would like to thank Brian Charland, Tim Horlacher, and Nick Noss along with Randy Gowat for their continued support both inside and outside of the university. Without the support of my friends and classmates, the research presented would simply not be the same.

I would also like to thank my thesis committee, Professor Richard Crago and Professor Craig Kochel for their valued input and support throughout the research.

Special thanks are extended to Ben Hayes and Jim Gutelius for their assistance with the installation and implementation of research equipment at the White Deer Creek site, as well as their support with problems that arose with equipment used at the site.

Most importantly, I would like to thank my advisor, Professor Jessica Newlin for all of the time and effort she has put into advising me throughout my time spent here at Bucknell University. Without her guidance, this opportunity to expand myself as a professional would not have been possible.

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Abstract

The long-term performance of infrastructure depends on reliable and sustainable designs. Many of Pennsylvania's streams experience sediment transport problems that increase maintenance costs and lower structural integrity of bridge crossings. A stream restoration project is one common mitigation measure used to correct such problems at bridge crossings. Specifically, in an attempt to alleviate aggradation problems with the Old Route 15 Bridge crossing on White Deer Creek, in White Deer, PA, two in-stream structures (rock cross vanes) and several bank stabilization features were installed along with a complete channel redevelopment. The objectives of this research were to characterize the hydraulic and sediment transport processes occurring at the White Deer Creek site, and to investigate, through physical and mathematical modeling, the use of in-stream restoration structures. The goal is to be able to use the results of this study to prevent aggradation or other sediment related problems in the vicinity of bridges through improved design considerations. Monitoring and modeling indicate that the study site on White Deer Creek is currently unstable, experiencing general channel down-cutting, bank erosion, and several local areas of increased aggradation and degradation of the channel bed. An in-stream structure installed upstream of the Old Route 15 Bridge failed by sediment burial caused by the high sediment load that White Deer Creek is transporting as well as the backwater effects caused by the bridge crossing. The in-stream structure installed downstream of the Old Route 15 Bridge is beginning to fail because of the alignment of the structure with the approach direction of flow from upstream of the restoration structure.

1. Introduction

1.1 Stream Channel Response at Bridge Crossings

When a bridge crosses over a stream it obstructs the stream's normal conditions. The natural response of the stream can cause unwanted changes in the vicinity of the bridge and possibly cause a high level of channel instability. To an engineer, a stream that is migrating, or cutting into nearby infrastructure is considered unstable. A geologist would see this channel as being in a state of disequilibrium due to the changes made to the watershed. The stream is trying to approach a new equilibrium by adjusting and creating a new channel path or adjusting the balance between hydraulic and sediment loads transported by the channel. An abundance of research has proven that channel modifications, such as infrastructure placed along, over, or throughout the stream cause accelerated channel instability (Ruhe (1970); Emerson (1971); Wilson (1979) and Simon (1989, 1992)). Channel instability, in turn, can cause infrastructure damage and even failure. Therefore, bridge safety relies heavily on the stability of both the structure and the underlying stream.

Aggradation, or sediment deposition, is observed at many bridges within the United States. Nationwide, problems associated with aggradation can be observed and, in some cases, have caused extensive damage (Moore and McCarl 1987). Aggradation occurs when flow is slower before, under, and after the bridge compared to the rest of the stream reach. Often, bridge openings are widened to accommodate a water discharge as determined by current bridge design guidelines. When sediment carried in a faster flow reaches slower moving flow in the vicinity of a bridge, the sediment settles down onto the streambed. Aggradation can pose a serious problem to the overall bridge structure. This process causes the bridge waterway opening size to be reduced. Each bridge is designed to convey a certain magnitude flood, or design flood, through the bridge span. When the span becomes partially blocked by sediment, the bridge will no longer be able to pass the design flood through the opening. This can cause flooding upstream of the bridge and more frequent overtopping of the bridge deck; both of which present a public safety hazard and can increase the risk of bridge failure.

While the structural integrity of bridges is important to maintain, the presence of bridge infrastructure over stream channels also can have an impact on the integrity of the ecosystem supported by the stream channel. Since streams naturally collect inputs of sediment and water and transport them through a channel network, over time channel features develop. These features represent the stream's history of hydrologic and geomorphic processes (Jones et. al. 2000). Because a roadway crossing can change the normal way that sediment and/or water is carried by the channel, the flow could become increased or change directions from its normal path. For instance, erosion and aggradation of sediment at bridge crossings are a response to the changed sediment and water transport in the channel, and can change the environment of the ecosystem that is supported by the stream. To prevent sediment transport and flooding problems, engineers need to understand the water conditions at which sediment deposition or erosion begins to occur and design bridges so that these conditions are anticipated and accounted for within the design. This more comprehensive design approach would reduce the risks to public safety, of bridge failure, and of ecosystem discontinuity.

1.2 Geologic and Land Use History Effects on Sediment Processes

In addition to the local disturbance of a stream channel by a bridge crossing, large scale disturbances also can have an effect on the equilibrium of a stream channel. Stream behavior is greatly dependent on the land use and geology within its watershed. Major disturbances such as forest fires, mining, and logging can cause many changes to the stream's behavior. It is well documented that such a disturbance can cause immediate downstream aggradation and widening, due to a pulse of sediment, followed by a gradual recovery period, after the disturbance has ended, where the channel incises into the deposited sediment (Gilbert 1917; James 1991, 1994). When this happens, the aggradation and widening can cause more frequent and larger flooding in the aggraded areas (Glenn 1911), while the incising process can cause damage to existing infrastructure.

A study in 2006 investigated channel response due to forest clearance on the southern Blue Ridge Mountains (Price and Leigh 2006). Several streams were investigated for changes in morphology and sedimentology after human induced impact on the watersheds. The study showed that a common occurrence among all streams was that the bankfull width to depth ratios were lowered, the wetted width of baseflows was narrower, and there was an increase in fine sediments on the stream bed. All of these changes can cause damaging effects to the stream, any nearby infrastructure, and its ecology. A decrease in bankfull width to depth ratios and wetted width both indicate that the channel is incising. Incision can lead to the undercutting and eventual failure of nearby infrastructure. An increase in the amount of fines is associated with decreases in the health of macro-invertebrate life (Roy et al. 2003).

A separate study, by Delaney et al. (2006), on Grays Run in Central Pennsylvania looked into the channel adjustments due to the historic land use changes of the Grays Run watershed. The findings show that Grays Run is currently in disequilibrium conditions with respect to channel dynamics and sediment movement. Channel widening and incising processes could be found along the stream as well as gravel point bars and mid-channel bars actively migrating downstream. Both erosional and depositional processes were observed, sometimes at the same location along Grays Run. These processes are all part of the channels' response to the historical logging of the area, along with the rapid reforestation that occurred in the years after. Often, these large scale disturbances can increase the adverse effects observed locally at bridge crossings.

1.3 Solutions to Sediment Problems at Bridges

Solutions to sediment transport problems at bridge crossings exist but can be highly expensive and/or only can allow for a short-term solution for a year or two. A common solution for aggradation is to dredge the deposited sediment as necessary. To do this, the sediment that piles up is excavated with machinery and transported away from the stream. Although this method reduces the blockage of bridge openings, it needs to be done throughout the entire service life of the bridge and has a possibility of introducing bank and bed erosion elsewhere on the reach (Johnson et. al. 2001). Some habitat is destroyed in this process, and the environment that can be created from dredging often acts as a barrier to the connectivity of the stream channel ecosystem (Jones et. al. 2000).

Dredging channels can easily become very costly, especially when large amounts of sediment are available from the upstream areas of the watershed. In 2001, Johnson et. al. found the cost of dredging for a single bridge to be approximately $12,000 a year. In 2002, it was estimated that it cost the United States $257 million dollars a year for dredging all sites with sediment deposited from inland erosion sources (Hansen et. al. 2002). Dredging is not only costly, but also is governed by regulations that can prevent the complete removal of all on-site aggradation. In Pennsylvania, dredging is only allowed to be done on the Department of Transportation's (DOT's) given right-of-way for a bridge crossing that they own. The right-of-way extends 7.5 meters upstream and downstream from the bridge. If any sediment is deposited outside of the DOT's right-of-way, it cannot be removed unless further permitting is completed.

1.4 In-stream Structures

Instead of dredging, another solution to sediment transport problems around bridges that is becoming more widely used involves the design and implementation of stream restoration structures within the target reach. There are many types of engineered restoration structures that are currently used in the industry to help stabilize a channel. These can be grouped into two main categories according to the Maryland Department of the Environment (MDE 2000). The first category, slope protection and stabilization techniques, include many permanent techniques that help stabilize and protect stream embankments from erosion. Two common examples of such measures are riprap and brush layering. Riprap is large rocks placed along the stream banks to resist the shear stresses that would normally cause erosion of the smaller existing bank material. Brush layering accomplishes the same feat, but uses woody vegetation instead of large rocks. Other methods available are gabions, live stakes, and toe protection. The second group, channel stabilization and rehabilitation techniques, are installed mainly for channel stabilization and to benefit/create aquatic habitat through scour, oxygenation, and providing cover. Examples of such techniques are weirs, vanes, and deflectors. Weirs and vanes are similar in appearance and are both designed to benefit aquatic habitat and help control the channel grade. The shape of a weir can vary from a broad 'U' shape to a narrow 'W' shape depending on how they are designed, while a cross vane is more of a flat bottomed 'V'. They are built across the entire channel to force the water to flow over the structure and concentrate toward the center of the channel. They are typically built using rock, but wood could also be used. Deflectors are structures placed along the channel banks to redirect flow and create scour sections for improved aquatic habitat. These are generally constructed using both rock and wood and look simply like a triangle pointing into the stream from the bank.

The use of an engineered cross vane could create a smooth transition from ordinary flow to the constricted flow zone in the vicinity of the bridge (Johnson et al. 2002). Figures 1a and 1b show examples of a cross vane with schematic diagrams adapted from Johnson et al. (2002) and Rosgen (1998). Based on the effects on channel hydraulics, cross vanes are well-suited for channeling flow, providing grade control, and creating aquatic habitat (MDE 2000). A cross vane is constructed of two tiers of large boulders stacked side by side across the channel, and is considered a rigid engineering design. As seen in Figure 1a, the wings of the cross vane are angled to force the flow into the center of the channel as it passes over the boulders.

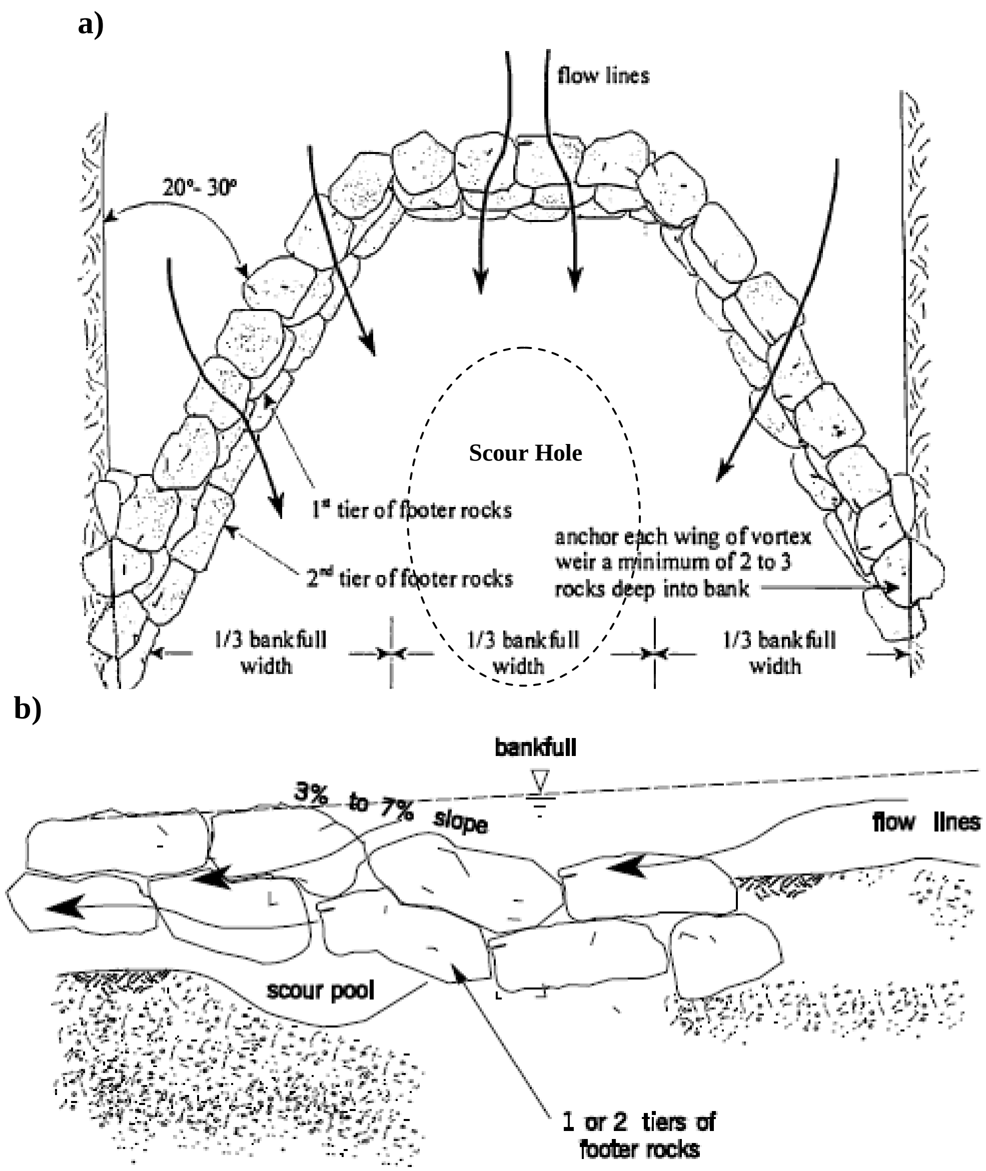


Figure 1 – Example of a cross vane as built in a stream in both (a) Plan View (Image adapted from Johnson et al 2002) and (b) Profile View (Image from Rosgen 1998).

The wings are also angled vertically, at less of a degree, down from the bankfull elevation near the banks to the bed elevation in the center of the channel. This creates an area of high shear stress and high velocity that, in turn, creates a large scour hole just downstream of the vane. Figures 1a and 1b display the expected location of the scour hole created by the structure. By placing a cross vane both upstream and downstream of a target bridge experiencing aggradation, the intention is that a high velocity zone beneath the bridge span is created. The higher velocity zone will not allow aggradation to occur in the bridge opening as it normally would without the in-stream structures. Rather, the sediment will be carried through the bridge span and out of its vicinity.

Research on the practice of stream restoration is limited and more research is needed to improve the design, construction, and performance of in-stream structures. A laboratory study done by Johnson et al. (2002) proved that restoration structures, particularly vanes, cross vanes, and W weirs, can be effectively used, in a flume, to create flow transitions through the restored stream in the vicinity of bridge openings. However, suggestions are made to increase field monitoring of the structures for longer time periods to capture larger events. Also, the authors mention different restraints that must be taken into consideration. Among these, they suggest placing the structures in higher velocity areas, and areas that do not experience backwater effects, as well as, using the structures in a stream that has a larger sediment size, gravel/cobble sized, to increase the effectiveness of the structures.

A field study in North Carolina by Miller and Kochel (2009) investigated the performance of in-stream structures built to stabilize the channel and increase aquatic habitat. The study documented the performance of 391 rock structures and ranked the integrity of each based on how well they functioned. The study found that within the six years of monitoring almost 24 percent of all rock structures experienced damage that caused a loss in function. It also found that any site that approached or exceeded a 20 percent change from its post construction channel capacity also experienced significant damage to any in-stream structures at that particular site, and throughout the entire study, this was true in over 40 percent of the cases. This implies that the channel adjustments caused by one in-stream restoration structure can eventually lead to the impairment of other in-stream structures.

In Northern and Central Pennsylvania, a field study was conducted by Kassab et. al. (2009) on the performance of a variety of stream restoration structures. The study involved investigating 22 sites, totaling over 300 restoration structures, and ranking the performance of the stream restoration structures. Structures investigated include: cross vanes (rock and log), deflectors, root wads, W-shaped weirs and many more. The ranking system was based on the structural integrity of each structure and also on how much erosion or deposition had occurred in its vicinity. Both of these factors influence how well the structure will perform. Their study concluded that approximately 75% of all structures have sustained some structural or erosional/depositional damages, while 35% of them are considered to have significant amounts of damage. Many of the damages were caused by erosional or depositional problems which either involved failure by burial of the structure with transported sediment, or by a moved or missing boulder on the structure.

In a separate study of a restoration project in Pennsylvania, performed by Niezgoda and Johnson (2006), the use of cross vanes, J-hooks, and rock linings was investigated for stream restoration purposes. The study involved long term modeling of the in-stream structures, field observations and surveying permanent cross sections to determine what the restoration project was accomplishing. The authors found both positive and negative aspects during the investigation. The in-stream rigid structures performed well to minimize bed degradation on the reach-wide scale, decrease the width-depth ratio, and provide areas of improved aquatic habitat. Problems found as a result of the structures' presence were possible areas of increased shear stress due to lateral flow constriction specifically around the outside of constructed meander bends.

2. Site Description

An ideal site was selected that experienced many of the problems discussed in the previous sections. The presence of a bridge crossing, historic deforestation/alteration of the watershed, and the installation of engineered cross vanes to mitigate aggradation at the bridge crossing are all found in White Deer Creek near White Deer, PA. The presented research focuses on the lower section of White Deer Creek at the Old Route 15 Bridge crossing. White Deer Creek is a tributary to the West Branch of the Susquehanna River beginning near Lavonia, PA and ending at White Deer, PA. Due to the surrounding ridges causing its long narrow shape, the White Deer Creek watershed is a trellis type watershed. The approximate watershed area is 122 square kilometers, with the majority of it currently being forested (94.5%). Figures 2 and 3 are current images of the research area on White Deer Creek.

Figure 2 – White Deer Creek upstream of the Old Route 15 Bridge, facing upstream (Image taken on 3-17-2010).

Figure 3 – White Deer Creek downstream of the Old Route 15 Bridge, facing downstream (Image taken on 9-23-09).

In the past, a large amount of deforestation and logging has caused the creek to become unstable (Skelly and Loy, Inc 2004a). That is, the stream is reacting to the large-scale disturbance of historic deforestation and causing damage to existing infrastructure along its length. Several sections of the stream also have been altered and straightened due to the construction of US State Route 15 and several other local roadways. The deforestation and channel alterations have caused a significant amount of gravel and cobble-sized sediment to begin moving downstream in the channel. This large amount of sediment had been building up near the Old Route 15 Bridge opening and caused the need for increased channel maintenance to prevent bridge infrastructure damage and increased flooding (see Figure 4a).

In 2004, a restoration project was planned for the lower portion of White Deer Creek to restore the creek and mitigate the sediment problems that occurred around the Old Route 15 Bridge. According to the designers, Skelly and Loy Inc. (2004b), there were five main problems associated with the site. These are:

1) Significant bank erosion downstream from the Old Route 15 Bridge
2) Coarse sediment deposition in the vicinity of the new and old Route 15 bridges
3) Poor aquatic habitat
4) Potential and existing infrastructure damage (failing riprap)
5) Chronic flooding problems (bridge conveyance blockage)

As built in 2006, this project involved the installation of two rock cross vanes (one upstream and one downstream of the Old Route 15 Bridge), channel bank stabilization using riprap (downstream of the bridges), and a reshaping of the channel to create a more uniform cross section. The riprap creates a non-erodible channel bank so that the channel will not change its course near the bridge or erode the bank farther toward the adjacent property. A large aggradation bar in the vicinity of the bridge was removed, as well as an abandoned water pipe. A woody riparian cover also was established to provide bank stabilization and aesthetic appeal along the channel banks (Skelly and Loy, Inc 2004b). Figures 4a and 4b display the section downstream of the bridge before and after the rock cross vane was installed. The cross vanes were installed, upstream and downstream of the bridge, to create a faster section of the channel to “carry” the sediment through the bridge opening and to prevent it from depositing in the vicinity of the bridge. A second benefit from the vanes is the creation of aquatic habitat, i.e. the creation of a scour hole.

a) **b)**

Figure 4 – Downstream from the Old Route 15 Bridge on White Deer Creek; (a) Before any restoration was implemented. (b) After the rock cross vane was installed, highlighted by bold white lines (Both images are facing upstream).

Since the completion of the restoration project, the upstream cross vane has completely filled in with sediment (Figure 5) and does not have a scour hole as intended by design (see Figure 1), while the downstream vane is still functioning (Figure 4b), despite several spots where the boulders in the cross vane wings have started to pull apart and move from their original locations. The channel cross sections throughout the entire restored reach have become less uniform than when the project was constructed, and several sections of the riprapped bank (downstream of the bridge) have loosened and even fallen into the channel (Figure 6).

Figure 5 – Image of the upstream rock cross vane (white lines) and the large gravel/cobble sized sediment that has buried it (White arrow shows flow direction).

Figure 6 – Image of a failed section of the riprapped bank downstream of the Old Route 15 Bridge (facing upstream).

3. Thesis Objectives

The research was conducted based upon two main objectives. The first objective was to characterize the White Deer Creek study site by monitoring and collecting hydraulic and sediment data. By consistently monitoring White Deer Creek, a continuous set of sediment and hydraulic data was recorded and used to help determine, or explain, the stream behavior, as well as to calibrate or validate hydraulic and sediment transport mathematical models and inform the development of physical models. The second objective was to investigate the use of stream restoration structures built to prevent aggradation in the vicinity of a bridge through various modeling approaches. Investigating the upstream cross vane failure to predict its burial will provide data needed to determine possible errors in design, maintenance, and/or construction of this in-stream structure. Also, comparing sediment and hydraulic processes upstream and downstream of the bridge will provide explanation for any observed differences in function or behavior of the two cross vanes. The monitoring and modeling results will provide suggestions for future use and feasibility of in-stream structures as mitigation for sediment aggradation at bridge crossings in similar regions.

4. Methodology

4.1 Watershed and Channel Investigation

To provide a better understanding of White Deer Creek, a watershed-wide field investigation was conducted. Several day walks of different areas of the creek were completed looking mainly at how the stream is behaving outside of the research area. Pebble counts were taken to quantify the sediment sizes and distributions throughout the creek. Pictures were taken of areas of aggradation, scour, and possible sediment sources to help understand the quantities of sediment moving through the creek.

4.2 Monitoring and Data Collection at the Study Site

The changes in the condition of the Old Route 15 Bridge crossing at White Deer Creek and the changes in the bed elevation within the vicinity of the bridge were monitored over the research period, from June 2009 through March 2012. The bed elevation monitoring was accomplished using a total station system to survey the main channel and overbanks from established reference points. The TOPCON GTS-603AF total station is an electronic surveying system that allows its user to survey the latitude, longitude, and elevation at any location within sight range of the total station device. The surveyed conditions of the stream were tied into the 1983 National Elevation Dataset values using an existing benchmark located on the Old Route 15 Bridge. This enabled overbank/floodplain elevation estimation from a National Elevation Dataset of the site retrieved from the United States Geologic Survey website (National Elevation Dataset 2011).

Pebble counts (Wolman 1954) helped characterize the sediment on the bed and on gravel bars deposited within the channel. Pebble counts also provided the sediment gradation data needed to model a sediment transport event. The pebble counts involved collecting a large number, no less than 100, of individual sediment pieces randomly and recording their intermediate diameters. The intermediate diameter of a pebble was determined using a gravelometer with multiple sized square openings ranging from 2 mm to 362 mm. The smallest opening through which a pebble passed was recorded as the intermediate diameter. The pebble counts were taken throughout the entire site as well as in upstream areas of the watershed.

To estimate quantities of moving sediment, scour chains were installed in the stream. A scour chain is a long metal chain connected to a metal rod that is secured deep into the bed of the channel. The chain is connected so that it can freely slide up and down the rod so that when the bed sediment begins to move the chain will slide down to the lowest point of sediment movement or scour. The chain can also show the amount of sediment that has deposited on an area by measuring the depth of sediment above the chain and along its length.

Several cross-sectional velocity distributions were measured and used for calibration, analysis, and for the determination of flow rates. The tools available to measure velocity distributions were the Marsh-McBirney electromagnetic velocity meter and a Sontek RiverSurveyor® acoustic Doppler current profiler (ADCP). The Marsh-McBirney velocity meter is used to measure several velocity values spaced across the stream cross section, along with the water depth at each measurement location. The data can then be numerically integrated to calculate a total flow rate for that cross section of the reach. The second tool, the ADCP, is floated across the cross section, secured by a cable, as it gathers three dimensional velocity data continuously through a wireless connection to a nearby laptop. As the ADCP floats across a section, it measures velocity and depth using acoustic Doppler technology. After the cross section is completed, the velocity distribution of the cross section is obtained along with an estimate of the integrated discharge for that cross section.

The research site was monitored so that outside interferences were minimized. That is, signs were posted to avoid human caused movement of the channel bed, and when any artificial movement was found, such as hand-made dams, they were removed immediately.

4.3 Geophysical Study of Previous Channel Alignment

Geophysical investigation methods were used to determine the historic channel location in the vicinity of the study site. Around the 1970's White Deer Creek was relocated to its current position, as mentioned in the site description (Section 2). Using aerial photography and a grid of gravity measurements, the floodplain was investigated for signs of a previous channel. Gravity measurements indicate the relative density of material lying beneath the measurement location. Using a suspended mass on a spring, the gravimeter measures the displacement of the mass and provides an average gravity value at each measurement location. A filled in channel would be a less dense zone of fill, with lower gravity values, rather than the existing material that makes up the floodplain which has had a longer undisturbed time to settle.

4.4 Physical Modeling

Physical modeling experiments were completed to investigate the impact of cross vanes on the natural flow through a channel. The use of a sediment filled flume allowed the possibility to test various channel and structure configurations and take measurements that may not be possible in the field during a large storm. Frequent velocity measurements and photographs provide time-based tracking of changes occurring within the channel during a high flow event. The physical modeling was conducted in the basement of the O'Leary building on the Bucknell University campus. This facility includes a 12.2 meter long by 2.4 meter wide geomorphic flume with an adjustable slope, sediment recirculation capabilities, and an overhead rainfall system. A trapezoidal sand-bed channel was designed with two rock cross vane restoration structures, along with a removable bridge as seen below in Figures 7a and 7b. Several separate runs were designed by changing parameters within the experiment between each run, such as upstream sediment loading and bridge presence. The rock cross vane structures were placed 1.5 meters upstream and downstream from the bridge location (measured to the center of the cross vane). This location was chosen by scaling down the field site at White Deer Creek. Several constraints were placed on the scale of the physical model that did not allow a completely scaled physical model of the White Deer Creek study site. These include: sediment size, channel slope, and sediment recirculation rate.

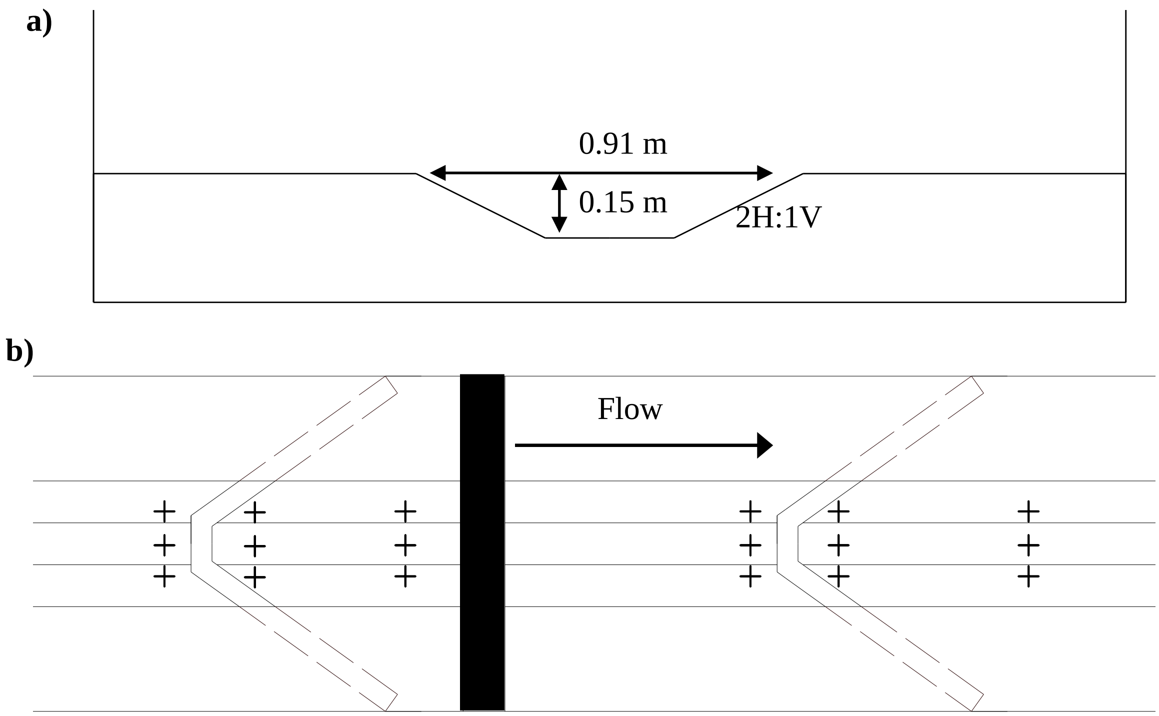


Figure 7 – Cross-section (a) and Top view (b) of the channel design used for the experiment (+ signs indicate a velocity measurement location, black zone indicates the removable bridge location).

4.5 Mathematical Modeling –Hydrology

The Hydrologic Modeling System, HEC-HMS (Scharffenberg and Fleming 2010), is a well known model commonly used to model the hydrology of a given watershed. The model was developed by the United States Army Corps of Engineers, Hydrologic Engineering Center and is available, at no cost, on their website. The model is designed to simulate precipitation-runoff processes of watershed systems. HEC-HMS was needed to estimate the hydrograph of a storm that passed over the White Deer Creek watershed on October 1st 2010 and provide a peak flow value to use for further research because White Deer Creek does not have a discharge gaging station. The model also was chosen due to its familiarity for those involved in design of bridge hydraulics and stream restoration structures.

HEC-HMS solves for a direct runoff hydrograph by using a discrete representation of the excess precipitation. A loss method is applied to known total precipitation values to determine the excess precipitation that becomes runoff. Using the convolution equation (Equation 1), a storm runoff hydrograph ordinate is calculated at each time step from an incremental excess precipitation value and the value of the estimated unit hydrograph ordinate. The baseflow estimate is added to the storm runoff hydrograph to develop a total hydrograph. The Snyder Unit Hydrograph (Snyder UH) method was selected for use because it was created for ungaged watersheds such as White Deer Creek, specifically in the Appalachian Highlands region of the eastern US. A recession baseflow method was utilized for the baseflow method and an initial and constant loss method was used for determining losses. These methods were chosen because the parameters used in the each method could be estimated using known watershed characteristics.

$$Q_n = \sum_{m=1}^{n \leq M} P_m U_{n-m+1} \qquad \text{Equation - 1}$$

Where:
Q_n = storm hydrograph ordinate
P_m = rainfall excess depth at interval m to m+1
U_{n-m+1} = UH ordinate at interval n-m+1
M = total number of discrete rainfall pulses

The Snyder UH method uses lag, peak flow, and total time base as the important components defining a UH. In 1938, Snyder (Scharffenberg and Fleming 2010) found a relationship to calculate the UH peak flow, Equation 2, from collecting field measurements from gaged watersheds.

$$\frac{U_p}{A} = C\frac{C_p}{t_p} \qquad \text{Equation - 2}$$

Where:

U_p = peak of standard UH (cms for SI or cfs for imperial)
(Standard UH has relationship $t_p = 5.5t_r$, t_r = rainfall duration)

A = watershed drainage area (km^2 for SI or mi^2 for imperial)

C_p = UH peaking coefficient

t_p = time to peak (hrs)

C = conversion constant (2.75 for SI or 640 for imperial)

After calculating the peak and time to peak of the UH, HEC-HMS uses this information to find an equivalent UH using the Clark method to calculate the times and values of all other UH ordinates. The Clark method is a linear reservoir model that represents the storage processes in the watershed. The model is based on the continuity equation (Equation 3).

$$\frac{dS}{dt} = I_t - O_t \qquad \text{Equation - 3}$$

Where:

dS/dt = time rate of change of water in storage at time t

I_t = average inflow to storage at time t

O_t = average outflow to storage at time t

The storage at time t, Equation 4, is then combined with Equation 3 and solved using a finite difference approximation to yield Equation 5 that is used to calculate the outflow at each time step. The average outflow at time t used for each UH ordinate is then calculated by averaging the outflow value at time t with the outflow value from the previous time step.

$$S_t = RO_t \quad \text{Equation - 4}$$

Where:

S_t = storage at time t

R = constant linear reservoir parameter

$$O_t = C_A I_t - C_B O_{t-1} \quad \text{Equation - 5}$$

Where:

C_A, C_B = routing coefficients

O_{t-1} = average outflow to storage at time t-1

4.6 Mathematical Modeling – Hydraulics and Sediment Transport

Several mathematical models for hydraulic and sediment transport processes were investigated to determine their applicability to the research needs. An applicable model had to have two capabilities: 1) The model must be able to represent and model a bridge crossing using the appropriate bridge hydraulics, 2) The model must have the capacity to simulate cobble-gravel sized sediment transport in a stream reach.

After gathering several models, they were further narrowed by comparing the modeling strengths and weaknesses. The differences between uncoupled (1-D), semi-coupled (1.5-D), and fully coupled (2-D/Finite Element) models result in advantages and disadvantages when selecting one over another. For example, a 1-D model has the advantage of commonly being easier to create and run compared to the semi/fully coupled models. But a 1-D model does not simultaneously solve both the flow equation and sediment continuity equation like a fully coupled model does, rather it first solves for a hydraulic solution which it uses to iteratively solve for a sediment solution. Although all three types had positives and negatives, many researchers agree that an uncoupled model runs into problems dealing with the sediment boundary conditions used within the model when stream conditions are rapidly changing (Lyn 1987; Holly and Rahuel 1990).

Others have shown that a semi-coupled sediment transport model predicts transport very similar to a fully coupled model (Kassem and Chaudhry 1998), and there is some agreement that a semi-coupled model can be used with reasonable success (Holly et al. 1990; Bhallamudi and Chaudhry 1991).

HEC-RAS is the model commonly selected to predict one-dimensional hydraulics and bridge hydraulics, and is accepted by FEMA for floodplain delineation. The HEC-RAS model is obtained online at the United States Army Corps of Engineers, Hydrologic Engineering Center website. This model is free to the public and was recently updated to include sediment modeling (Brunner 2002). The HEC-RAS model is the most commonly-used model in engineering design affecting rivers or streams and is therefore very accessible to design engineers. Because of its wide familiarity with people involved in the design of bridges and its wide acceptance for these types of projects, it was ideal to include in the mathematical modeling of White Deer Creek. HEC-RAS can handle bridge hydraulics effectively and has many commonly used methods available for predicting sediment transport potential within a stream reach, such as Ackers and White, Meyer Peter and Muller, and Yang equations.

The solution method in HEC-RAS calculates water surface elevations between cross sections using the standard step method. This involves iteratively solving the energy equation, Equation 6, and the energy head loss equation, Equation 7, which takes into account both the friction and any contraction or expansion losses that occur so that the energy is conserved between the two cross sections. The energy equation provides computational ease when longer sections are analyzed, such as the relatively long sections of channel between each HEC-RAS cross section.

$$Z_2 + Y_2 + \frac{a_2 V_2^2}{2g} = Z_1 + Y_1 + \frac{a_1 V_1^2}{2g} + h_e \qquad \text{Equation - 6}$$

Where:
Z_1, Z_2 = elevation of the main channel bed
Y_1, Y_2 = depth of water at cross sections
V_1, V_2 = average velocities (total discharge/total flow area)
a_1, a_2 = velocity weighting coefficients
g = gravitational acceleration
h_e = energy head loss

$$h_e = L\bar{S}_f + C\left|\frac{a_2 V_2^2}{2g} - \frac{a_1 V_1^2}{2g}\right| \qquad \text{Equation - 7}$$

Where:
L = discharge weighted reach length based on overbanks and main channel
C = expansion or contraction loss coefficient
$\bar{S}_f$ = representative friction slope between two sections

The iterative process starts by estimating a water depth for the upstream cross section if the flow is subcritical. If the flow is supercritical, then the downstream cross section depth is the starting point for the standard step calculation. The water depth is used to calculate the total conveyance (Equation 8) and velocity head, and then to calculate the average friction slope, $\bar{S}_f$.

$$K = \frac{C}{n} A R^{2/3} \qquad \text{Equation - 8}$$

Where:
K = conveyance for the section (m^3/s for SI or cfs for imperial)
n = Manning's roughness coefficient for the section
A = flow area for the section (m^2 for SI or ft^2 for imperial)
R = hydraulic radius for section (area / wetted perimeter, m for SI or ft for imperial)
C = Constant (1 for SI or 1.49 for imperial)

Using these values, Equation 7 is used to calculate the energy head loss between the sections. Then using Equation 6, a depth for the second cross section is calculated and compared with the initial estimated depth value. These steps are repeated until the calculated depth at the next section (upstream if subcritical or downstream if supercritical flow) is within 0.003 meters of the estimated value.

To model a sediment transport event, HEC-RAS uses a quasi-unsteady flow series. A quasi-unsteady flow series is created from a storm hydrograph by dividing the hydrograph into flow duration steps, in which the flow, stage, temperature, and sediment loading are considered constant. The flow durations are further broken down into computation increments, in which the bed elevation and hydrodynamics are updated after each increment. Therefore, rather than modeling a constantly changing flow hydrograph, HEC-RAS models a constant flow over the flow duration, and only allows bed elevation and hydrodynamics to change between each computational increment. This approach helps to increase model stability when simulating sediment transport events.

Between each cross section, HEC-RAS solves for sediment continuity using Equation 9, the Exner Equation. The Exner equation is used by first calculating a sediment transport capacity within the active layer of the channel based on the current hydrodynamics of the cross section. The active layer is the depth of sediment that moves during a sediment transport event capable of moving the largest diameter sediment found in the channel bed. The transport capacity of the section is then compared to the sediment supply input to that section. If the supply is greater than the capacity aggradation must occur within the section; and if the capacity is greater than the supply there must be erosion. The bed elevation is adjusted to account for the surplus or deficit in sediment load.

$$(1-\lambda_p)B\frac{\partial \eta}{\partial t} = -\frac{\partial Q_s}{\partial x}$$ Equation - 9

Where:
B = channel width
η = channel elevation
λ_p = active layer porosity
t = time
x = distance
Q_s = transported sediment load

The sediment transport capacity, or how much sediment the water can move given an unlimited sediment supply, is determined by first calculating a transport potential for each sediment size found in the bed. The Yang sediment transport equations (Equations 10 and 11) were used to calculate the transport potential. Equation 10 is used if the sediment size is less than 2 mm, and Equation 11 is used if the sediment size is greater than or equal to 2 mm. The Yang equations are dimensionless, empirical equations developed using a large amount of laboratory data along with a smaller sample of field data. Sediment transport potential for a certain size sediment grain class is determined based on the excess stream power available in the section. Stream power is the product of velocity and shear stress. Excess stream power is the amount of stream power that exceeds the critical stream power that is needed to move the sediment. The equation is useful for estimating the transport potentials of both sands and gravels and, based on research at a similar site in north-central Pennsylvania (Newlin 2007), has been found to simulate a transport event better than other equations.

$$\log C_t = 5.435 - 0.286\log\frac{\omega d_m}{\nu} - 0.457\log\frac{u_*}{\omega} + \left(1.799 - 0.409\log\frac{\omega d_m}{\nu} - 0.314\log\frac{u_*}{\omega}\right)\log\left(\frac{VS}{\omega} - \frac{V_{cr}S}{\omega}\right) \quad \text{Equation - 10}$$

$$\log C_t = 6.681 - 0.633\log\frac{\omega d_m}{\nu} - 4.816\log\frac{u_*}{\omega} + \left(2.784 - 0.305\log\frac{\omega d_m}{\nu} - 0.282\log\frac{u_*}{\omega}\right)\log\left(\frac{VS}{\omega} - \frac{V_{cr}S}{\omega}\right) \quad \text{Equation - 11}$$

Where:
C_t = total sediment concentration (ppm)
d_m = median particle diameter
ω = particle fall velocity
u_* = shear velocity
V = average channel velocity
S = energy gradient

Once the transport potential for all sediment sizes is found, Equation 12 is used to calculate a transport capacity for the section considering the size distribution of the sediment on the channel bed.

$$T_c = \sum_{j=1}^{n} \beta_j T_j \quad \text{Equation - 12}$$

Where:
T_c = total transport capacity
n = number of grain size classes
B_j = percentage of the active layer composed of material in grain size class “j”
T_j = transport potential computed for the material in grain size class “j”
j = incremental value for each grain size class

Based on the model review, another model seemed like an ideal candidate for multi-dimensional modeling of White Deer Creek. The second model is the two dimensional model FESWMS, finite element surface water modeling system. FESWMS adds more complexity into the modeling approach with two dimensional hydraulics and sediment transport in a finite element approach. This means that FESWMS will model the stream as a system of node points that make up a finite element mesh, rather than a set of cross sections, with averaged conditions, crossing the stream along its length as with the HEC-RAS model. FESWMS was developed by the Federal Highway Administration to simulate the movement of water and non-cohesive sediments in rivers estuaries and coastal waters (Froehlich 2003). Like the HEC-RAS model, FESWMS can handle bridge hydraulics and sediment transport appropriately for the research needs. The Surface Modeling Software (SMS) was used for pre-processing of the input data to FESWMS and for post-processing FESWMS results. SMS provides a smooth graphical user interface that runs with several models, FESWMS included.

FESWMS uses two dimensional depth-averaged flow equations to describe the flow at each finite element node. These flow equations are developed by integrating the three dimensional conservation of mass, Equation 13, and the three dimensional momentum equation in the x and y directions (Equations 14 and 15, respectively) with respect to the vertical axis. The momentum equation is preferable to the energy equation when modeling smaller channel sections, such as those modeled using a finite element mesh in FESWMS, due to the ability to neglect local boundary shear stresses in these sections.

$$\frac{\partial z_w}{\partial t} + \frac{\partial q_1}{\partial x} + \frac{\partial q_2}{\partial y} = q_m \qquad \text{Equation - 13}$$

Where:

z_w = water surface elevation

q_1 = unit flow rate in the x direction ($q_1/H = U$, depth averaged flow in x dir)

q_2 = unit flow rate in the y direction ($q_2/H = V$, depth averaged flow in y dir)

q_m = mass inflow/outflow rate per unit area

H = water depth

$$\frac{\partial q_1}{\partial t} + \frac{\partial}{\partial x}\left(\beta\frac{q_1^2}{H} + \frac{1}{2}gH^2\right) + \frac{\partial}{\partial y}\left(\beta\frac{q_1 q_2}{H}\right) + gH\frac{\partial z_b}{\partial x} + \frac{H}{\rho}\frac{\partial p_a}{\partial x} - \Omega q_2 + \frac{1}{\rho}\left[\tau_{bx} - \tau_{sx} - \frac{\partial(H\tau_{xx})}{\partial x} - \frac{\partial(H\tau_{xy})}{\partial y}\right] = 0$$

Equation - 14

$$\frac{\partial q_2}{\partial t} + \frac{\partial}{\partial x}\left(\beta\frac{q_1 q_2}{H}\right) + \frac{\partial}{\partial y}\left(\beta\frac{q_1^2}{H} + \frac{1}{2}gH^2\right) + gH\frac{\partial z_b}{\partial y} + \frac{H}{\rho}\frac{\partial p_a}{\partial y} + \Omega q_1 + \frac{1}{\rho}\left[\tau_{by} - \tau_{sy} - \frac{\partial(H\tau_{yx})}{\partial x} - \frac{\partial(H\tau_{yy})}{\partial y}\right] = 0$$

Equation - 15

Where:

β = isotropic momentum flux correction coefficient (account for variations in the velocity in the vertical direction)
g = gravitational acceleration
ρ = water mass density
p_a = atmospheric pressure at the water surface
Ω = coriolis parameter
τ_{bx}, τ_{by} = bed shear stresses acting in the x and y direction
τ_{sx}, τ_{sy} = surface shear stresses acting in the x and y direction cause by wind
τ_{xx}, τ_{yy}, τ_{xy}, τ_{yx} = shear stresses caused by turbulence, where τ_{xy} is shear stress in the x direction acting a plane perpendicular to the y direction

Using Equations 14 and 15, FESWMS calculates the water surface elevation at each node to determine if the node is wet or dry (i.e. if the water surface elevation is greater than the bed elevation at the node). If the element is considered wet, it is 'turned on' and considered in all other calculations, such as velocity and shear stress, otherwise it is 'turned off'.

To model sediment transport, FESWMS uses a two dimensional depth averaged sediment transport continuity equation, Equation 16. The equation is transformed from the x and y direction into the s, or streamwise direction, because it is assumed that sediment is only transported in the direction of the water flow.

$$(1 - \eta_s)\frac{\partial z_b}{\partial t} + \sum_i \frac{\partial q_{si}}{\partial s} = 0$$ Equation - 16

Where:
η_s = porosity of the bed material
q_{si} = volumetric transport rate of a single sediment size class
z_b = bed elevation
s = refers to the streamwise direction

Similarly to HEC-RAS, FESWMS first calculates the potential sediment transport rate for each individual grain size specified in the bed using a user-specified sediment transport equation. The Yang equations also were selected in the FESWMS model for the study site. FESWMS then calculates the total sediment transport rate based on percentages of each grain size in the bed. Using this total and Equation 16, FESWMS then balances the sediment inputs and outputs at each node point throughout its finite element mesh. Similar to the Exner equation for sediment continuity used in the HEC-RAS model, if the sediment supply is greater than the capacity for sediment transport, the bed elevation, z_b, is increased according to Equation 16.

Both HEC-RAS and FESWMS require several basic inputs to create a working model: geometric data, steady or unsteady flow data, and sediment data. For each surveyed data set at White Deer Creek, a sub-model was created based on the elevations gathered in the survey. Therefore, a total of five separate geometric sub-models were used in both HEC-RAS and FESWMS identified by the date of the field survey. Each sub-model was then calibrated with a data set that was collected and relevant to that sub-model. Relevant data included any hydraulic or sediment measurements and any observations that were made of high water marks or new erosion or deposition area at the time the survey was completed. The hydraulic calibration process for each sub-model was similar and involved changing the Manning's roughness, or n value. The process began by estimating an n value for the different segments of the reach and running the model under an observed flow. Next, by comparing the results from the model to measured or observed values in the field, adjustments were made to the n values to match the field conditions more closely. Once a sub-model matched the relevant data closely, the model was then tested against a second set of measured field data for verification. This process increased the confidence in the model parameters that were used and the ability of the model to predict future events.

Once hydraulic calibration was completed, the sediment transport models were created. To create each, the sediment size distribution measured in the creek was entered into the model to provide the necessary sediment size data. An equilibrium sediment load boundary condition was utilized during the sediment modeling for both models which sets the incoming sediment load to the creek equal to its transport capacity. When performing a sediment transport analysis, the transport equation used in both models was the Yang equation for sand and gravel. These equations were chosen based on the sediment sizes found in White Deer Creek and also were suggested in literature to help prevent model instability issues (Froehlich 2003). The author states that this equation had the highest level of successful runs while modeling sediment transport in FESWMS.

5. Results and Discussion

5.1 Watershed and Channel Investigation

A total of five segments of White Deer Creek were investigated by field observation. These sites were chosen based mainly on access to the creek. Each individual investigation is highlighted within the entire watershed of White Deer Creek in Figure 8 (Image adapted StreamStats in Pennsylvania).

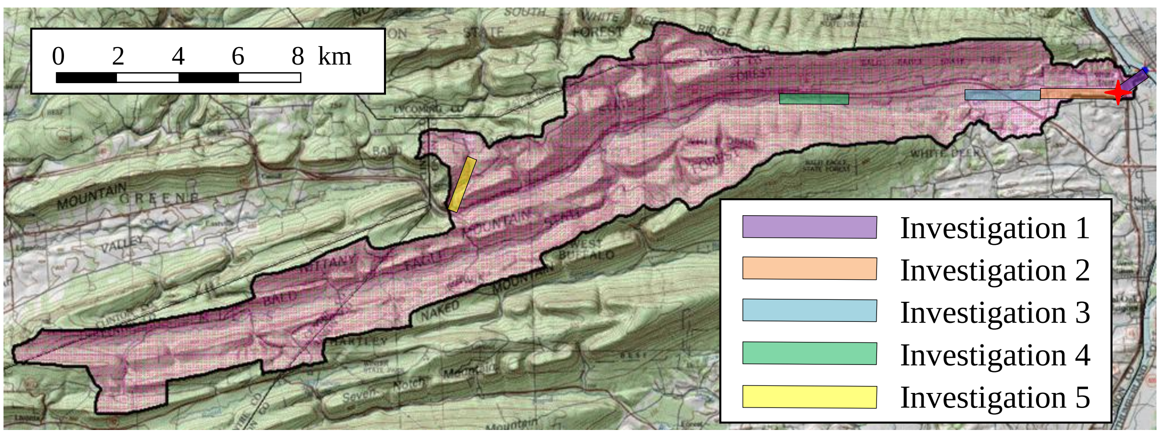


Figure 8 – White Deer Creek watershed along with highlighted locations of each investigation (Red + mark indicates the location of the research site).

The first investigation looked at the section of White Deer Creek between the research site and the Susquehanna River. As the distance to the river decreased, the sediment size of the channel bed also decreased, until eventually the channel bed was completely covered in sand sized particles. One pebble count was taken during this investigation around 305 meters downstream from the Old Route 15 Bridge crossing, at a riffle section in the channel. The second investigation was conducted within a 2.4 kilometer stretch upstream from the research site. It began upstream at a covered bridge and ended at the current US Route 15 Bridge near the study site. As shown in Figures 9a and 9b, sediment aggradation and erosion were visible at several locations throughout the reach. The sediment sizes in the creek were similar to what is found at the research site. In certain areas of this section, the creek flows directly over shale bedrock and no sediment is on the bed as shown in Figure 10.

(a) **(b)**

Figure 9 – Images taken during the second day investigation of White Deer Creek. (a) Aggradation between two older vegetated bars. (b) Eroded section of a bank (approx. 1.25 meters high).

Figure 10 – Image taken during the second day investigation of White Deer Creek showing the shale bedrock channel bed.

The third day investigation started 5.6 kilometers upstream of the site at a bridge crossing and ended at the covered bridge where the second investigation began. There were visible signs of recent erosion and deposition in the channel as shown in Figure 11. Three pebble counts were conducted at different locations throughout this reach. The first pebble count was taken at the beginning of the investigation, just downstream of a bridge crossing in a pool within the channel. The second was taken at the bridge crossing of Interstate 80 in a riffle section of the channel, and the third in a riffle section upstream of the covered bridge at the end of the investigation reach.

Figure 11 – Image taken during the third day investigation of White Deer Creek displaying a large aggradation bar angled across the span of the channel.

The fourth day investigation was conducted 8.9 kilometers upstream from the research site. The investigation passed over an existing dam used for a water supply reservoir and owned by PA American Water. This reach is experiencing many areas of erosion as shown in Figures 12a and 12b.

(a) **(b)**

Figure 12 – Images taken during the fourth day investigation of White Deer Creek. (a) Section of significant bank erosion. (b) Showing the sediment stored within the bank.

Two pebble counts were taken during the fourth investigation, one in a pool located at the beginning of the reach, the other at a power line crossing a riffle section located at the downstream end of the investigation reach.

The fifth investigation was the farthest upstream within the watershed. It was conducted 25 kilometers upstream from the research site on a tributary to the main stem of White Deer Creek. Figure 13 below shows a typical section of the creek at this location. In this area, the creek would disappear beneath the surface and reappear in very rocky sections farther downstream.

Figure 13 – Image taken during the fifth day investigation of White Deer Creek.

The results of the watershed investigation provide information regarding sediment sources, storage and processes occurring naturally in the channel. Figures 9, 11, and 12 show that transported sediment has come from sources such as bank erosion and active depositional bars throughout the channel and its floodplain. It was observed that sediment is stored within large bars in the channel and in smaller side channels that are only active during storm events. All pebble count data taken during the watershed investigation is summarized in a sediment size distribution plot, Figure 14. The sediment sizes follow the general trend that as you move farther upstream in the watershed, the sediment found in the streambed generally increases in diameter.

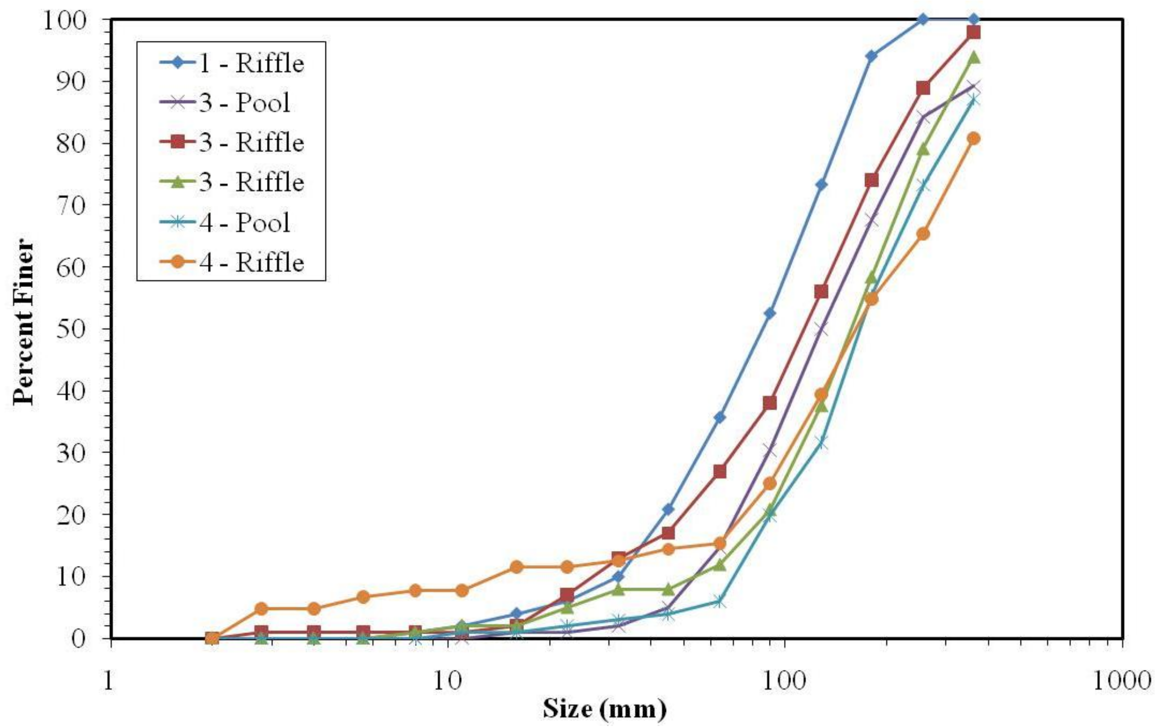


Figure 14 – Sediment size distribution plot for the pebble counts taken during the White Deer watershed investigation (Pebble counts are labeled according to reach number and location taken).

5.2 Monitoring and Data Collection at the Study Site

A total of five surveyed geometric data sets of White Deer Creek were completed. From the oldest to the most recent, the survey times were June 2009, June 2010, November 2010, June 2011, and March 2012. Each survey provided bed elevation data for cross sections in the vicinity of the study site at the time the survey was conducted. Figure 15 shows the bed profile of each surveyed data set. The bed elevations shown represent the elevations of the channel thalweg, or lowest bed elevation of each cross section. After the initial survey in June of 2009, metal pins were used to permanently mark the location of each measured cross section. All surveys were referenced to the elevation datum of permanent benchmark located on the wing-wall of the Old Route 15 Bridge.

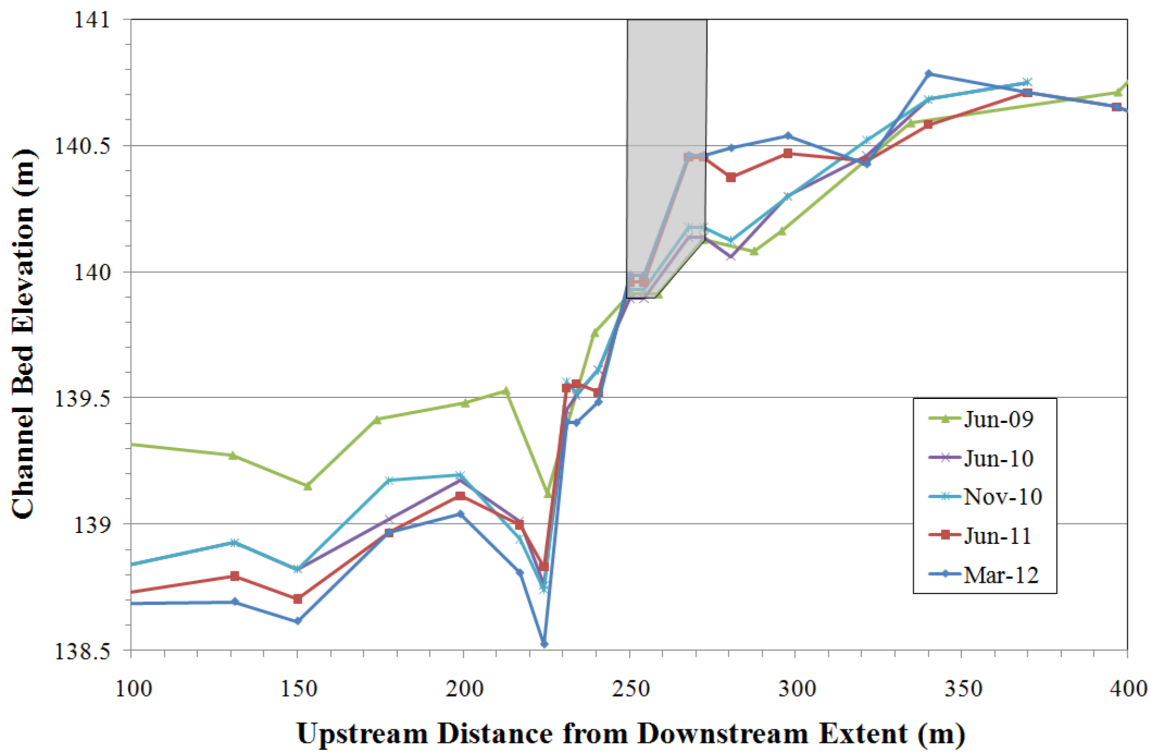


Figure 15 – Channel bed profile plot of each survey taken on White Deer Creek.

The channel cross sections were compared to detect more detailed changes between surveys. Figure 16 shows cross section data of White Deer Creek collected 55 meters downstream of the Old Route 15 Bridge crossing. Figure 17 shows cross section data collected 12 meters upstream of the Old Route 15 Bridge crossing.

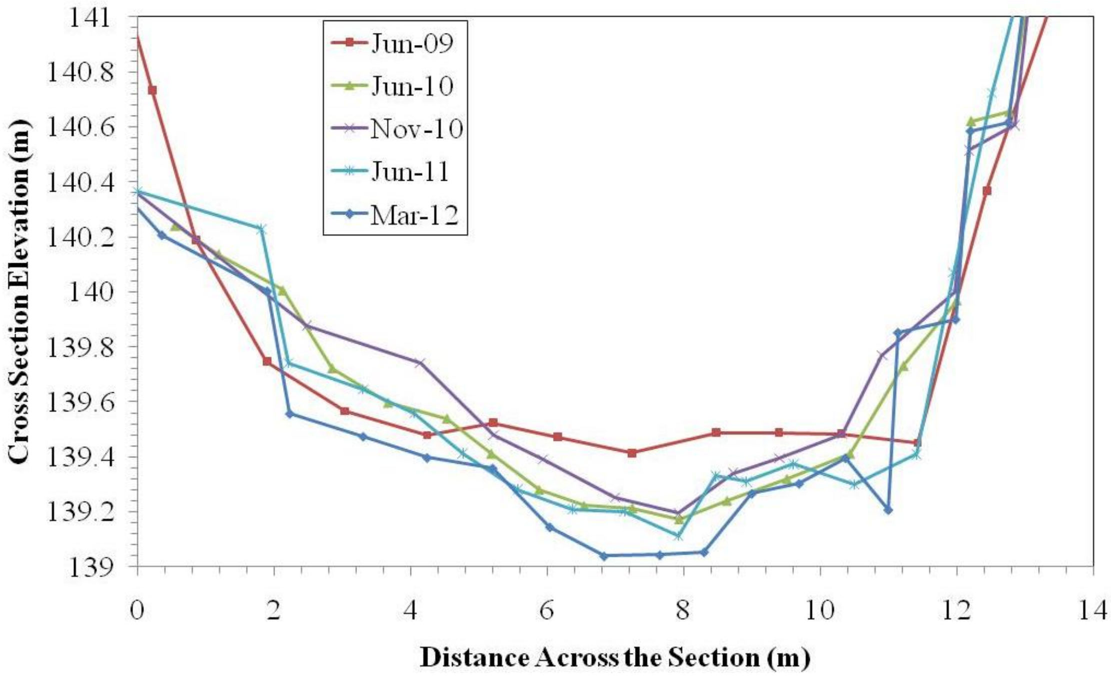


Figure 16 – Cross section data collected downstream of the Old Route 15 Bridge crossing.

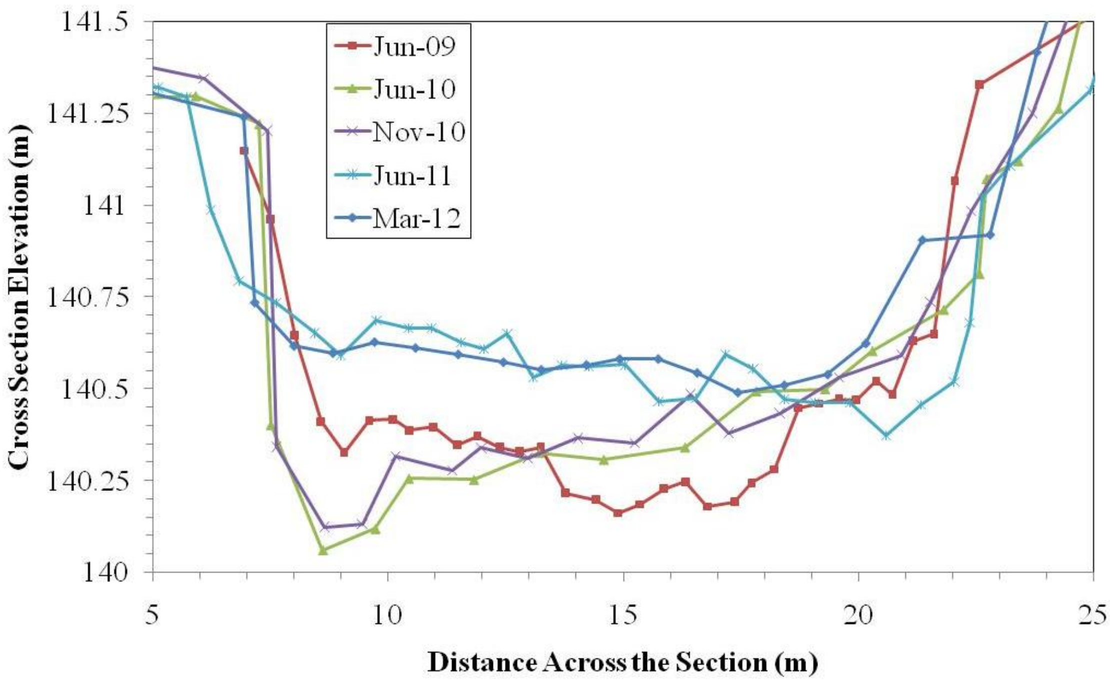


Figure 17 – Cross section data collected upstream of the Old Route 15 Bridge crossing.

A total of five pebble counts were taken throughout the study site during the research period, three of which were taken during 2010 and the remaining during 2011. The three taken during 2010 were spaced accordingly: (1) in a pool upstream of the Old Route 15 Bridge, (2) in a riffle between the bridge and the downstream structure, and the last (3) in a pool downstream of the downstream structure. The first of the 2011 pebble counts was taken in a riffle section directly beneath the Old Route 15 Bridge and the second was taken at the lower end of the research site located in a riffle section just downstream of a bend in the channel. All of the pebble count data is shown in Figure 18 along with an average of all pebble counts.

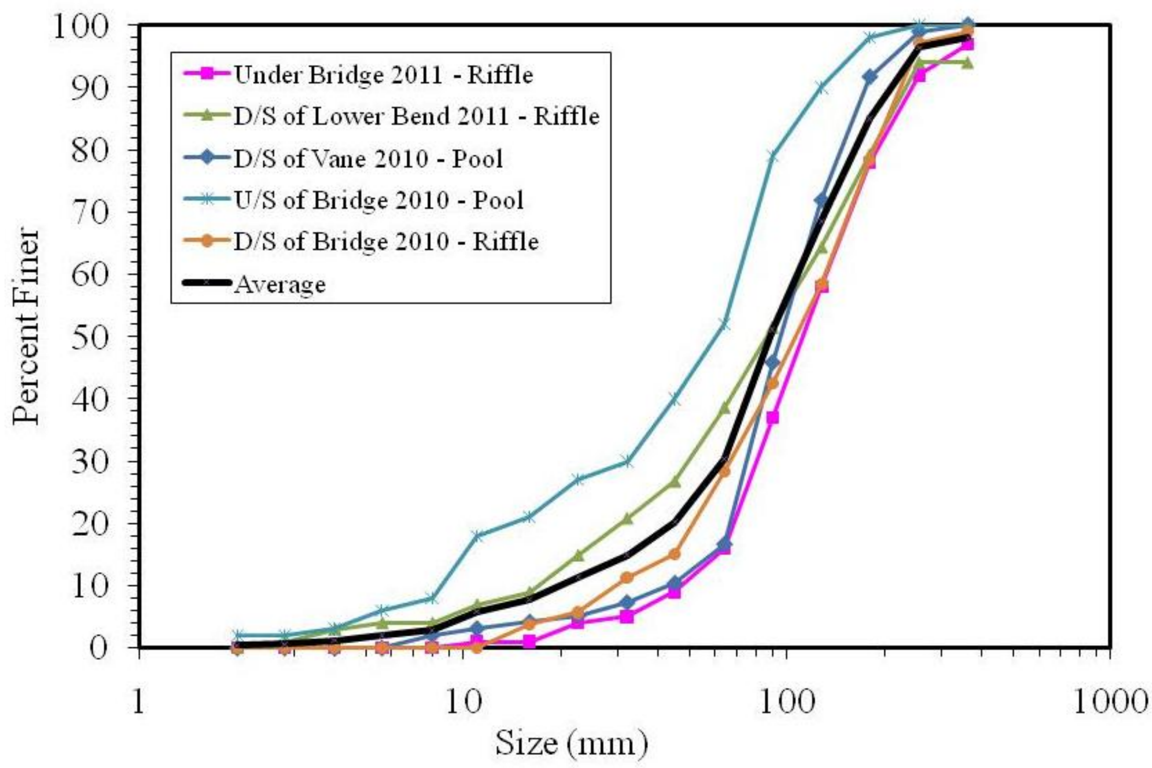


Figure 18 – Measured sediment size distributions at various locations on the White Deer Creek research site (Legend refers to the Old Route 15 Bridge).

The stream bed profiles in Figure 15 show significant channel erosion in the lower section of the research site as time progressed. For sections upstream of the Old Route 15 Bridge the opposite is shown, the channel has experienced aggradation. Figures 16 and 17 show the local areas in the cross sections that experienced the erosion and/or aggradation. Looking closer at Figure 16, the downstream cross section of the channel is becoming more channelized (narrower and deeper) as time progresses. The results match very well with visual observations of the site over the research period. The downstream channel has become much more incised and the bank stabilization riprap is falling into the channel after being undercut by the flow. On the other hand, observations in the section upstream of the bridge crossing show several areas where sediment bars are slowly growing in the channel.

As time progressed during the monitoring period, the channel experienced decreasing magnitudes of changes between each survey (Figure 15). Downstream of the US Route 15 Bridge, the largest change is between the first two surveys (June 2009 to June 2010). This indicates the channel may still have been experiencing the initial rapid response from the construction of the restoration project. However, upstream of the US Route 15 Bridge, the largest change in the channel thalweg is between the November 2010 and the June 2011 surveys.

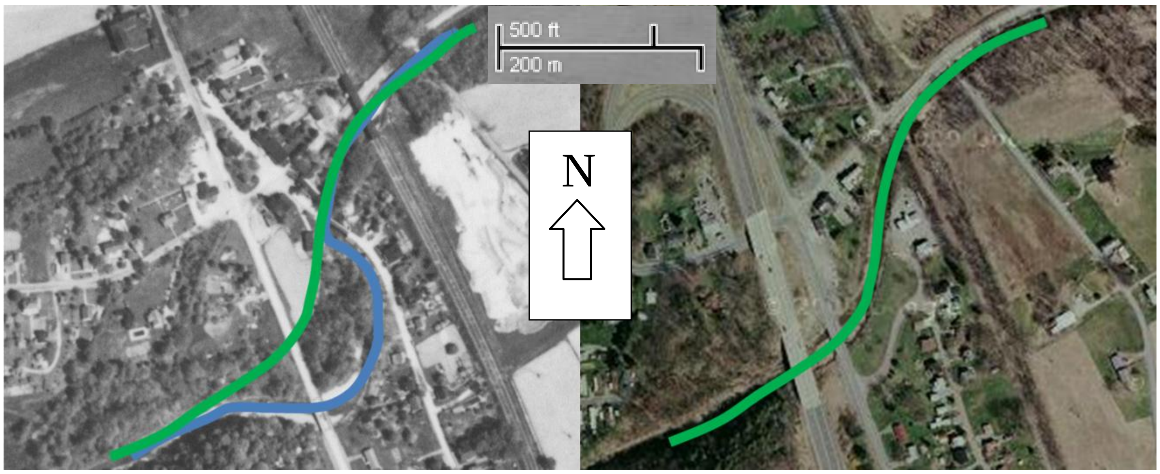
From the pebble counts in Figure 18, the d_{16}, d_{50}, and d_{84} transported bed material sizes White Deer Creek are 35, 85, and 180 mm respectively. The critical shear stresses associated with each sediment size (d_{16}, d_{50}, and d_{84}) are 34, 82, and 175 N/m^2, respectively. This means that in order for most of the active layer of the bed to be transported, the average bed shear stress in the stream must be at least 175 N/m^2.

Two scour chains were installed in July 2011 to help quantify the amount of sediment eroding or depositing on the creek bed. The first scour chain was installed in the center of the main channel between the Old Route 15 Bridge and the downstream restoration structure. The second scour chain was installed in the thalweg of the channel downstream of the downstream restoration structure. One measurement was recorded, using the upstream scour chain, during the fall of 2011 after a high flow event. The scour chain was completely buried under a minimum of 76 mm of sediment with several sections buried up to 230 mm deep. Conservatively, the measurements indicate deposition of around 1.42 cubic meters of sediment across the entire section. The second scour chain, installed beneath the downstream restoration structure, was not recovered after the storm had passed.

5.3 Geophysical Study of Previous Channel Alignment

By determining the location of the old channel, several geomorphic values of a stream, such as stream length and slope, can be determined and compared from the pre-straightened to the present channel. The section of White Deer Creek that is being monitored was relocated sometime between 1957 and 1972. Figures 19a and 19b show the relocation based on overlapping aerial photographs (obtained from the Penn Pilot Photo Centers website) and estimating the channel location. The collected gravity data from geophysical investigation methods help to locate the downstream extent of the channel relocation more precisely and verify the estimation from the aerial photographs.

The gravity measurements were setup in a 25 m x 25 m grid outlined in the yellow zone in Figure 20. The blue line represents the approximate channel location that was investigated. Figure 21 shows the gravity analysis results overlain on an image of the study area.



(a) **(b)**

Figure 19 – (a) 1957 aerial image of the White Deer Creek study site. Green line indicates current channel location, Blue indicates the historic alignment. (b) Recent aerial image of White Deer Creek with channel position outlined in green.

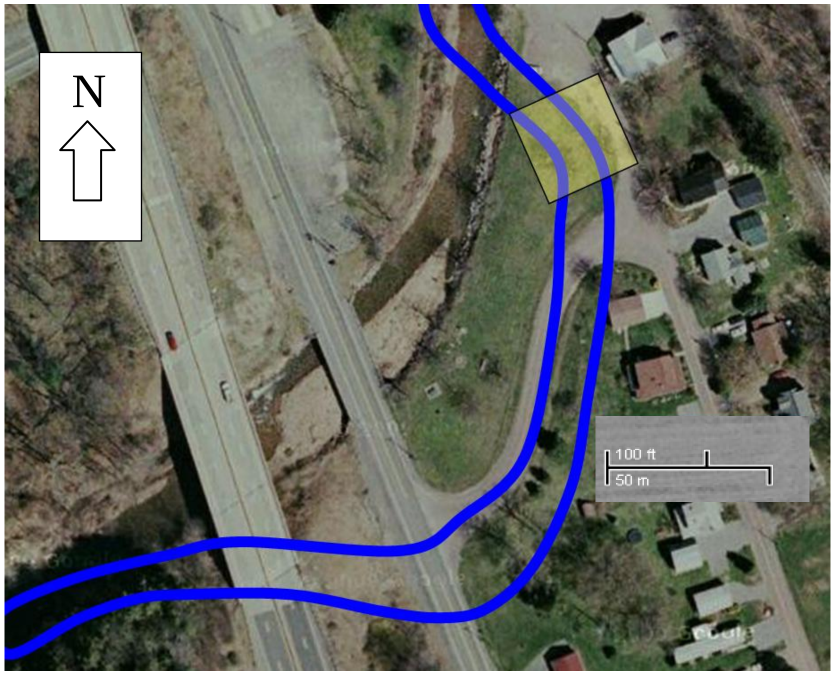


Figure 20 – Gravity data measurement location at the White Deer Creek study site.

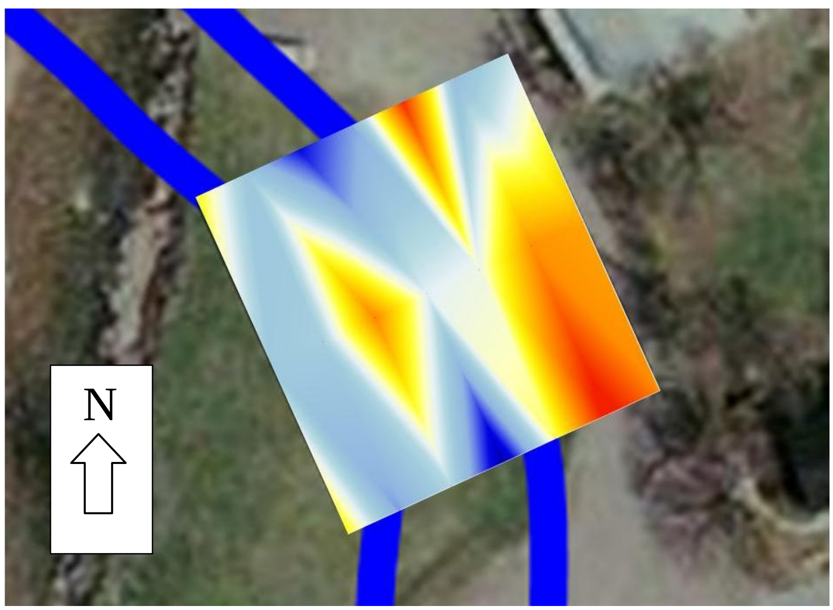


Figure 21 – Contour map of the gravity measurements taken at the White Deer Creek study site, refer to Figure 20 for location on site.

On the contour map of the gravity measurements in Figure 21, the blue area indicates a lower gravity, or a less dense subsurface than the red zones. When filling in an old channel, the likely backfill would be the excavated material from digging the new channel. As the new channel is created, the loose excavated sediments would be placed into the old channel bed. The old channel location would now be less dense than the nearby non-excavated floodplain. These results show a relatively low gravity area next to a higher gravity area as expected. The low density zone matches well with the approximated old channel location indicated on the overlapping aerial photos (Figure 19).

Based on the results from this channel alignment study, it is estimated that the total stream length of this section before the alteration was 373 meters. The current stream length is 267 meters which makes a total decrease in stream length of 106 meters. This was a 29 percent decrease in length which corresponds to a 40 percent increase in slope, assuming the downstream and upstream bed elevations stayed constant. Such a large increase in bed slope, and decrease in channel length can cause many different channel instability issues which explain why the channel has begun to cut into the outside banks of the bend, as well as experience aggradation/degradation at several locations along the reach. The straightened section also caused the sinuosity of the stream to drop 28 percent, from 1.63 to 1.17. These geomorphic parameter changes help to explain the channel response to instability that has been recorded throughout the research period.

5.4 Physical Modeling

The general physical modeling setup was discussed previously in the Methodology (Section 4.4). Table 1 shows a list and description of the specific trials conducted as the physical modeling experiments.

Table 1 – Description of the trial runs for the physical modeling experiments.

Trial	Description of Trial	Comments
1	No bridge, No sediment supply	Control Trial
2	No bridge, Very high sediment supply	Active sediment recirculation
3	No bridge, Moderate sediment supply	Sediment pile upstream
4	Bridge with a 0.3 m opening, No sediment supply	No wingwall on the bridge
5	Bridge with a 0.2 m opening, Moderate sediment supply	No wingwall on the bridge, Sediment pile upstream
6	Bridge with a 0.2 m opening, No sediment supply	Wingwalls built on the bridge at 45 degree angles

The initial setup for trial one is shown in Figure 22. The straight channel had two cross vanes in place and was run with no sediment loading at the upstream end of the channel. Figure 23 displays an image taken while measuring velocity as the trial was taking place, and Figure 24 displays the measured velocities in a contour plot. The contour plot shows velocity directions, indicated by the arrows, and magnitudes, indicated by arrow size and contour color. The black bold lines represent the locations of the cross vanes. The flow is roughly 76 mm deep and has eroded away the sloped banks to create a more rectangular channel. The velocity increases in the center of the channel downstream of the cross vanes and from looking at Figure 23, the flow does converge over the vane as intended by the design of the structure.

Figure 22 – Photograph of the initial setup for trial 1 (facing upstream).

Figure 23 – Velocity measurements taking place during trial 1 (facing upstream).

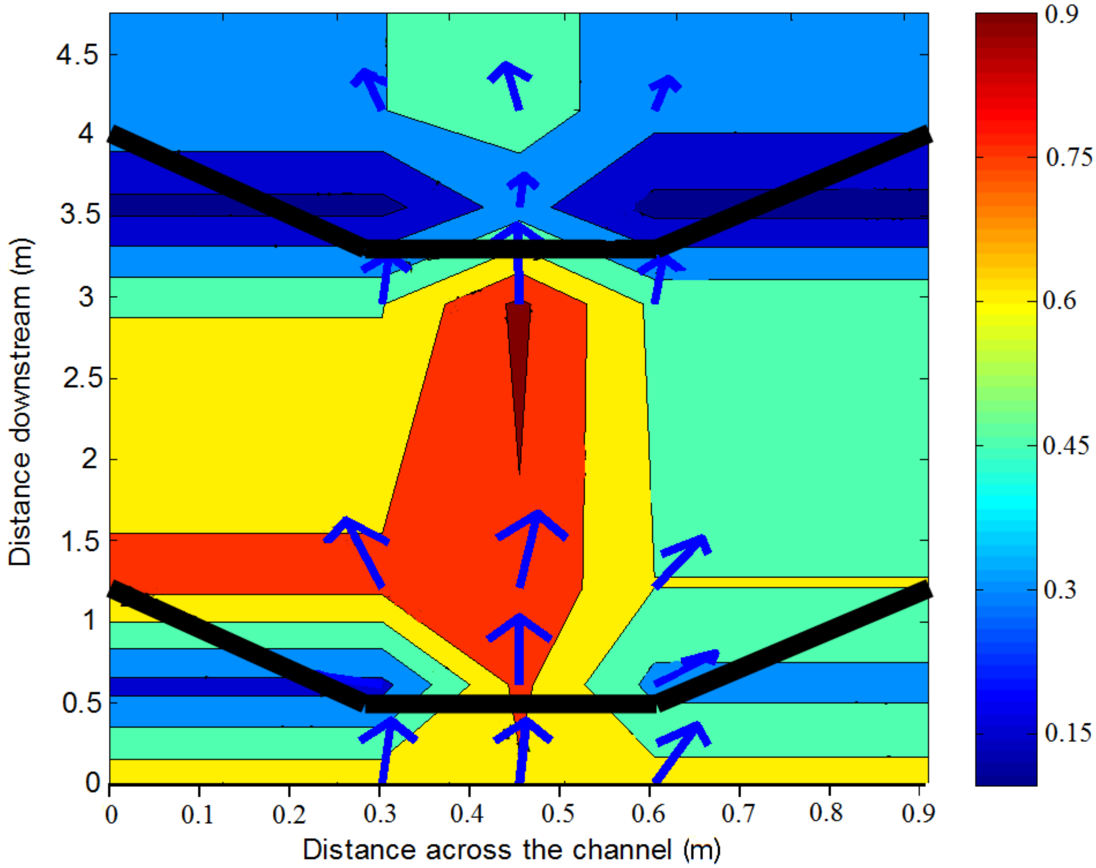


Figure 24 – Velocity directions/magnitudes for trial 1 (Arrow size is proportional to magnitude; Contours indicate velocity magnitude in m/s).

The setup for trial two was the same as trial one with two cross vanes on a straight channel. However, trial two was run with a high sediment loading rate at the upstream end of the channel. This was accomplished by using the sediment recirculation pump. The end result of the trial is shown in Figure 25. From looking at the resulting channel photograph, the channel had changed from a straight channel to a meandering channel. Both cross vanes were buried by sediment, and the channel had widened by up to 150 percent at several sections.

Figure 25 – Photograph of the resulting channel from trial 2 (facing upstream).

Trial three was very similar to the second trial except that a sediment pile was placed at the upstream end of the channel rather than having a constant sediment recirculation. The sediment pile was meant to simulate a pulse of sediment supply and to provide insight to how the nature of the sediment supply affects the burial of the vanes. The velocities in Figure 26 are very consistent and show expected patterns of flow near the structures. Figure 27 shows the upstream cross vane while trial three was taking place. The converging flow over the cross vane is clearly demonstrated in the photograph. In this scenario, the upstream vane was not buried and the downstream vane failed on the right side. The downstream vane failure is attributed to the erosion that occurred near the edges on the upstream side of the structure.

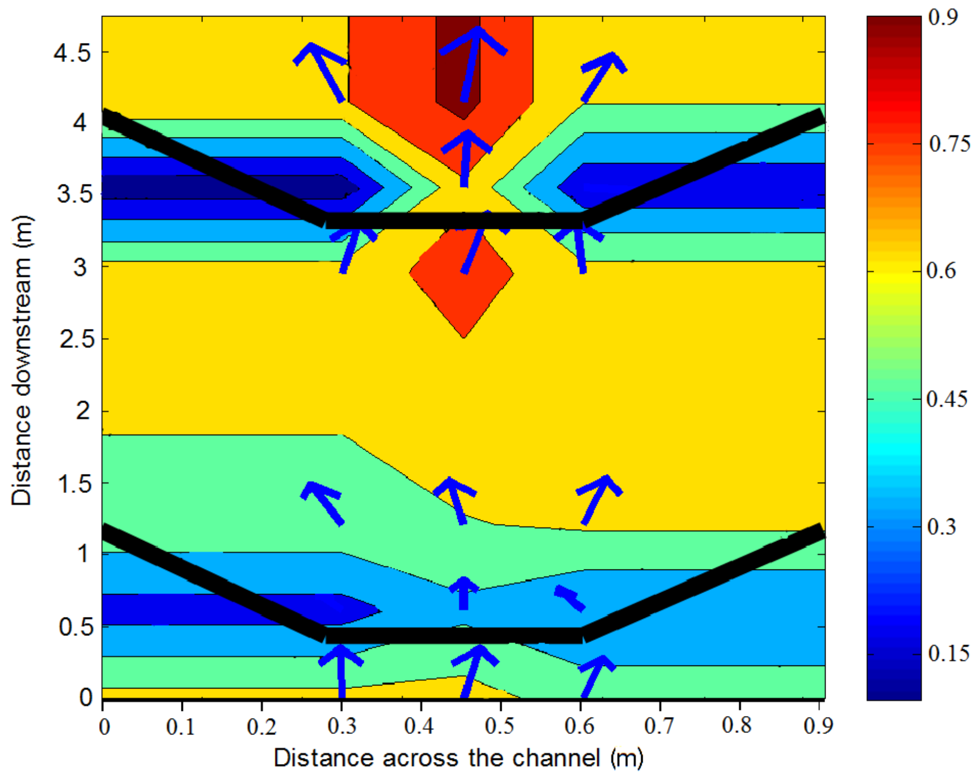


Figure 26 – Velocity directions/magnitudes for trial 3 (Arrow size is proportional to magnitude; Contours indicate velocity magnitude in m/s).

Figure 27 – Photograph of the upstream cross vane during trial 3 (flow towards bottom).

The fourth trial was the same as the first trial but included a bridge between the cross vanes. In this trial, the bridge had a 0.3 meter opening for the water to pass through, thus constricting it to a 0.3 meter rectangular channel rather than the original 0.91 meter wide trapezoidal channel. Figure 28 shows an image of the initial setup of the channel, Figures 29-31 show the results associated with this trial. The velocity contour plot (Figure 29) shows that near the downstream cross vane the velocities seem to skew to the left overbank, then back to the right. Evidence for this behavior is observed in Figures 30 and 31. Figure 30 shows that the upstream vane is in very good condition at the end of the trial run, but both the downstream vane (Figure 30) and the bridge (Figure 31) have failed. The flow had undercut the bridge which then allowed a small flow path around the right side of the bridge to form. This new flow path forced the water towards the left bank of the channel directly above the downstream cross vane. Resulting from this higher velocity and flow, the downstream vane was eroded and the large stones began to fall into the scour hole that was initially created. In the very beginning of the trial, backwater created by the bridge constriction was slightly evident but as soon as the bridge failed all signs of the backwater disappeared.

Figure 28 – Photograph of the initial setup for trial 4 (facing upstream).

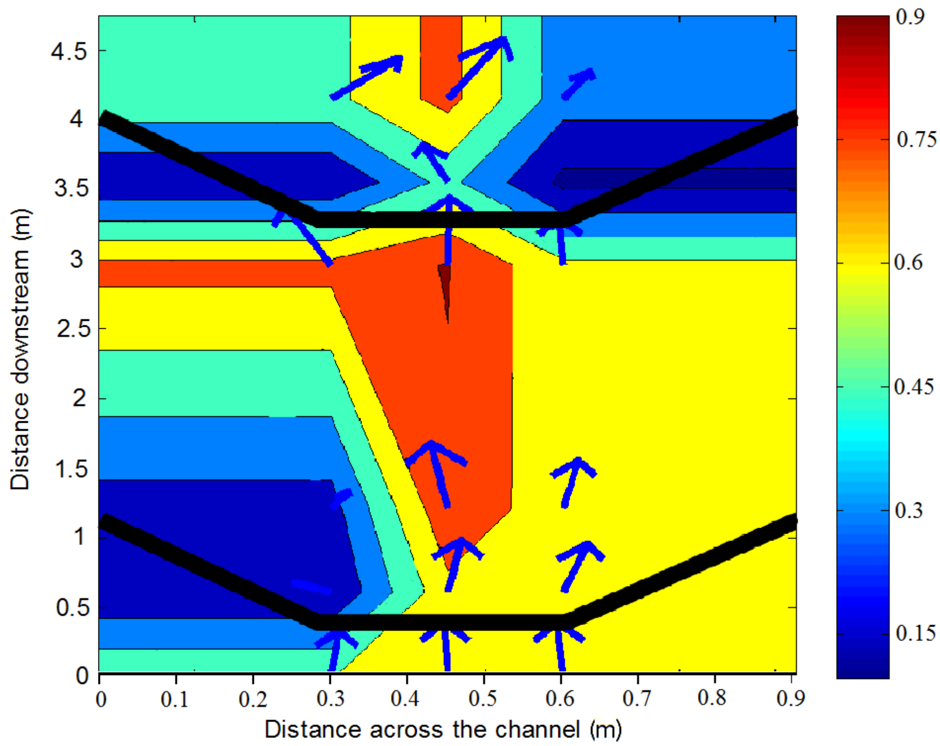


Figure 29 – Velocity directions/magnitudes for trial 4 (Arrow size is proportional to magnitude; Contours indicate velocity magnitude in m/s).

Figure 30 – Photograph of the upstream cross vane after trial 4 (facing downstream).

Figure 31 – Photograph of the downstream cross vane after trial 4 (facing downstream).

The initial setup of trial five was similar to trial four but the bridge now only had a 0.2 meter opening and also was supported beneath the cinderblocks by a slab of wood to prevent undercutting of the structure as occurred in trial four. The set up of trial five provided a more realistic representation of the bridge foundation. A moderate sized sediment pile also was located at the very top of the flume to provide a pulse supply of movable sediment for the channel to transport. Looking at the channel photos, Figures 32 and 33, the bridge again failed on its right side and forced the water to the left side of the downstream cross vane. This caused the left side of the cross vane to fail. One major difference between trials four and five was that the upstream vane was covered with sediment following the trial run.

Figure 32 – Photograph of the upstream cross vane after trial 5 (facing downstream).

Figure 33 – Photograph of the downstream cross vane after trial 5 (facing downstream).

Trial six required the addition of wing walls to the bridge to provide a smoother flow transition at the bridge and no sediment pile was placed at the upstream end of the channel. The wing walls were constructed at 45 degree angles out from the opening of the bridge. Again, the bridge had a 0.2 meter opening and was supported beneath it by a wooden plank. The resulting velocities shown in Figure 34 demonstrate a generally lower velocity upstream of the bridge compared to downstream. Figures 35 and 36 show photos of the upstream and downstream cross vanes, respectively, following trial six. The upstream vane shows some signs of burial, but not nearly as much as during trial five, and the downstream vane failed in the same location as before. The bridge did not fail this time, as it did previously, so the downstream vane failure is only a result of the erosion of the sand that supports the sides of the vane, in particular the left side of the downstream vane.

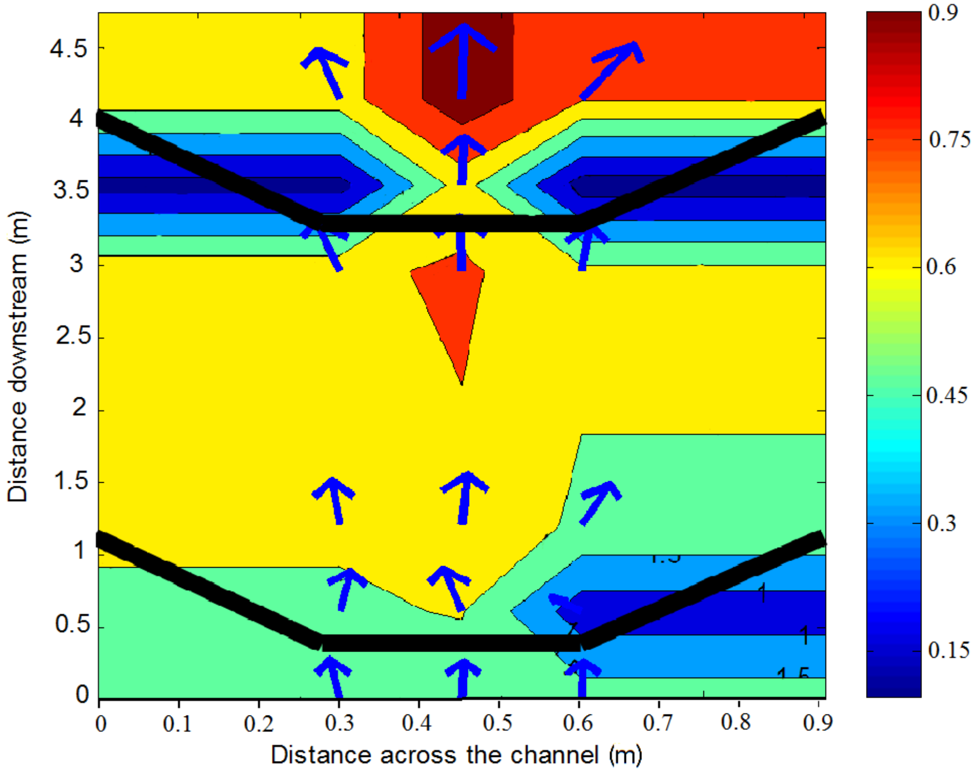


Figure 34 – Velocity directions/magnitudes for trial 6 (Arrow size is proportional to magnitude; Contours indicate velocity magnitude in m/s).

Figure 35 – Photograph of the upstream cross vane after trial 6.

Figure 36 – Photograph of the downstream cross vane after trial 6.

Several observations can be made as a result of the physical modeling trials. It is important to note that some of the behavior seen in the experiment, like the unstable banks and constantly failing structures, can only be accurately compared to a field site with similar characteristics, such as a sandy bed. Even so, the flow behavior in the vicinity of the cross vanes should remain similar regardless of the bed material. When the cross vanes did not fail, they were consistently channeling the flow toward the center of the channel and creating a deep scour hole directly downstream of each vane. Problems with a cross vane's ability to channel flow toward the center of the channel and create a scour hole arose when either the backwater effects or an upstream sediment supply was introduced during the trial. In both cases, the cross vanes were partially covered by sediment. In the case of the backwater, the lower channel velocities, which indicate lower shear stresses, did not support the transport of the supplied sediment load and the surplus sediment deposited in the region where the backwater effects were observed. In the case of the higher sediment load (trial two), the channel was overloaded with sediment and the supply was greater than the capacity along the entire experimental section. Problems also arose when the flow direction toward the downstream cross vane was altered from straight due to the bridge failure. The hydraulic performance of the rock cross vanes seemed to match the expected behavior according to design more closely when the flow was approaching the structure from straight upstream. As the approach flow skewed to one side or the other, the vane created less convergence in the center of the channel and failure was observed more frequently.

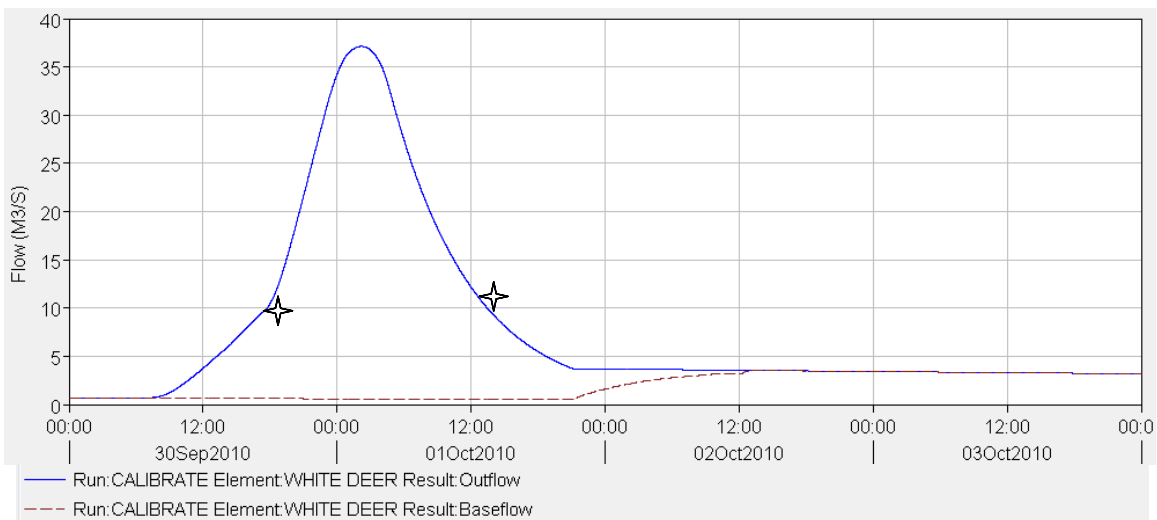
5.5 Mathematical Modeling –Hydrology

HEC-HMS was used to recreate the hydrograph resulting from a storm passing through the watershed. Using high water marks, two measured flow rates and times, and several parameters of the watershed, an HEC-HMS model was calibrated to predict the hydrograph for the October 1st, 2010 storm. Table 2 describes the calibrated HEC-HMS model created for the White Deer Creek watershed.

Table 2 – Model setup and parameters used to recreate a storm hydrograph for the White Deer Creek watershed.

Loss Method	Initial Loss (mm)	11.4
	Constant Rate (mm/hr)	7.6
	Impervious (%)	1
Baseflow Method	Baseflow (cms)	0.64
	Recession Constant	0.93
	Ratio to Peak	0.1
Snyder Unit Hydrograph	Peaking Coefficient	0.7
	Standard Lag (hr)	9.5
	Basin Coefficient	1.7

The initial loss was first estimated based on the percent of the watershed that was forested and the constant rate of loss was first estimated using soil types within the watershed. Adjustments to both values were made while calibrating the model to the measured data. Both final values are within acceptable ranges for a watershed similar to the White Deer Creek watershed. The baseflow was estimated based on a previously measured baseflow using a Marsh McBirney velocity meter. This measurement was taken in August 2010 and is a reasonable estimate for the baseflow for the October 1st storm. The recession constant and ratio to peak were estimated based on suggestions within the HEC-HMS reference manual (Scharffenberg and Fleming 2010), and because a recession baseflow method was used, these values were not changed during the calibration process. All Snyder UH parameters initially were estimated based on watershed characteristics. After calibration, these parameters fit within acceptable ranges of values based on the watershed characteristics. The high water marks and flow rates were recorded downstream of the Old Route 15 Bridge throughout the storm duration. The first measured flow rate was taken with the ADCP from the bridge on September 30th at 4:30pm and was 8.64 cms, and the second was taken on October 1st at 1:00pm and was 10.62 cms. Figure 37 below displays the predicted hydrograph obtained from HEC-HMS for the storm event on October 1, 2010 along with markers indicating the two measured flow rates. The hydrograph matches the measured times and flow rates with a maximum error of 0.3%. According to the calibrated HEC-HMS model, the peak flow of 37.12 cms arrived on October 1st at 2:10am. This peak flow also was calibrated using high water marks measured after the storm. The high water marks measured were high grass patches along the banks of the channel that had been forced down due to the flow above them. To do this, the peak flow was used to run a hydraulic analysis in a calibrated HEC-RAS model and the predicted water surface elevations were compared to the measured high water marks. The measured high water marks were consistently lower than the predicted water surface elevation, all being roughly 0.38 meters less. Although the values do not closely match, the consistent difference is reasonable due to the type of high water marks used. It is expected that the water surface be above the actual elevation of the grassy high water mark due to the grass needing the waters weight to force it down. The resulting hydrograph was then used further in the modeling of White Deer Creek.



Peak Discharge :	37.1 (M3/S)	Date/Time of Peak Discharge :	01Oct2010, 02:10
Total Precipitation :	95.50 (MM)	Total Direct Runoff :	17.70 (MM)
Total Loss :	77.81 (MM)	Total Baseflow :	5.75 (MM)
Total Excess :	17.70 (MM)	Discharge :	23.44 (MM)

Figure 37 – Resulting hydrograph from HEC-HMS for the October 1st, 2010 storm data (stars indicate flows measured with the ADCP).

5.6 Mathematical Modeling – Hydraulics and Sediment Transport

Several mathematical model comparisons were made to help understand the hydraulic and sediment transport processes occurring at the White Deer Creek study site as well as to investigate the model performance. The scenarios along with the models (HEC-RAS and FESWMS) used for them and the data they provided are listed in Table 3. Again, sub-model refers to the survey that was used to provide the spatial and elevation data to the model.

Table 3 – List of scenarios used in the mathematical model analysis of hydraulic and sediment transport processes.

Comparisons	Models Used	Sub-Models Used	Information from Model	Analysis
Channel Bed Shear Stress	HEC-RAS, FESWMS	All	Locations of high/low shear stress during high flow events	Determine locations of predicted deposition/scour
Channel Velocity	HEC-RAS, FESWMS	All	Velocity values during periods of high flows	Determine locations and direction of high velocity
Bed Elevation Change	HEC-RAS	June 2010 to Nov 2010	Changes in bed elevation before/during/after transport events	Determine areas of aggradation or scour

The shear stress comparison involved running all of the created sub-models and calculating the average bed shear stress throughout the channel. Afterwards, locations of high and low shear stress were investigated to look into the potential for sediment deposition/movement in each area. The different sub-models also were compared to determine if there were any changes in shear stresses from one sub-model to another. Particular areas of interest are locations near the restoration structures, locations where aggradation or scour is visible in the field, and locations around the Old Route 15 Bridge crossing. A similar analysis was done with the velocity throughout the channel to investigate potential scour zones.

5.6.1 Channel bed shear stress

The shear stress results from the two-dimensional FESWMS model are displayed in Figures 38 - 42 as contour plots. Contour plots were created with a two and ten year flow, 51.8 and 115.8 cms respectively, for each different survey. These flow values were estimated using the StreamStats software made available through the USGS website (StreamStats in Pennsylvania) which uses the flood frequency regression equations developed by Roland and Stuckey (2008) to estimate these flow rates. Based on regression equations for bankfull channel characteristics (Chaplin 2005), the bankfull discharge for White Deer Creek is estimated to be 34.5 cms which would translate to a recurrence interval of less than a two year storm. The two and ten year flows were chosen to investigate both a minor flood event, above the channel bank in certain areas, as well as a major flood event, flooding outside of the main channel banks. The restoration structures are likely to experience these flows during their design life. These moderate flows are more likely to be responsible for maintenance of the channel form than very low flows or larger less frequent floods because of their relative frequency and ability to move the bed material.

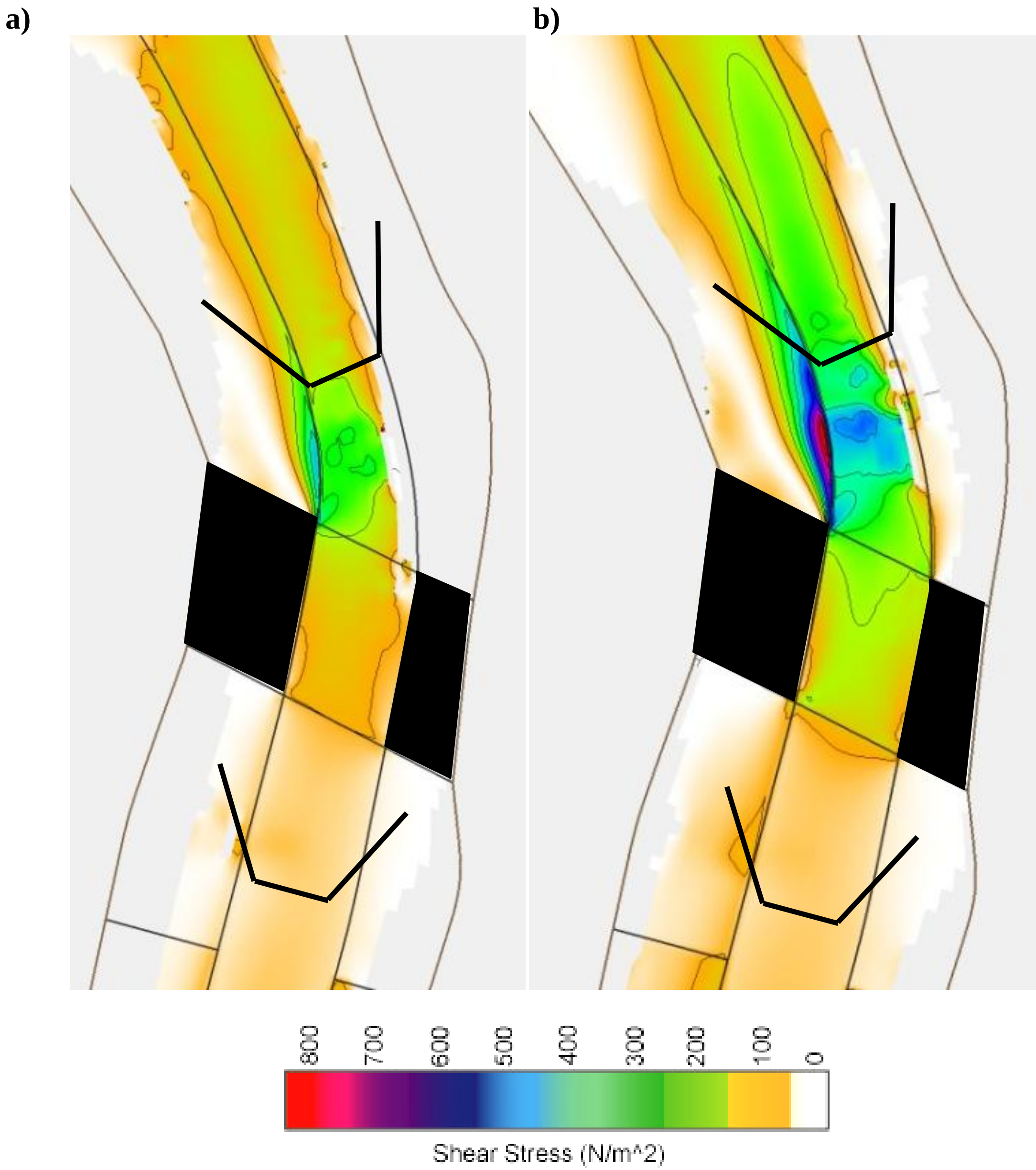


Figure 38 – FESWMS shear stress contours for the June 2009 survey at the two and ten year flow rates (a and b, respectively) (The black rectangles indicate the bridge (a 14 m x 21 m span) and approximate location of the restoration structures).

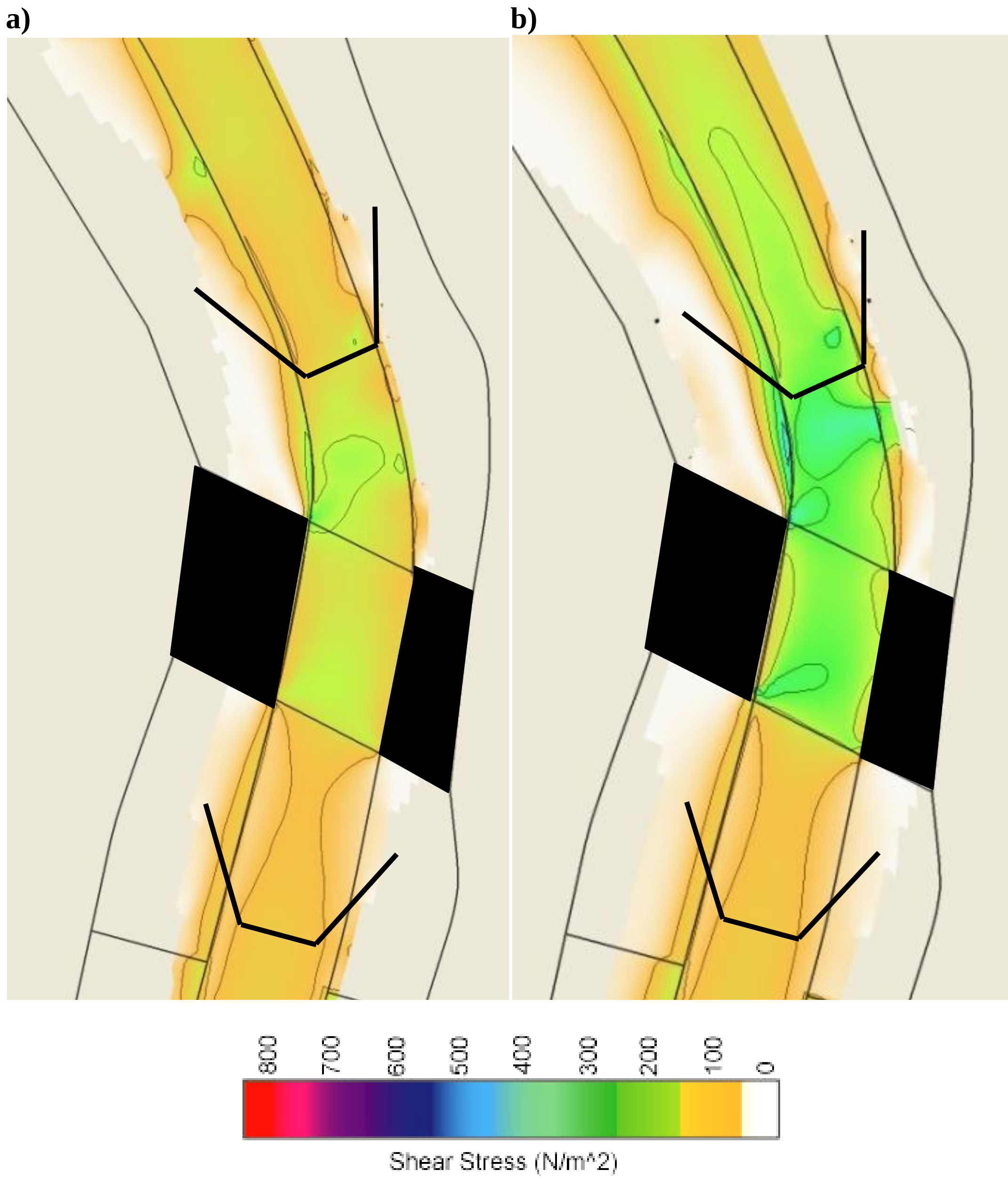


Figure 39 – FESWMS shear stress contours for the June 2010 survey at the two and ten year flow rates (a and b, respectively) (The black rectangles indicate the bridge (a 14 m x 21 m span) and approximate location of the restoration structures).

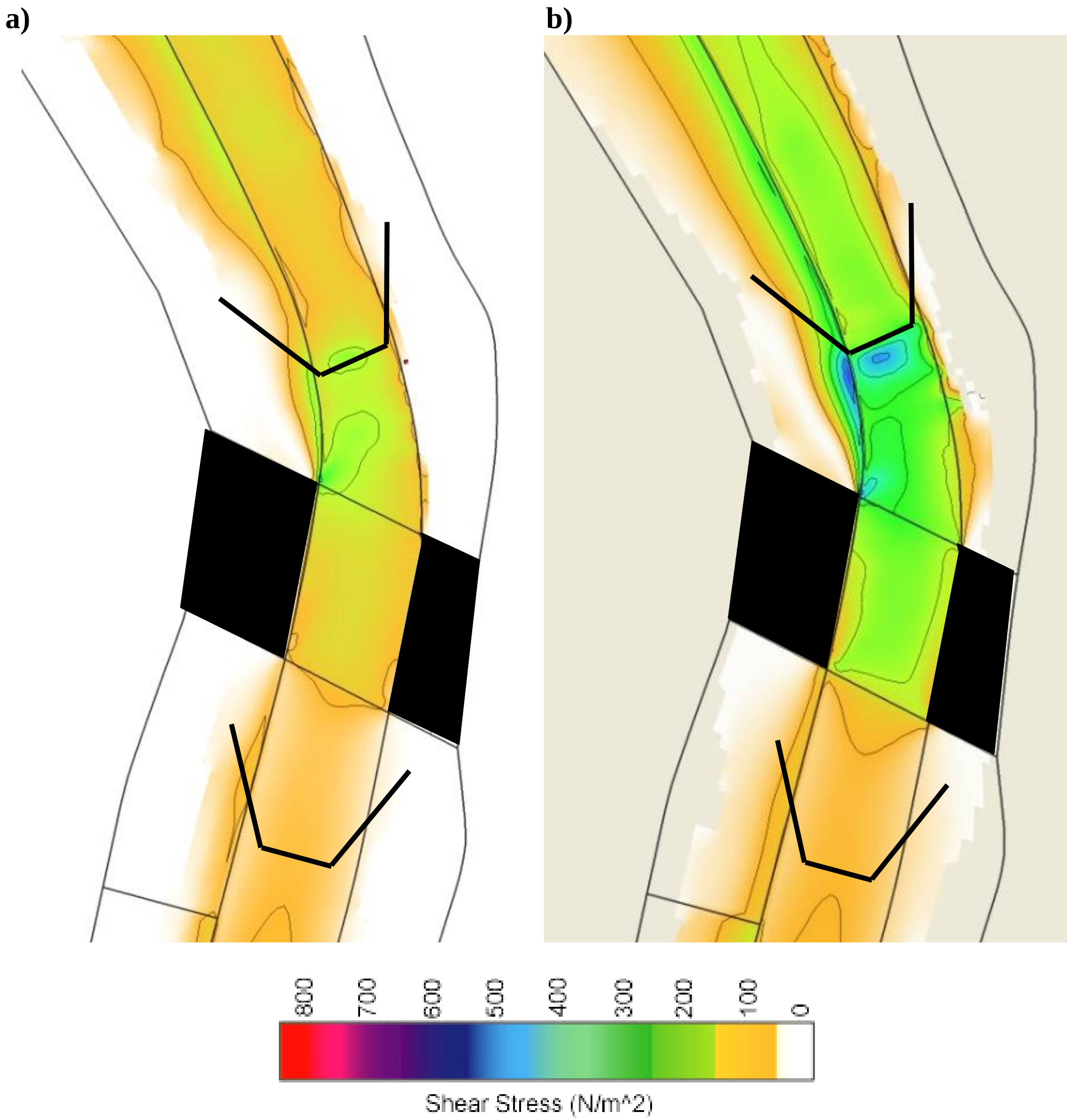


Figure 40 – FESWMS shear stress contours for the November 2010 survey at the two and ten year flow rates (a and b, respectively) (The black rectangles indicate the bridge (a 14 m x 21 m span) and approximate location of the restoration structures).

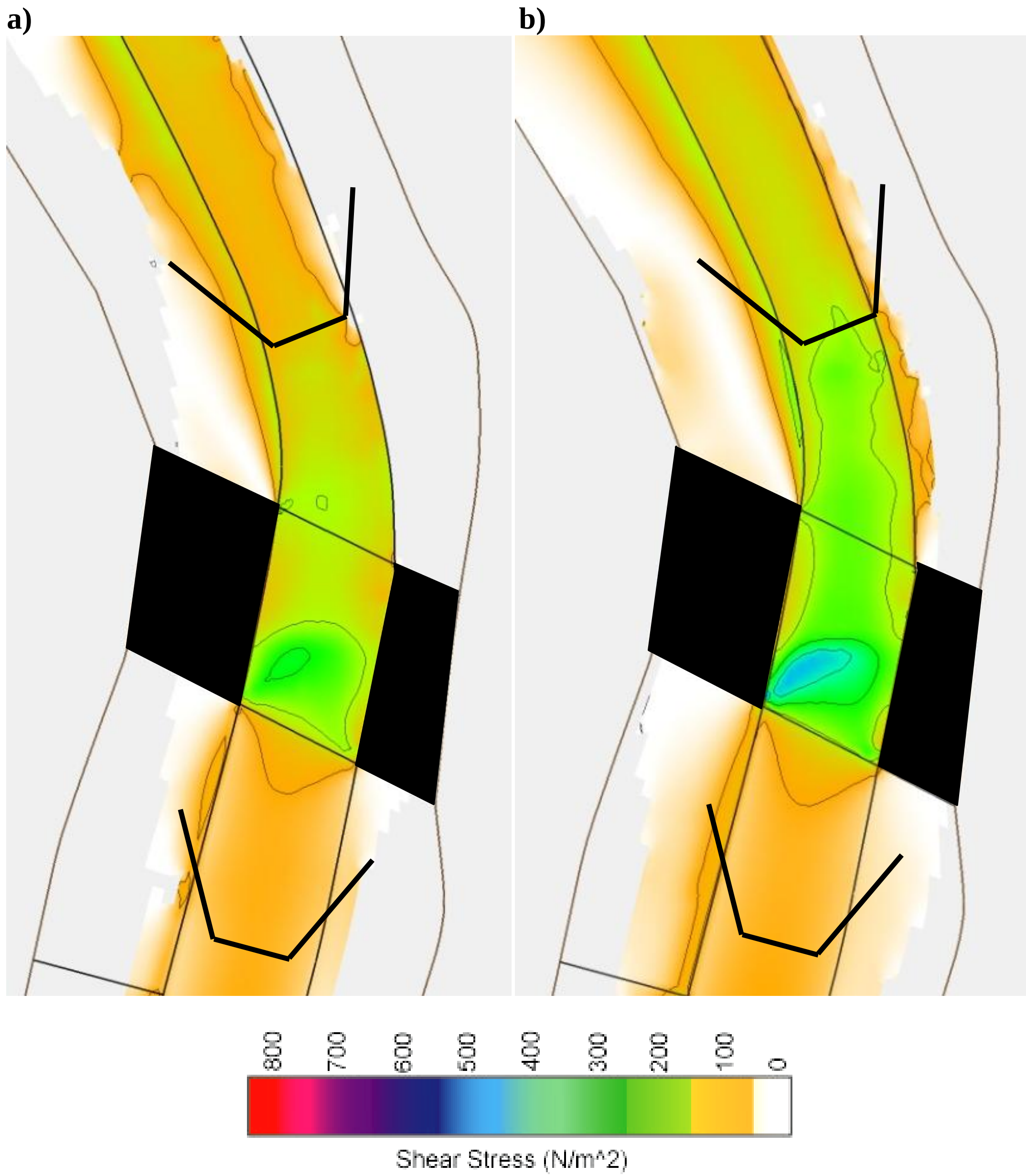


Figure 41 – FESWMS shear stress contours for the June 2011 survey at the two and ten year flow rates (a and b, respectively) (The black rectangles indicate the bridge (a 14 m x 21 m span) and approximate location of the restoration structures).

Figure 42 – FESWMS shear stress contours for the March 2012 survey at the two and ten year flow rates (a and b, respectively) (The black rectangles indicate the bridge (a 14 m x 21 m span) and approximate location of the restoration structures).

The results of HEC-RAS are provided as an average shear stress value at given sections of the stream due to the one-dimensional modeling approach. In Table 4, the sections are listed from upstream to downstream in increasing order i.e. section 1 is the farthest upstream and section 6 is the farthest downstream. The results for the average shear stress were calculated only in the main flow section of the channel, excluding any overbank flow.

Table 4 – Shear stress calculated by HEC-RAS for the main section of the channel.

Main Channel Bed Shear Stress (N/m^2) - HEC-RAS							
Location		Flow	Jun-09	Jun-10	Nov-10	Jun-11	Mar-12
Section 1	Over U/S Restoration Structure	2 Year	32.1	81.9	68.9	55.5	80.0
		10 Year	17.2	80.0	103.4	42.1	67.0
Section 2	U/S side of Old Rt 15 Bridge	2 Year	40.7	59.9	63.2	77.1	73.3
		10 Year	46.4	68.5	87.1	54.6	54.1
Section 3	D/S side of Old Rt 15 Bridge	2 Year	30.6	69.4	48.8	64.6	63.2
		10 Year	27.8	72.8	79.5	91.9	67.0
Section 4	U/S of D/S Restoration Structure	2 Year	50.3	25.4	137.9	79.5	71.3
		10 Year	38.3	10.5	80.0	47.9	92.4
Section 5	Over D/S Restoration Structure	2 Year	36.4	13.9	59.4	58.9	38.8
		10 Year	30.6	5.7	60.8	42.6	31.6
Section 6	D/S from D/S Restoration Structure	2 Year	135.5	82.4	115.4	177.6	98.2
		10 Year	120.2	109.6	148.9	136.9	192.5

The shear stress results from the two dimensional FESWMS modeling (Figures 38-42) show trends in shear stress values throughout the channel. The shear stresses, for both the two and ten year storms, decrease upstream of the bridge and increase downstream towards the restoration structure. The decrease upstream of the bridge is a direct effect of the backwater conditions created by the bridge constriction during the storm. The backwater effect is greater with the ten year storm than it is with the two year storm, and its effect on shear stress also is much greater. For both storms the shear stress upstream of the bridge is between 0-150 N/m^2, yet the shear stress downstream for the two year storm is between 250-350 N/m^2 and for the ten year storm it is between 350-550 N/m^2. The results from the one-dimensional HEC-RAS model provide an average cross sectional value of shear stress. Because of this, the predicted values are smaller than what FESWMS predicts as local shear stress values at some of the individual finite element nodes. However, trends similar to the FESWMS model results still can be seen in the HEC-RAS results shown in Table 4. For example, looking at the most recent surveyed sub-model, March 2012, the shear stress values in the main channel, over the restoration structure located upstream of the Old Route 15 Bridge, for the two year and ten year flows are 80 and 67 N/m^2 respectively. This result follows the same trend as FESWMS in that the ten year storm has a larger backwater effect than the two year storm.

When comparing the HEC-RAS shear stress values between each sub-model, the differences in the values must be attributed to the change in channel geometry. More specifically, the change in the measured channel cross section could be causing all discrepancies between each sub-model at each of the six sections shown in Table 4. In some cases, values are predicted that seem unreasonable. Examples of this can be seen in section five, over the downstream restoration structure, where all sub-models predict that the two year storm will create larger shear stresses than the ten year storm. One possible reason for this is the channel geometry is creating conditions where the lower, two year flow is more channelized when compared to the larger, ten year flow. Because HEC-RAS reports an average shear stress value for the entire section, the less channelized ten year flow includes additional lower values of shear stress across its section in the averaging.

The low shear stress upstream of the bridge causes the necessary conditions for aggradation in the vicinity of the Old Route 15 Bridge and the upstream restoration structure. This provides reason to believe that the upstream rock cross vane failure by burial is a direct result of the bridge backwater effect on the shear stress. The high shear stresses found downstream of the bridge indicate that scour and erosion are expected in that area. The higher values of shear stress indicate that the stream has an increased capacity to move sediment in that area. This matches with the visual and measured observations (Figure 43) in the downstream areas of the site, and can partly explain the bank erosion currently taking place as well as the downstream cross vane failure.

Figure 43 – Image of the bank erosion downstream of the downstream restoration structure (facing upstream).

5.6.2 Channel velocity

The velocity magnitude contour results from FES-WMS are displayed in Figures 44 - 48 with general velocity directions shown by black arrows. Similar to the shear stress contours, a velocity contour was created for the two and ten year flows for each sub-model.

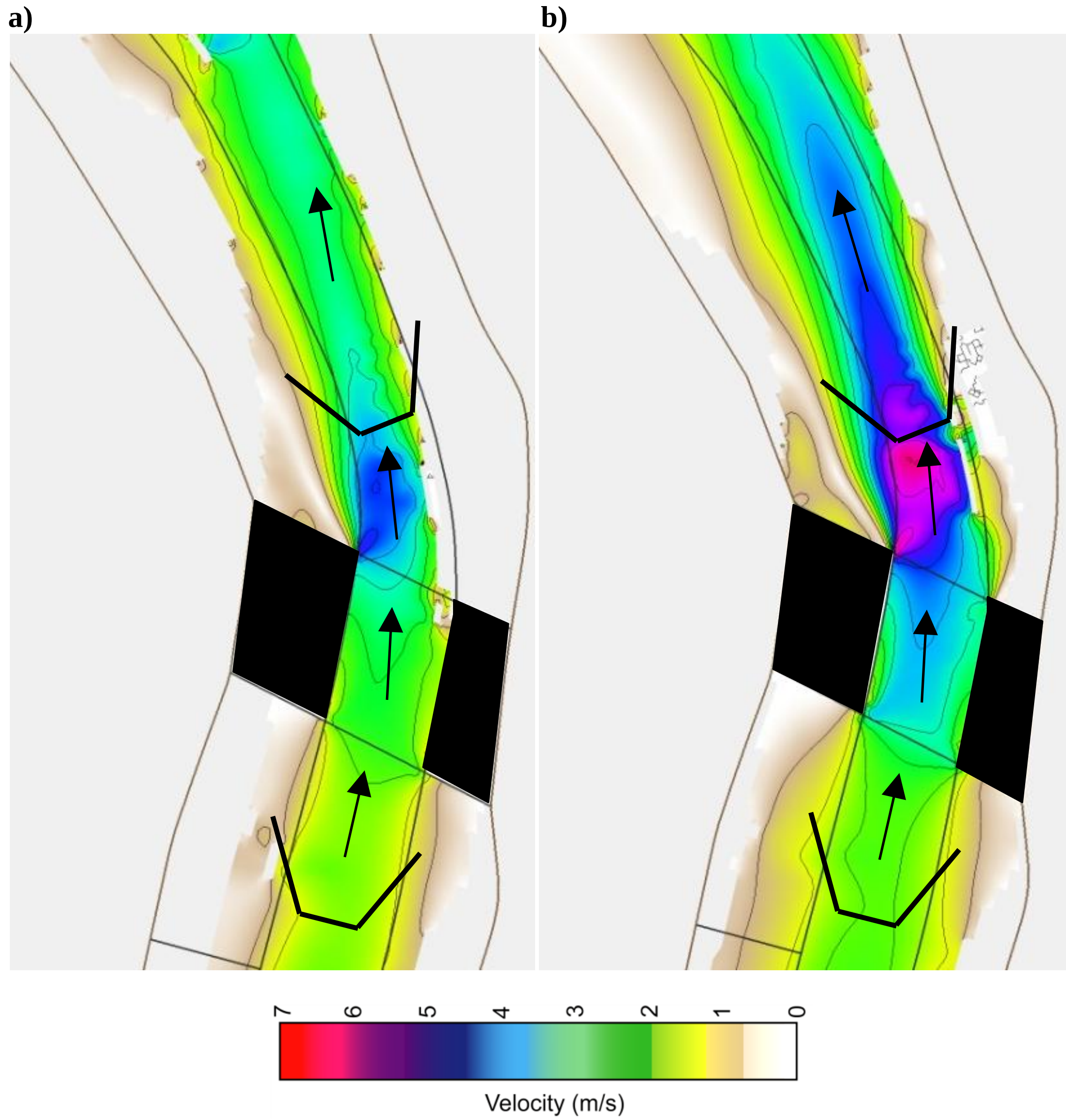


Figure 44 – FESWMS velocity contours for the June 2009 survey at the two and ten year flow rates (a and b, respectively) (The black rectangles indicate the bridge (a 14 m x 21 m span) and approximate location of the restoration structures).

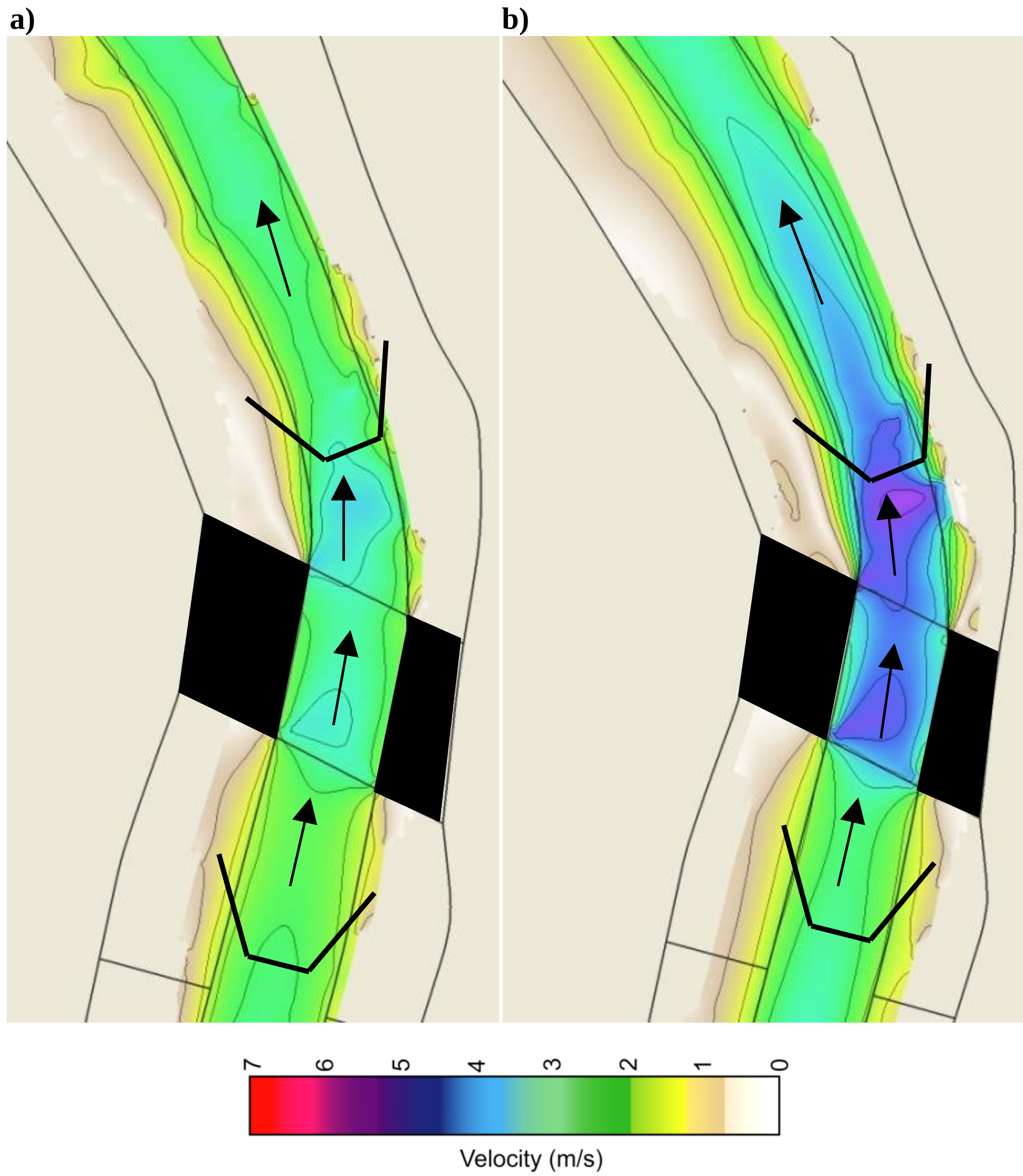


Figure 45 – FESWMS velocity contours for the June 2010 survey at the two and ten year flow rates (a and b, respectively) (The black rectangles indicate the bridge (a 14 m x 21 m span) and approximate location of the restoration structures).

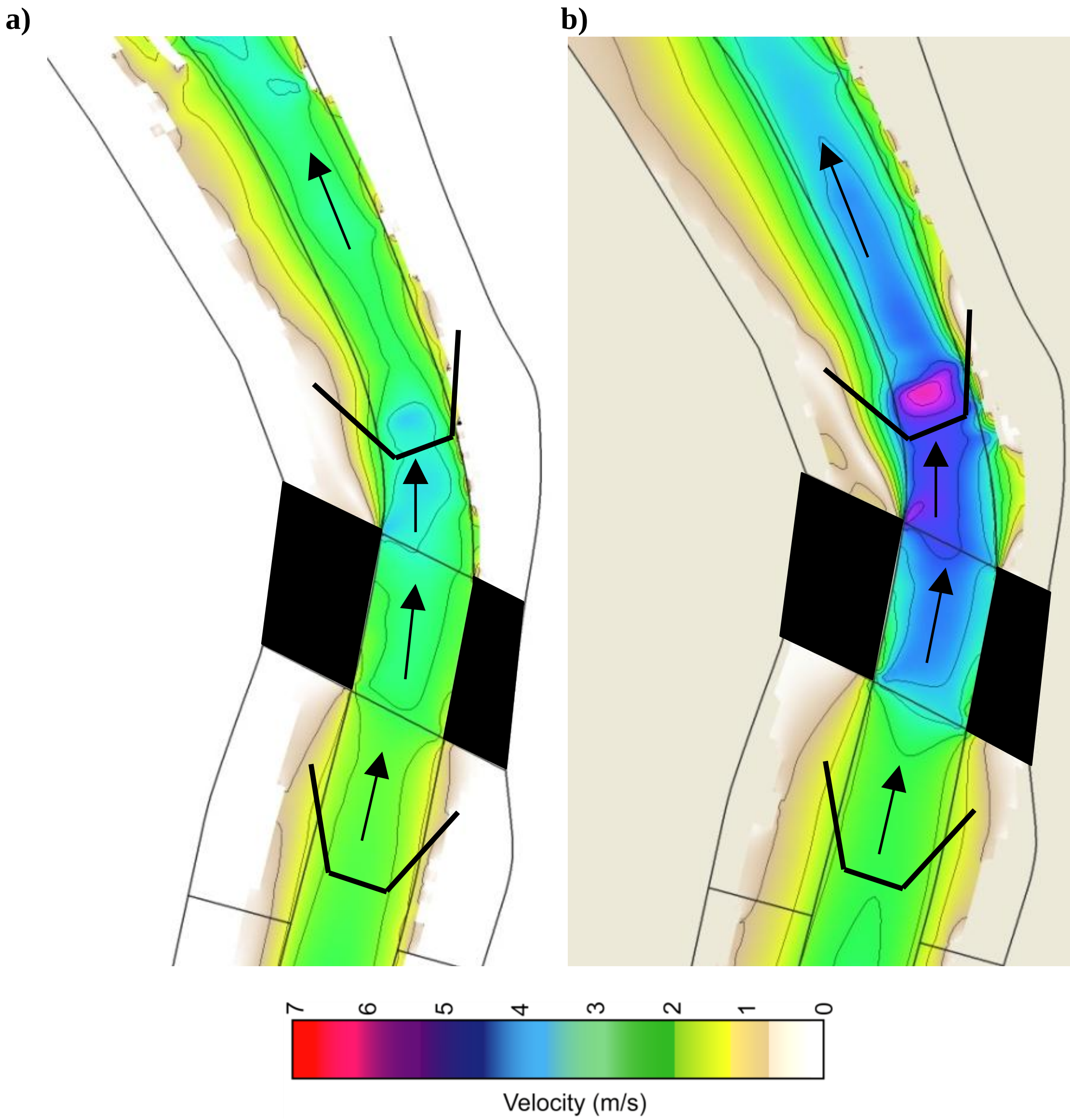


Figure 46 – FESWMS velocity contours for the November 2010 survey at the two and ten year flow rates (a and b, respectively) (The black rectangles indicate the bridge (a 14 m x 21 m span) and approximate location of the restoration structures).

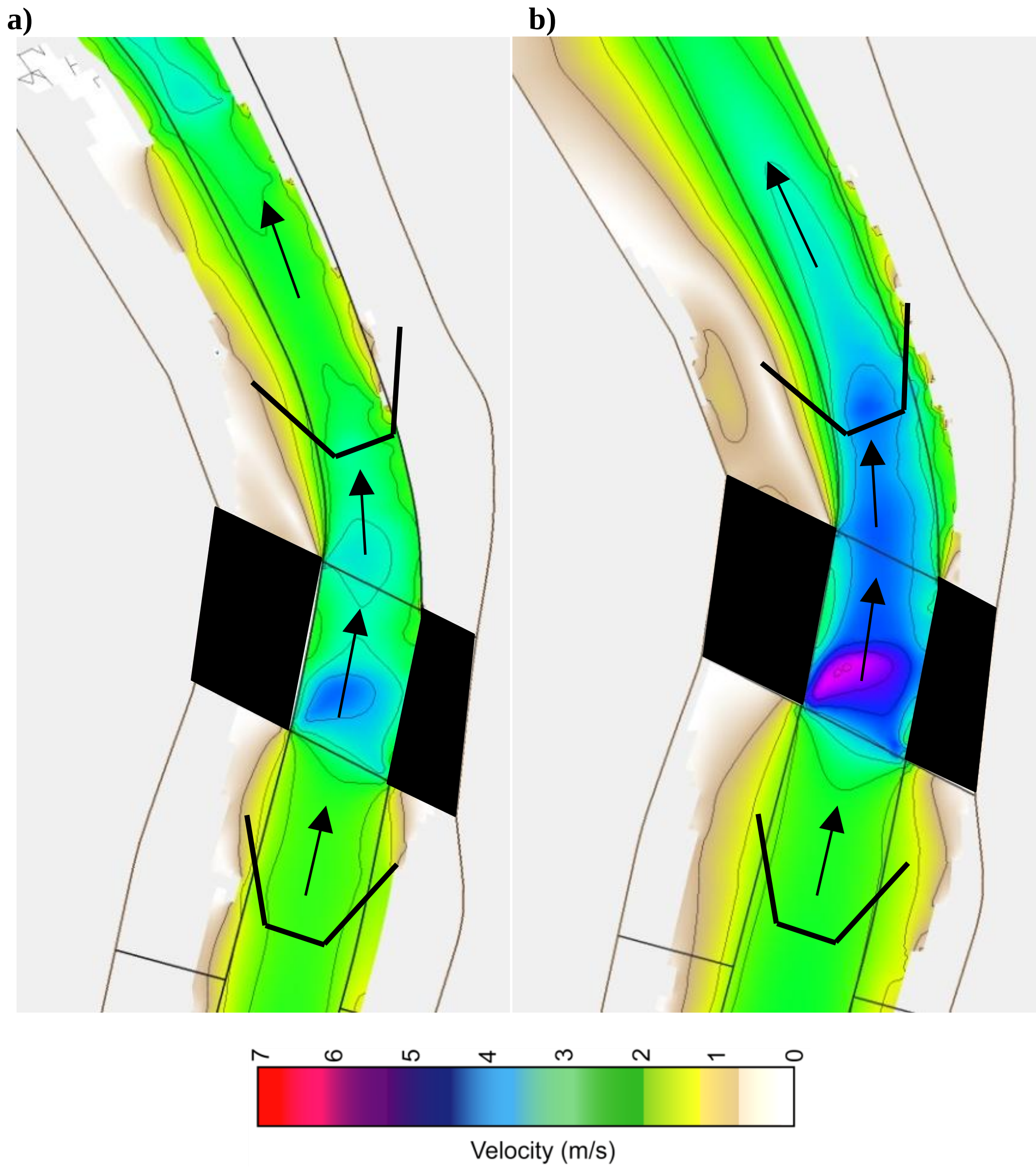


Figure 47 – FESWMS velocity contours for the June 2011 survey at the two and ten year flow rates (a and b, respectively) (The black rectangles indicate the bridge (a 14 m x 21 m span) and approximate location of the restoration structures).

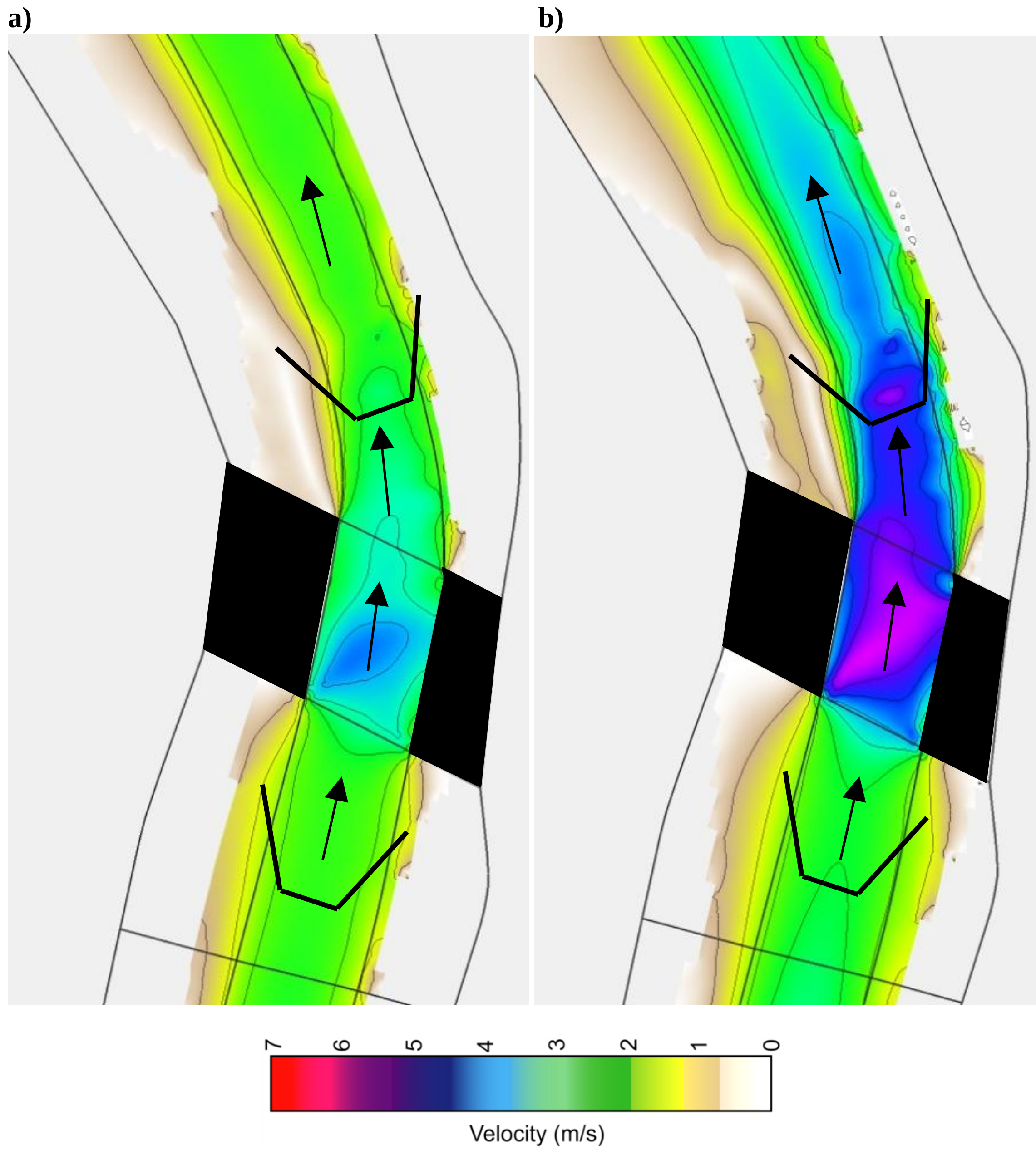


Figure 48 – FESWMS velocity contours for the March 2012 survey at the two and ten year flow rates (a and b, respectively) (The black rectangles indicate the bridge (a 14 m x 21 m span) and approximate location of the restoration structures).

Using HEC-RAS, the velocity magnitudes were calculated as an average for each cross section of the channel. Similarly to the shear stresses, the velocity magnitude results for the main channel are summarized in Table 5.

Table 5 – Average velocity calculated by HEC-RAS for the main cross section of the channel.

Main Channel Velocity (m/s) - HEC-RAS							
Location		Flow	Jun-09	Jun-10	Nov-10	Jun-11	Mar-12
Section 1	Over U/S Restoration Structure	2 Year	1.55	1.99	2.16	1.90	2.04
		10 Year	1.25	2.13	2.88	1.87	2.26
Section 2	U/S of Old Rt 15 Bridge	2 Year	1.54	1.94	1.80	2.02	1.98
		10 Year	1.83	2.29	2.33	1.95	1.94
Section 3	D/S of Old Rt 15 Bridge	2 Year	1.33	2.05	1.58	1.81	1.80
		10 Year	1.35	2.30	2.19	2.37	2.08
Section 4	U/S of D/S Restoration Structure	2 Year	1.72	1.37	2.97	2.29	2.23
		10 Year	1.59	0.94	2.51	1.97	2.79
Section 5	Over D/S Restoration Structure	2 Year	1.49	0.99	2.04	1.96	1.69
		10 Year	1.44	0.68	2.23	1.83	1.66
Section 6	D/S from D/S Restoration Structure	2 Year	3.10	2.40	2.78	3.40	2.65
		10 Year	3.19	2.86	3.34	3.31	3.94

The FESWMS velocity contours presented in Figures 44 - 48 follow the same relationship as the shear stress contours. The velocity is lower in the backwater area upstream of the bridge, 1-3 m/s, and is higher within the bridge constriction and downstream section, 3-6 m/s. The results also show that the higher velocity is closer to the right bank of the channel downstream of the bridge and downstream restoration structure. This is caused by both the bridge constriction directing the velocity and the natural response of the channel caused by the bend at this location. Again, the HEC-RAS velocities generally are lower in magnitude than the FESWMS values due to the one-dimensional modeling approach and cross sectional averaging. Even so, larger velocities are simulated downstream of the bridge crossing and of the downstream restoration structure. For example, looking at the most recent surveyed model, March 2012, the main channel velocity value during the ten year storm just downstream from the Old Route 15 Bridge is 2.08 m/s, while the velocity downstream from the downstream restoration structure is 3.94 m/s. Similarly to the shear stress discussion, discrepancies in the HEC-RAS velocity results can be found between different sub-models. This is again attributed to the changes in the geometry of local cross sections between each sub-model.

5.6.3 Simulation of observed bed elevation changes

A sediment transport scenario for modeling observed bed elevation changes only was performed with the HEC-RAS model due to model instability when running sediment transport in FESWMS for White Deer Creek. While attempting to create a sediment transport model for White Deer Creek in FESWMS, problems stemming from finite element wetting/drying during storms and the resulting unrealistic corner node values for shear stress caused instability in the model. The model was then simplified to include only the main channel and exclude the overbanks/floodplain under the assumption that the majority of transport occurs within the main channel section. Even with this simplification, FESWMS could not successfully model a sediment transport event on White Deer Creek. Therefore, the sediment transport scenario involved running a sediment transport analysis on a given surveyed sub-model of HEC-RAS and then comparing the resulting channel bed elevations to the subsequent surveyed sub-model. This scenario was only possible when flow data was collected for the storm event or events between the two different sub-model survey dates. The results allow for the opportunity to further calibrate the sediment transport model and the ability to judge the sediment modeling capability of HEC-RAS for White Deer Creek.

The elevation change analysis with HEC-RAS was run to analyze the October 1^{st}, 2010 storm event (calibrated and simulated with HEC-HMS, see section 5.5). This event was chosen because a single significant storm event occurred between two separate surveys, June 2010 and November 2010. Figure 49 displays the simulated change in bed elevations, along the profile of the stream, from before to after the storm as well as the total change in sediment mass at each section with the locations of the bridge and restoration structures highlighted.

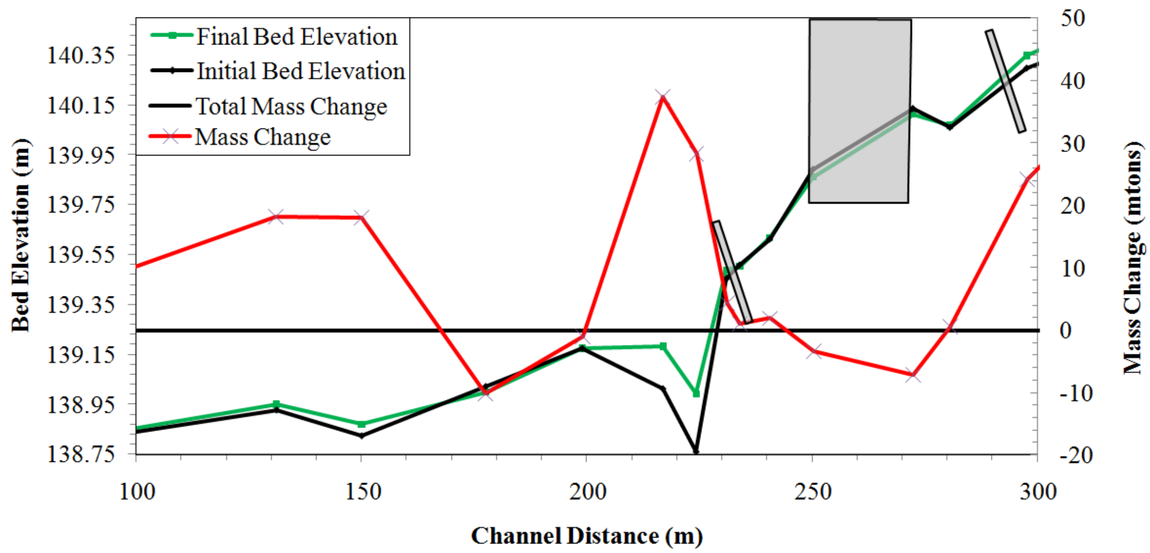


Figure 49 – Plot showing the change in the bed profile after the October 1, 2010 storm, and the total mass change at each section. Highlighted zones indicate the bridge and restoration structure locations.

The change in mass is calculated at each cross section by determining the total mass of sediment at the end of the run and subtracting the initial mass at the section. As displayed in the plot, several sections of the creek experienced a larger amount of aggradation or scour while others had little to no change in bed elevation. Figure 50 uses the bed elevations measured before and after the storm as comparisons to the HEC-RAS predicted bed elevations after the storm event. The predicted values from model simulations vary in comparison with the measured values.

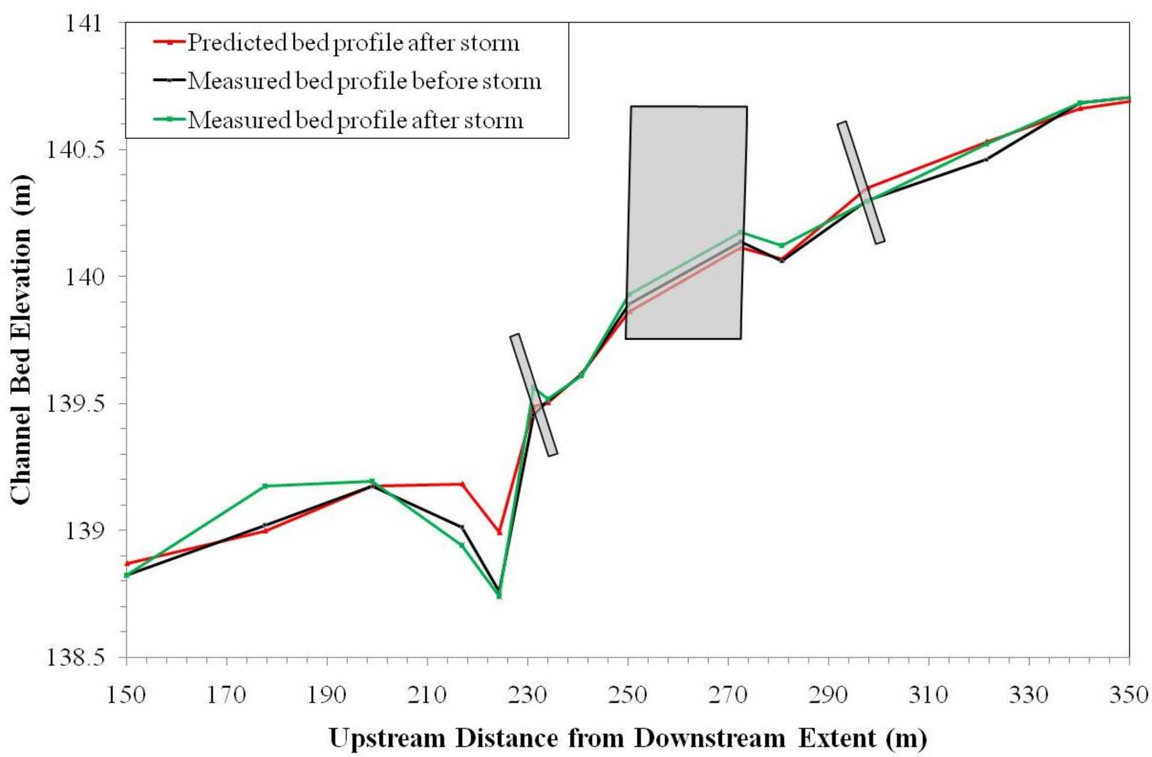


Figure 50 – Plot showing the measured bed elevations before and after the October 1, 2010 storm plotted against the HEC-RAS predicted bed elevation. Highlighted zones indicate the bridge and restoration structure locations.

The HEC-RAS bed elevation results predict aggradation of the channel bed upstream from the bridge, within the vicinity of the downstream restoration structure, and at the downstream section of the channel. The model also predicts scour beneath the bridge and just downstream of the restoration structure. As displayed in Figure 50, the predicted results only simulate the measured field results upstream of the bridge where the aggradation has taken place. The sections in the vicinity of and downstream from the bridge and downstream restoration structure are predicted incorrectly. Potential reasons for this are that both the bridge and cross vane hydraulics create larger two and three dimensional flow components which cannot be accurately modeled in the one-dimensional model, as demonstrated by the general velocity direction arrows shown in Figures 44 - 48. The cross vane located upstream of the bridge is not performing as designed due to backwater effects, and therefore, the flow characteristics in this section are more one dimensional.

6. Conclusions and Implications

Through data collection and monitoring at the White Deer Creek site, the research has shown that White Deer Creek currently is unstable. The stream is trying to re-establish equilibrium from the channel relocation as well as from the higher sediment load created by the past logging of the watershed. In several locations, the channel bed is experiencing aggradation and scour. The scour is causing the failure of the riprapped channel banks and channelization downstream from the downstream restoration structure. The channel also has migrated to the outside of the bend downstream of the downstream restoration structure towards its pre-relocation alignment.

The research has provided several causes for the failure of the stream restoration structures installed in sites similar to the White Deer Creek study site. As demonstrated by the physical modeling study, a high sediment load increases the potential for the burial of a restoration structure. Backwater conditions from the bridge crossing also create the appropriate conditions for the burial of a restoration structure as seen in both the physical modeling study and the mathematical modeling studies investigating the channel shear stresses. Looking at the erosion failure mode of a restoration cross vane structure, the physical modeling, mathematical modeling studies investigating velocity magnitude and direction, field observations, and the geophysical studies all demonstrate that the alignment of the in-stream structure with the flow direction is directly linked to erosion along the wings of the structure that eventually causes failure along one side of the structure.

White Deer Creek is unique in that the upstream restoration structure failed due to depositional processes and the downstream restoration structure is failing due to erosive processes. Because there is evidence of channel bed aggradation upstream, a portion of the sediment load that White Deer Creek is transporting must be lost in the backwater section upstream of the Old Route 15 Bridge which decreases the sediment supply downstream of the bridge. The decrease in supply creates the condition where the capacity of the flow to transport sediment is greater than the supply, so the water erodes sediment from the channel bed or banks to make up the deficit. This results in the observed down cutting of the channel bed in survey data as well as erosion observed along the banks.

The necessary mathematical modeling complexity for simulation of bridge hydraulics and sediment processes as well as stream restoration hydraulics and sediment processes also can be evaluated. A one-dimensional model cannot fully represent the multi-dimensional hydraulics created by both the bridge crossing and restoration structures. This is evident when comparing the one-dimensional HEC-RAS velocity results with the two-dimensional FESWMS velocity results. In these hydraulically complex zones, the one-dimensional model commonly under predicts the velocity, in some cases by almost half, but in zones demonstrating more simplified hydraulics, such as a straight channel with fairly uniform cross section (upstream of the bridge crossing), the velocities simulated by both HEC-RAS and FESWMS match very well. The one dimensional model also cannot accurately simulate depositional and erosional patterns accurately within the vicinity of the bridge and restoration structures. Again, this likely is due to the oversimplified hydraulics used during the one-dimensional simulation. However, many problems arose when attempting to simulate sediment transport with the FESWMS model, such as decreased model stability and unrealistic results. In the case of White Deer Creek, FESWMS could not successfully complete an entire simulation without crashing unless the simulation was oversimplified. This suggests that more site specific data may need to be collected including, consistent flow and sediment discharge rates, to help improve the results from a one-dimensional sediment transport simulation. Because the ability to simulate the multi-dimensional hydraulic processes at bridges and restoration structures is important, additional multi-dimensional sediment transport simulation models that use other solution schemes (such as finite difference) should be explored for use at these structures.

The results of this research study provide insight into the design and construction of stream restoration structures built to stabilize the channel and prevent aggradation within the vicinity of a bridge crossing. Aligning the structure with the flow direction along with the effects of backwater must be considered within the design of any restoration structure built in the vicinity of a bridge to avoid any premature failures or unnecessary maintenance costs. In the case of White Deer Creek, the downstream vane was not in alignment with the flow direction which increased the scour potential along the failing side of the cross vane. Also the backwater effects from the bridge are noticeable even with the two year flood event which would suggest that the upstream cross vane would likely fail two years after construction. Although these problems may stem from the initial design of the bridge crossing, any restoration structure should be designed according to the current site hydraulics or, if warranted, a different approach may be needed to mitigate the sediment problems taking place at a bridge crossing. In addition to providing insight into the design and construction of stream restoration structures, the results have implications for the design of bridge crossings. Current bridge waterway opening design considers only the passage of a specified design water discharge without overtopping the bridge. Additional considerations for bridge crossings may include the minimization of backwater effects for channel-forming flows and minimal channel relocation requirements that account for the sediment transport dynamics of a bridge crossing site.

7. Future Research

The White Deer Creek research site is well suited for future and/or continuing research. Actions were taken during this research to allow for many different types of measurements to be taken more efficiently in the future. To guarantee access to water depth and discharge data, a stream gage was installed on the upstream bridge pier (current US Route 15). As seen in Figure 51, the stream gage consists of sections of PVC pipe connected together that extend up the pier about three meters and out into the channel about 1.5 meters.

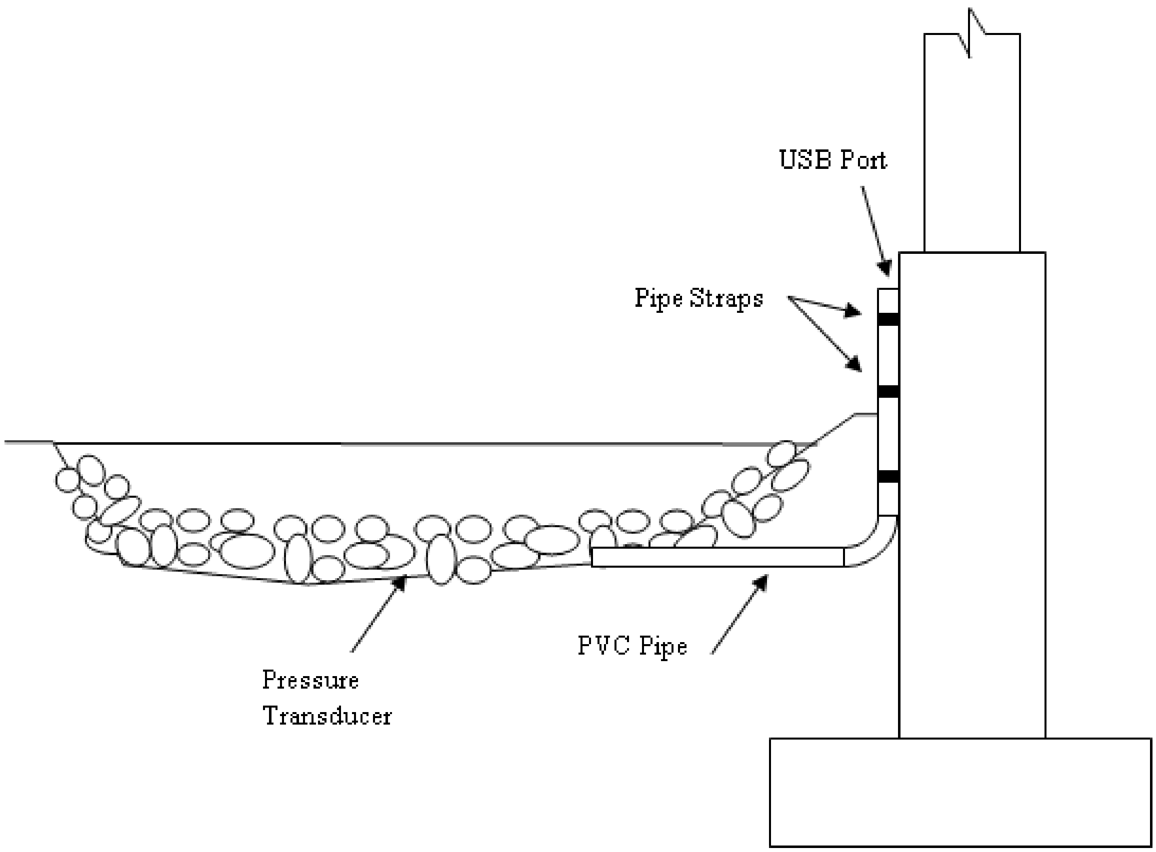


Figure 51 – The setup of the installed stream gage to measure water depth on White Deer Creek.

Inside the pipe is a pressure transducer that records the differences in pressure at a set time interval. Once the gage is calibrated, the pressure differences can then be used to calculate water depth above the transducer which can be calibrated to predict the flow rate at that time. The installation of a stream gage with a submerged pressure transducer, scour chains, and permanent cross section locations will allow for more efficient access to data needed for future research. Additional sediment bedload equipment also has been obtained to measure sediment discharge rates directly during higher flows. Future research at the White Deer Creek study site could focus on the continuation of monitoring and collecting geometric, hydraulic, and sediment data. Continued monitoring will lead to more informed mathematical models that can be used for channel response prediction to potential channel mitigation measures.

Future research also is needed in the use of multi-dimensional modeling to investigate sediment transport processes of a natural channel. As the model matches a natural stream more closely, the instability of the model solution becomes larger. Many problems arose in this research when attempting to create a two-dimensional sediment transport model that was not oversimplified. Although a two-dimensional sediment transport model could be created, it involved using a larger finite element mesh to model only the main section of the channel. With more research on multi-dimensional sediment transport modeling techniques, the modeling capability will increase and allow for a more realistic model of a natural channel.

Research on design guidelines and techniques for rock cross vanes also is needed for increasing the effectiveness and service life of the rock cross vane structure. This research has shown that conditions that could cause failure are present at both the two and ten year flows. Current design guidelines only involve using a bankfull elevation and width to aid in the construction and placement of a rock cross vane. Mathematical and physical modeling studies involving prediction sensitivity analyses to test different rock cross vane configurations and dimensions could provide useful data related to the rock cross vane design. By looking into different configurations and designs of the rock cross vanes and the effect they have on the channel hydraulics, the design of the cross vanes could be improved. This could prevent early failures such as those that have occurred at the White Deer Creek site.

8. References

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