HYDRAULIC MODELING AND PERFORMANCE EVALUATION OF LOW WATER CROSSINGS IN ILLINOIS

BY

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THESIS

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ABSTRACT

The Illinois Department of Transportation (IDOT) and local agencies monitor and regulate the 146,764 miles of roadway that are open to public travel in the State of Illinois. There are many old and aging bridges, culverts, and low water crossings on rural low-volume roads that need to be replaced.

Low water crossings (LWCs) have been used as an economical alternative to culverts and bridges, designed without overtopping, on low-volume roads where there is a low number of floods. The lack of design guidance has posed difficulty for county engineers in Illinois in deciding when, where, and which type of low water crossing to use. The resulting structure is often either overdesigned or underdesigned.

A study was conducted to design the guidelines for LWCs in Illinois at the University of Illinois at Urbana-Champaign in collaboration with the U.S. Army Corps of Engineers - Construction Engineering Research Laboratory (CERL) and support from the IDOT. The study included literature review, a LWC survey, and case studies on LWCs in Illinois.

The results of a survey conducted among the county engineers in Illinois about their experience with LWCs are presented, and commonly used LWCs are also discussed. In this study, five existing LWCs in Illinois are selected, modeled in HEC-RAS to analyze their performance. The ability of the LWC to pass the design flow, the effect of the LWC on the floodplain of the stream, sediment transport, and movement of fishes across the LWC have been taken into consideration in the performance evaluation.

From the case studies, it was found that most of the modeled LWCs were able to pass the design flow, but are not conductive to the sediment transport and aquatic organism movement. Results from the flood inundation studies show that the change in the inundated area compared to the baseline scenario is within 5% in most of the cases. There is a significant decrease in the shear stress and velocity in the cross section upstream of the crossing,

restricting the sediment transport. LWCs are acting as a sediment trap, which over the long period of time will modify the channel characteristics and affect the stream dynamics.

LWCs provide a restriction to the flow of water and increase inundation under higher flows but allow smooth and safe movement of vehicles across the streams. There are over 26,000 road-stream crossings in Illinois, and implementation of proper LWC design guidelines could save local agencies significant funding and provide better adaptability and storm-proofing characteristics, as well as reduce impacts to aquatic organism passage.

Keywords: Low water crossings, fords, HEC-RAS, floodplains, modeling

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CHAPTER 1: INTRODUCTION

Illinois has 146,764 miles of road; the Illinois Department of Transportation (IDOT) maintains 15,978 miles, and local agencies maintain the rest (IDOT 2016). These agencies are responsible for regulating and monitoring the road networks that are open to public travel. There are many old and aging bridges, culverts, and low water crossings on rural low-volume roads, primarily serving agricultural areas, which need to be replaced with suitable alternatives. The economic burden for many counties can be huge if all of these structures are to be replaced by a bridge or culvert designed to convey the entire design flow.

A low water crossing (LWC) is a feasible and efficient road-stream crossing structure that may be implemented on roads with average daily traffic (ADT) values of less than 25. LWCs are road-stream crossing structures designed to be overtopped by high flows or by debris- or ice-laden flows (Clarkin et al. 2006). At times when the structures are overtopped, the road will be closed to traffic, and alternative routes must be used.

These relatively inexpensive structures are very useful for low ADT roads across ephemeral streams and where the normal depth of flow is low. In Illinois, these structures might be of particular interest on farmland access roads that are used only a few times a year for transport of machinery and other agricultural commodities and supplies. There is a high potential that construction of bridges and culverts will change the hydrological and hydraulic characteristics of the stream, leading to effects such as higher peak flow rates and runoff, increased downstream flooding, increased rates of sediment transport and deposition, increased erosion, widening of stream channels, etc. On the other hand, LWCs can also help in streambed stabilization, thus minimizing those effects in the stream.

Current IDOT bridge design guidelines require 1 ft of vertical clearance above the design high-water elevation for local agency roadways with an ADT < 250, where the minimum design flood frequency is commonly a 15-year event. Often, these requirements result in large waterway openings, bridges, or structures, and costly embankment construction—and are not conducive to the construction of a LWC.

In this study, five existing LWCs in Illinois were selected, modeled in HEC-RAS to analyze their performance. The ability of the LWC to pass the design flow, the effect of the LWC on the floodplain of the stream, sediment transport, and movement of fishes across the LWC have been taken into consideration in the performance evaluation. Suggestions for improved design and construction for LWCs are also included.

The results and findings of the study are a part of the Illinois Center for Transportation project R27-148 “Development of Low-Water Crossing Design Guidelines for Very Low ADT Routes in Illinois.” There are over 26,000 road-stream crossings in Illinois, and implementation of proper LWC design guidelines could save local agencies significant funding, due to lower construction and maintenance costs, less channel and flood plain blockage, and better storm proofing characteristics, as well as reduced impacts to aquatic organism passage (AOP).

CHAPTER 2: OBJECTIVES

The overall objective of this research was to develop guidelines that can be used to determine optimal, safe, and cost-effective LWC design to meet traffic needs, maintain the natural channel function, and allow the passage of water, sediment, debris, and AOP. The outcomes of this research will assist IDOT and local agencies in determining the safe, cost-effective, and environmentally friendly design of LWCs for low ADT routes in the state.

The specific objectives of this research project are as follows:

1. Conduct a thorough literature review of current practices and existing research publications and other federal, state, and county reports, studies, recommendations, and specifications related to LWC design.

2. Conduct a survey on the current status of Illinois LWCs, including the experience of local agencies with LWCs in Illinois.

3. Conduct field survey on selected LWCs that fall under the jurisdiction of Illinois public agencies.

4. Conduct performance evaluation of LWCs based on the LiDAR and field survey data using HEC-RAS model.

CHAPTER 3: REVIEW OF LITERATURE

3.1 Low Water Crossings (LWCs)

The Natural Resources Conservation Service (NRCS) defines a LWC as "a stabilized area or structure constructed across a stream to provide a travel way for people, livestock, equipment or vehicles" (NRCS 2011). Three main types of LWCs, which are designed to submerge at some flows, include unvented fords, vented fords, and low water bridges (Figures 3.1 through 3.3).

3.1.1 Unvented Fords

An unvented ford is a structure that crosses streams that are dry most of the year or where normal stream flow is less than or equal to 6 inches in depth. Unvented fords are usually used for ephemeral streams or streams with shallow flows. They typically cross streams at, or slightly above, the elevation of the streambed without pipes. The grades of the roadway approaches are shaped to provide a smooth transition with acceptable slopes of less than 10% (Lohnes et al. 2001). The crossing may be constructed of crushed stone, riprap, precast or cast-in-place concrete slabs or other suitable material. Based on the crossing surface, unvented fords are divided into two categories: unimproved or improved.

- *Unimproved fords* are simply natural crossings (Figure 3.1).
- *Improved fords* have a stable driving surface of rock, concrete, asphalt, concrete blocks, concrete planks, gabions, geocells, or a combination of materials.

Unvented fords are called "at-grade" if the crossing is placed directly on the stream channel bottom, whereas "above-grade" structures are raised to a certain height above the channel bottom.

- *At-grade LWC*s provide a minimal barrier for AOP, and there is less chance for channel modification (due to aggradation or degradation).
- *Above-grade LWC*s may act as a dam and trap the sediment flow, which may lead to channel aggradation upstream and degradation (scour) downstream.

Figure 3.1: Unvented ford across Big Creek in Hamilton County, Illinois.

3.1.2 Vented Fords

Vented fords have a driving surface elevated above the channel bottom with vents (pipes or culverts) that allow low flows to pass beneath, keeping vehicles out of the water during low flow (Clarkin et al. 2006). High water will periodically flow over the crossing. Approaches are designed to provide acceptable grades of less than 10% by shaping the roadway or adjusting the elevation of the crossing (Lohnes et al. 2001). The pipes or culverts may be embedded in earth fill, aggregate, riprap, or concrete.

Vented fords differ from culverts because higher flows overtop the vented ford. Thus, the vented ford is designed to pass low flow such as 1% exceedance flow or 1-year flow and higher flows pass over the structure. However, other parts of the crossing such as approach roads, embankments, etc. are designed for higher flows such as 10- or 25-year flow, depending upon the desired lifetime of the structure.

The vents can be one or more pipes (Figure 3.2), box culverts, or open-bottom arches. The opening and number of vents depends on the stream geometry and flow characteristics, and is defined by the vent-area ratio (VAR). A low VAR refers to a small vented area relative to the bankfull channel area, while high VAR refers to a vented area equal to or greater than the bankfull channel area (Clarkin et al. 2006). Bankfull flow can be defined as the flow that just overtops the stream banks and begins to flow out over the floodplain (Leopold et al. 1964).

Figure 3.2: Vented ford in Jackson County, Illinois.

3.1.3 Low Water Bridge

Low water bridges are defined as open-bottom structures with elevated decks and a total span of at least 20 ft (Clarkin et al. 2006). They may include one or more piers with abutments and are structurally very similar to other bridges except they are built lower, allowing periodic overtopping. Low water bridges generally have greater capacity and can pass higher flows underneath the driving surface than most vented fords. However, they are designed and installed with the expectation they will be under water at higher flows (Howard et al. 2011). They are constructed at about the elevation of the adjacent stream banks, with a smooth cross section designed to allow high water to flow over the bridge surface without damaging the structure. A low water bridge is preferred in an area with ADT over 200 and where minimum disturbance to the channel geometry and aquatic organism passage is desired. To function as low water bridges, the structures should be such that they pass flow most of the time, yet be low enough to be overtopped by larger floods (Figure 3.3).

Figure 3.3: Low water bridge at Montgomery County, North Carolina (Filer, 2008).

3.1.4 Advantages and Disadvantages of Low Water Crossings

Low water crossings have advantages as well as disadvantages. Some of the advantages of LWCs are as follows:

- Structures designed for overtopping, hence less damage during overtopping
- Less likely than culverts to be plugged and damaged by debris or vegetation
- Less expensive than culverts or bridges
- Less susceptible than other structures to failing during flows higher than the design flow
- Good for storm proofing roads where large amounts of sediment and debris are expected, such as after a large storm event or forest fire
- Readily available materials and fast construction

Some of the disadvantages of LWCs are as follows:

- Periodic or occasional traffic delays during high-flow periods
- Not well-suited to deeply incised channels
- Not desirable for high use or high-speed roads
- Can be difficult to design for aquatic organism passage

- Can be dangerous to traffic during high-flow periods
- Periodic maintenance is required

3.2 Current Status of LWC Design Guidelines

LWCs are suitable for low-volume roads in flat, arid regions, such as the southwestern and midwestern United States, over streams with wide floodplains, and over streams where the depth of normal flow is very shallow. Various national and state agencies—such as the USDA Forest Service and the USACE Construction Engineering Research Laboratory (CERL), the Department of Transportation, and the Departments of Fish and Wildlife in several states—have published guidelines for construction of LWCs, making traffic conditions, aquatic organism passage, and stream morphology the primary criteria.

The U.S. Department of Transportation Federal Highway Administration published an executive summary titled *Design and Construction of Low Water Stream Crossings* (Motayed et al. 1982). It summarizes the commonly used low water crossings, and their selection criteria and design considerations, based on the design and performance of existing structures and interviews with highway officials.

The United States Department of Agriculture Forest Service published a LWC design manual (Clarkin et al. 2006), which consists of geomorphic, biological, and engineering design considerations. It is the most comprehensive manual, and it details LWCs, their benefits, selection criteria, design elements and consideration tools, and best management practices to follow, and it also provides several case studies.

A study was done by CERL in collaboration with the University of Illinois at Urbana-Champaign (Svendsen et al. 2006) in which design and testing of LWCs for military operations are detailed. The study demonstrates site-specific LWC designs, which have low maintenance problems and associated costs. Apart from that, CERL has also published public works technical bulletin 200-1-115 (Howard et al. 2011), *Low-Water Crossings—Lessons Learned*, which details the experiences with LWC installations for military purposes.

Iowa has design and construction guidelines (Lohnes et al. 2001), prepared by a collaboration between the Iowa Department of Transportation and Iowa State University. The guidelines include a summary of selection criteria (site selection, LWC selection) and provide details about the construction of LWCs (unvented ford, vented ford, and low water bridge). The guideline also provides recommendations on signage at LWC sites.

A study report was published in 2009 that provides design guidance for LWCs in areas of extreme bed mobility in Edwards Plateau, Texas, on the basis of a study done by Texas Tech University in collaboration with the University of Houston and Auburn University (Thompson et al. 2009). In the project, researchers used a qualitative physical model, as well as numerical modeling (HEC-RAS), to compute sediment transport.

The Missouri Department of Natural Resources has provided guidelines for temporary stream crossings, in which the minimum requirements for LWCs are included, along with construction guidelines and methods for erosion control and stream bank protection (Missouri DNR, 2016).

Similarly, Section 5-9 of the *Indiana Drainage Handbook* (Burke et al. 1999) discusses stream crossings, their construction, and repair recommendations. It provides an overview of factors to consider when these practices are undertaken. The Indiana Department of Natural Resources has also provided general guidelines for stream crossings on its website (Indiana DNR 2016).

Massachusetts, Vermont, and Washington have each published a stream crossings handbook with special emphasis on the aquatic organism and fish passage. The Massachusetts Riverways program published a stream crossings handbook (Singler and Graber 2005) that contains minimum design standards for stream crossings, taking fish and wildlife passage into consideration. It also provides guidelines on replacing aged crossings.

The Vermont Department of Fish and Wildlife developed guidelines for the design of stream crossings for passage of aquatic organisms (Bates and Kirn 2009). The guidelines focus on design, installation, and maintenance of stream crossings to provide aquatic organism passage and aquatic habitat connectivity in the rivers and streams. They have suggested that any of the three design methods—a low-slope option, stream simulation option, and hydraulic option—may be used in designing the culverts.

The Washington Department of Fish and Wildlife developed guidelines for the design of water crossings (Barnard et al. 2013), giving special emphasis to fish passage and habitat protection. The manual contains five different design methods: no-slope culvert design, stream-simulation culvert design, bridge design, temporary culvert and bridge design, and hydraulic design.

The Kansas Department of Transportation, in collaboration with the University of Kansas, is also preparing design guidelines for LWC construction. Previously, it had been following the selection and design guidelines prepared by the Iowa DOT and the signing strategies manual prepared by the Texas Transportation Institute at Texas A&M University.

3.3 LWCs and Environment

In general, regardless of site specifics, the primary advantages of LWCs over culverts and bridges may include lower construction and maintenance costs, less channel and flood plain blockage, and less susceptibility to failure during high-flow events (Clarkin et al. 2006).

LWCs are less expensive to construct, less complicated to design, quicker to construct, and require fewer materials than traditional culverts or bridge crossings do, especially for unvented fords (Howard et al. 2011). In some cases, the initial cost of more complex LWCs may exceed those of simple culverts, but the lower long-term maintenance and repair costs associated with the LWCs may still make them more economical (Clarkin et al. 2006).

However, environmental effects must also be considered when deciding whether to use LWCs or not. Unvented fords are the most inexpensive to construct, but they may not be the safest or most environmentally friendly for the stream if the traffic volume surpasses the capacity of the crossing (Howard et al. 2011). Fords, especially simple unhardened crossings, are subjected to runoff and gullies at the ingress and egress of the crossings. When heavy vehicles cross streams, they can greatly contribute to stream bank and soil erosion in the area due to excessive vegetation loss and soil disturbance (Howard et al. 2011; Svendsen et al. 2006).

Field studies of hardened LWCs have shown that, when implemented properly, these crossings maintain stream water quality, reduce stream habitat fragmentation, and decrease maintenance expenses over the unimproved fords (Sample et al. 1998; Svendsen et al. 2006). Hardened LWCs are likely to scour on the approaches and the downstream edge of the crossing, especially when perched above the channel bottom (Howard et al. 2011). They should be built such that the main flow channel is not narrowed because it might result in increased flow velocities.

Malinga (2007) assessed the impact of simple LWCs on stream stability at Fort Riley, Kansas, and found that poorly located crossings can change the direction of stream flow, causing bank erosion on areas immediately below crossings, while backwater pools upstream of the fords acted as sinks for sediment and disrupted the sediment transport. Also, there is a need to constantly modify simple LWCs relative to the level of stream instability at the site, and such crossings can contribute to the further geomorphological instability of the stream.

Vented fords keep vehicle tires dry during base flow conditions, preventing soils and other pollutants from vehicles entering the stream (Howard et al. 2011). However, vented fords can also cause the stream to lose its natural hydrological properties, and culverts can clog due to debris and sediment, which is less likely to occur at unvented crossings (Howard et al. 2011). The geomorphic response of streams at vented fords (concrete slabs with one or more culverts) at Fort Riley, Kansas, included: mean riffle spacing upstream of the LWCs was double that of downstream reaches, greater deposition of fine sediments occurred directly upstream, and incised channels downstream. The vented fords also slowed or blocked the transportation of water, sediments, and debris downstream during bankfull flows.

The USDA Forest Service requires that all low water bridges receive specific hydrologic, hydraulic, structural, and foundation design in accordance with the latest version of the American Association of State Highway Transportation Officials (AASHTO) *Standard Specifications for Highway Bridges* (Clarkin et al. 2006). Low water bridges have an elevated driving surface, maintain a more natural streambed, allow more natural sediment and aquatic organism movement than culverts, and are the best LWC type for fish passage (Clarkin et al.

2006). Low water bridges can be more expensive to design and build, and they are still susceptible to clogging under conditions such as high debris loading (Howard et al. 2011).

Brown (1994) found that sediment is contributed to the stream at LWCs by five major processes: (1) creation of wheel ruts and concentration of surface runoff, (2) existence of tracks and exposed surfaces, (3) compaction and subsequent reduction in the infiltration rate of soils leading to increased surface runoff, (4) backwash from a vehicle as it emerges after fording the river, and (5) undercutting of banks by bow wave action.

Wang et al. (2013) deduced that most of the sediment entering the streams following road construction was from the stream crossings and approaches to the crossings. When the approach fill slopes became re-vegetated, they stabilized, and the annualized sediment loads declined; however, sediment exports remained above the pre-disturbance levels.

LWCs have the potential to deliver sediment into adjacent streams, especially when best management practices (BMPs) are not implemented. Studies have shown that BMPs could effectively reduce erosion and total suspended sediment loads near LWCs (Brown et al. 2013; Wear et al. 2013). Brown et al. (2013) found that approaches to the stream crossing have a high potential for impacting the water quality in the stream. They evaluated the sediment delivery rates associated with reopening legacy roads and found that annual sediment delivery from bare approaches was 7.5 times higher than that from gravel (hardened) approaches. They concluded that implementation of BMPs such as hardening of the surface and appropriate spacing of water control structures could reduce sediment delivery to streams.

Clarkin et al. (2006) stressed that improving stream bank stabilization techniques and ford substrate materials would help enhance the LWC longevity and reduce erosion in the surrounding area.

3.4 Aquatic Organism Passage

Culverts and LWCs, if not designed and installed properly, can act as a barrier to fish and other aquatic organism passage. Installation of LWCs at any site disturbs the natural regime of the channel. Common ways in which LWCs create an obstruction to aquatic organism passage (AOP) include drops at inlet and outlet, excessive water velocity, debris, excessive turbulence due to contraction at inlet region, insufficient low flows, etc. (Kilgore et al. 2010). This restriction of movement and migration of fish may lead to a decrease in fish population and change in the distribution of the aquatic organisms in the stream.

LWCs may have an impact on the aquatic organism in the stream due to channel modification during LWC installation. Studies have shown that stream crossings may change the form and function of stream ecosystem and habitat significantly and affect aquatic organism movement (Bouska et al. 2010; Cocchiglia et al. 2012; Warren Jr & Pardew 1998). Warren Jr & Pardew (1998) looked into movement of fish for 21 different species in seven families through the culvert, slab, open-box, and ford crossings and through natural reaches and found that overall fish movement was an order of magnitude lower through culverts than through other crossings or natural reaches. They also found that open-box and ford crossings showed little difference from natural reaches in overall fish movement. Bouska et al. (2010) studied fish passage at five concrete box culverts and five low water crossings (concrete slabs vented by culverts) and ten control sites (below a natural riffle) and found that culverts were acting as a barrier to fish movement.

Changes in stream hydrology and velocity occur when there is an alteration in the channel geometry that restricts movement and may also be inhospitable for many fish and invertebrates species (Cocchiglia et al. 2012). Water velocity and depth inside the culvert, and length and slope of the culverts cause barriers to fish passage (Rayamajhi et al. 2012). For improved AOP, the crossing should be similar in form and function to the natural bed of the stream channel (Bouska and Paukert 2010; Clarkin et al. 2006; Cocchiglia et al. 2012) (Figure 3.4).

Figure 3.4: Culvert for AOP, which mimics the natural bedstream (from Barnard et al. 2013).

The optimum design of LWC for wildlife and AOP should possess following qualities (Singler and Graber 2005):

- Crossing spans the entire stream up to bank
- Crossing has a natural streambed
- Water depth and velocity in upstream and downstream side of the crossings are similar
- Crossing has dry banks for wildlife passage

Fords with slots or small channel to allow AOP during very low flows provide a little hindrance to organism passage if they mimic the form of the reach (Clarkin et al. 2006; Howard et al. 2011). Unvented at-grade LWCs with streambed materials on the driving surface help in the passage of aquatic species. A series of embedded box culverts can be used in areas where aquatic organism habitat protection is of prime importance. Low water bridges have an elevated driving surface, maintain a more natural streambed, allow aquatic organism movement to greater extent than culverts, and are the best LWC type for fish passage (Clarkin et al. 2006).

Historically, culverts and LWCs have been designed for the efficient conveyance of water during normal and flooding conditions, with little attention given to AOP through the crossing. Designing LWCs for AOP generally results in a larger structure than necessary for hydraulic conveyance, but it has additional benefits of low maintenance and proves to be economically feasible in the long run (Schall et al. 2012).

It is advised to consult HEC-26, *Culvert Design for Aquatic Organism Passage* (Kilgore et al. 2010) while designing the crossing so that the LWC also facilitates adequate AOP. Current version of lists of endangered and threatened species of fishes, amphibians, and reptiles, which can be obtained from Illinois Department of Natural Resources (IDNR), should be consulted. Currently, the following tools are available for designing the culverts and low water crossings for AOP:

HY-8 Culvert Analysis Program (FHWA 2016): HY -8 can be employed in designing vented LWC. HY-8 v7.40 contains a calculator that helps with the FHWA's culvert AOP design procedure discussed in HEC-26, *Culvert Design for Aquatic Organism Passage* (Kilgore et al. 2010). HEC-26 contains stream simulation design method, which can be followed while designing the crossing so that the LWC also facilitates adequate AOP.

FishXing (USDA FS 2012): FishXing is free software developed by the USDA Forest Service that helps in assessment and design of culverts for fish passage. It models organism capabilities against culvert hydraulics for a range of expected stream flow, and compares the flows, velocities, and leap conditions with the swimming abilities of the fish species. It accommodates the iterative process of designing a new culvert to provide passage for fish and other aquatic species.

HEC-RAS (USACE 2016): HEC-RAS can be employed to find the flow velocity and shear stress in the LWC and the immediate cross sections; these values can be compared to the permissible values to see whether the LWC affects the AOP.

3.5 HEC-RAS

Hydrologic Engineering Center's River Analysis System (HEC-RAS) is hydraulic modeling software developed by the US Army Corps of Engineers, which allows the user to perform one-dimensional steady and unsteady flow calculations (Brunner 2010a). It was first released to the public in 1995 and has gained popularity among hydrologic modelers, which is evident by its prevalent use in modeling dams, bridges, and culverts.

The HEC-RAS system consists of

- A graphical user interface (GUI), which helps the user in data entry, editing, file management, hydraulic analysis and displaying of result
- Hydraulic analysis components, where users do all the modeling
- Data storage and management
- Graphics and reporting capabilities

The stable version HEC-RAS V 4.1.0 was released in 2010 and supports steady and unsteady flow water profile computations, sediment transport and water quality modeling (Brunner 2010a). In 2016, Hydrologic Engineering Center launched HEC-RAS V 5.0 which has two-dimensional modeling capabilities and can be used to perform 1D, 2D or combined 1D/2D modeling (Brunner 2016). The recent version 5.0.3 includes several new features such as Culvert inlet/outlet control changes, and several improvements in RAS Mapper.

Additionally, HEC-GeoRAS, an ArcGIS extension designed the US Army Corps of Engineers, is also available which makes it easier to process the geospatial data for use in HEC-RAS. It lets the user create geometry files in ArcGIS from the digital terrain model (DTM), which can then be imported into HEC-RAS to perform the modeling. HEC-GeoRAS can also be used to plot the inundation depths and flood extent by using the water surface profile results from the HEC-RAS computation (Ackerman 2012).

3.5.1 Equations used for Basic Profile Calculations in HEC-RAS

HEC-RAS computes the water surface profiles from one cross section to the next by solving the energy equation using the standard step method (Brunner 2010b). The energy equation is as given below:

$$Z_2 + Y_2 + \frac{a_2 V_2^2}{2g} = Z_1 + Y_1 + \frac{a_1 V_1^2}{2g} + h_e$$

Where: Z_1, Z_2 = elevation of the main channel inverts

Y_1, Y_2 = depth of water at cross sections

V_1, V_2 = average velocities

a_1, a_2 = velocity weighting coefficients

g = acceleration due to gravity

h_e = energy head loss

The energy head loss is given as:

$$h_e = LS_f + C\left|\frac{a_2 V_2^2}{2g} - \frac{a_1 V_1^2}{2g}\right|$$

Where: L = discharge weighted reach length

S_f = representative friction slope between two sections

C = expansion or contraction loss coefficient

The distance weighted reach length, L, is calculated as:

$$L = \frac{L_{lob}Q_{lob} + L_{ch}Q_{ch} + L_{rob}Q_{rob}}{Q_{lob} + Q_{ch} + Q_{rob}}$$

Where: L_{lob}, L_{ch}, L_{rob} = cross section reach lengths for flow in the left overbank, main channel and right overbank, respectively

Q_{lob}, Q_{ch}, Q_{rob} = arithmetic average of the flows between sections for the left overbank, main channel and right overbank, respectively

The total conveyance is calculated by subdividing the flow into different units. The flow in the overbank areas is subdivided using the information about locations where the change in the Manning's n-value occurs. Conveyance is calculated in each subdivision using the following form of Manning's equation based on English units:

$$Q = KS_f^{1/2}$$

$$K = \frac{1.486}{n} AR^{2/3}$$

Where: K = conveyance for subdivision

n = Manning's roughness coefficient for subdivision

A = flow area for subdivision

R = hydraulic radius for subdivision

3.5.2 HEC-RAS in Dam Investigations

HEC-RAS has been used to evaluate the impacts of dam installations, dam break, and flood routing analysis in the United States and throughout the world. Nislow et al. (2002) used HEC-RAS to study the effects of dam construction on the flood regime of natural floodplain communities in the Upper Connecticut River. In doing so, they compared the frequency and duration of flooding and the area flooded under different recurrence intervals for pre-and post-impoundment discharges. They found that the riparian communities which were flooding between 20 to 100 years' intervals before the dam construction would flood at more than 100 years' interval, thus isolating them from the riverine influence.

Maingi & Marsh (2002) used HEC-RAS to examine the impacts of dam installations on the hydrologic regime of Tana River in Kenya. They used HEC-RAS to estimate the flooding frequency of 71 vegetation sample plots located on the floodplain of the river. HEC-RAS analysis determined that the sample plots at an elevation greater than 1.80 m above dry season river level experienced more flooding after the Masinga Dam construction.

Xiong (2011) applied dam break tool in HEC-RAS to the Foster Joseph Sayers Dam in Pennsylvania and concluded that the dam breaks due to pining only increases the time period of high water surface level, thus prolonging the risk duration. It was also found that the dam break does not increase the downstream maximum water surface elevation by significant amounts at the design Probable Maximum Flood (PMF). Another finding of the study is the greater impact on the immediate downstream location compared to further downstream, based on hydrograph comparison.

3.5.3 HEC-RAS in Bridge and Culvert Study

Various hydraulic models are available which can be used for modeling bridges and culverts. There are one-dimensional, two-dimensional, and three-dimensional models with steady and/or unsteady flow regimes. Zevenbergen et al. (2012) provides an extensive review of the differences between the various types of numerical modeling approaches. Most bridge hydraulic studies use 1D analysis methods, though 2D models are becoming more common. 3D models are used to analyze complex flow fields. HEC-RAS is one of the frequently used models for bridge and culvert investigations. It is particularly useful for in-channel flows and when floodplain flows are minor.

Brandimarte and Woldeyes (2013) used HEC-RAS to estimate the backwater effects at bridge crossing in Tallahala Creek, Mississippi. They compared the estimated backwater due to design flood profile using both the deterministic and probabilistic approach. Another study conducted by Hadera and Asfaw (2016) looked into the causes of outlet erosion in highway crossings (bridges and culverts) in Ethiopia. They used HEC-RAS and HY8 for the hydraulic analysis of the bridge and culvert respectively. From their investigation, they concluded that the major cause of downstream erosion and gully formation is the lack of proper hydrologic and hydraulic design of the highway drainage.

Olaniyan et al. (2014) utilized HEC-RAS in modifying the existing culverts in River Omi in Nigeria, which had been inundating the surroundings. They found that the existing culvert could no longer accommodate the flow in the river and provided an alternative design of box culvert with a span of 7.2 m and rise 1.8 m.

3.5.4 HEC-RAS in LWC Study

HEC-RAS has a wide range of applications in modeling hydraulics, as discussed above. The reliability of the HEC-RAS in LWC analysis is based on the studies involving bridges, dams, culverts and other civil engineering designs. HEC-RAS and GIS have also been used as a powerful tool in floodplain delineation (e.g. Tate and Maidment, 1999; Tate et al., 2002; Yang et al., 2006). Leahy (2014) used HEC-RAS to model LWCs at a U.S. Army installation in Indiana.

Hydraulic analysis tools used for LWC design depend on the type of LWC to be designed or analyzed. The modeling of LWC in HEC-RAS is similar to bridge and culvert analysis. Based on the type of LWC, single culvert, multiple culverts, or bridge analysis is performed. Usually, one-dimensional modeling is done for steady flow conditions with the appropriate design flow.

Unvented fords are modeled as inline structure weirs. Vented fords are typically analyzed as culvert structures with weir flow over the road when the water overtops the structure (Schall et al. 2012). Weir flow takes place when the vented ford is overtopped. The hydraulic analysis and design of low water bridges are done the same way for normal bridges, with special consideration given to overtopping flows (Zevenbergen et al. 2012).

CHAPTER 4: METHODOLOGY

4.1 Illinois LWC Survey

A survey was conducted as a part of the research to obtain an overview of the distribution of LWCs in Illinois, along with county engineers' experiences with LWCs pertaining to design, construction, and maintenance. The survey questionnaire consisted of a document file with 14 questions (Appendix A) and a spreadsheet to document information on multiple LWCs. The summary of the survey is included in Appendix B. Some of the key findings of the LWC survey are as given below:

- Low-water crossings have been used extensively in southern and central Illinois, which are predominantly agricultural areas. In the northern counties surrounding Cook County, the high ADTs do not favor the construction of LWCs.
- LWCs are suitable for areas with average daily traffic less than 25 vehicles per day.
- LWCs, especially unvented and vented fords, are economical and hence are suitable for rural, low ADT roads that primarily serve as access roads to farmlands.
- Fords (unvented and vented) are the first choice for LWCs due to the simple design and low construction and other associated costs.
- LWCs are permitted to overtop, but only during a limited time of the year. Usually, the overtopping is limited to less than 5% of the year. However, the time during which a LWC is allowed to be overtopped is based on the usability and importance of the road in which the LWC is present. The judgment of an engineer is important in this decision, and the design flow needs to be selected accordingly.
- Few LWCs provide access to residential homes, which is suggested only in the presence of an alternative route nearby.
- Lack of warning signs increases the risk of accidents in the crossings and is a liability to a highway department. Thus, proper signage should be installed at the LWC site.

- If maintained properly, LWCs can also be a point of attraction in parks and recreational areas.

4.2 Study Sites

As a part of the research, case studies were conducted on five existing LWCs in Illinois, which includes two unvented and three vented fords. The sites were chosen to represent different geographic regions of Illinois. The following criteria were considered when selecting the sites for a channel cross-section survey to be used in the numerical modeling in HEC-RAS.

- Availability of LiDAR data
- Diversity in LWC types
- Size of the stream and contributing watershed area
- Channel and site stability
- LWC functionality
- Utilization of LWC
- Safety factors and signage
- Cooperation of the county highway department

Two vented LWCs in Edgar County were chosen for the case study based on an initial site assessment of 12 LWC sites in Edgar, Coles, and Christian counties. Another vented LWC in Logan County was also surveyed as the LWC was different in orientation from other sites. The crossing is skewed at a 35-degree angle to the streamflow direction. Similarly, two unvented LWCs, one each from Franklin County and Ogle County were also included. The unvented ford in Franklin County serves an agricultural area, whereas the one in Ogle County is inside a state park and used mostly by visitors for recreational purposes such as fishing.

The LWC sites selected for the detailed analysis, based on the preliminary survey results and the criteria discussed above, and their details are provided in Table 4.1 and Figure 4.1.

Table 4.1: LWC Sites Selected for Case Study

County	ID	Latitude	Longitude	Structure Type	Stream
Edgar	Edgar #1	39.5084	–87.9236	Above-Grade Vented	North Fork
	Edgar #3	39.5136	–87.7297	At-Grade Vented	Fork Big Creek
Franklin	Franklin	38.0171	–88.7879	At-Grade Unvented	Tributary to Akin Creek
Logan	Logan	40.0673	–89.5458	At-Grade Vented	Tributary to Salt Creek
Ogle	Ogle	41.9924	–89.4707	At-Grade Unvented	Pine Creek

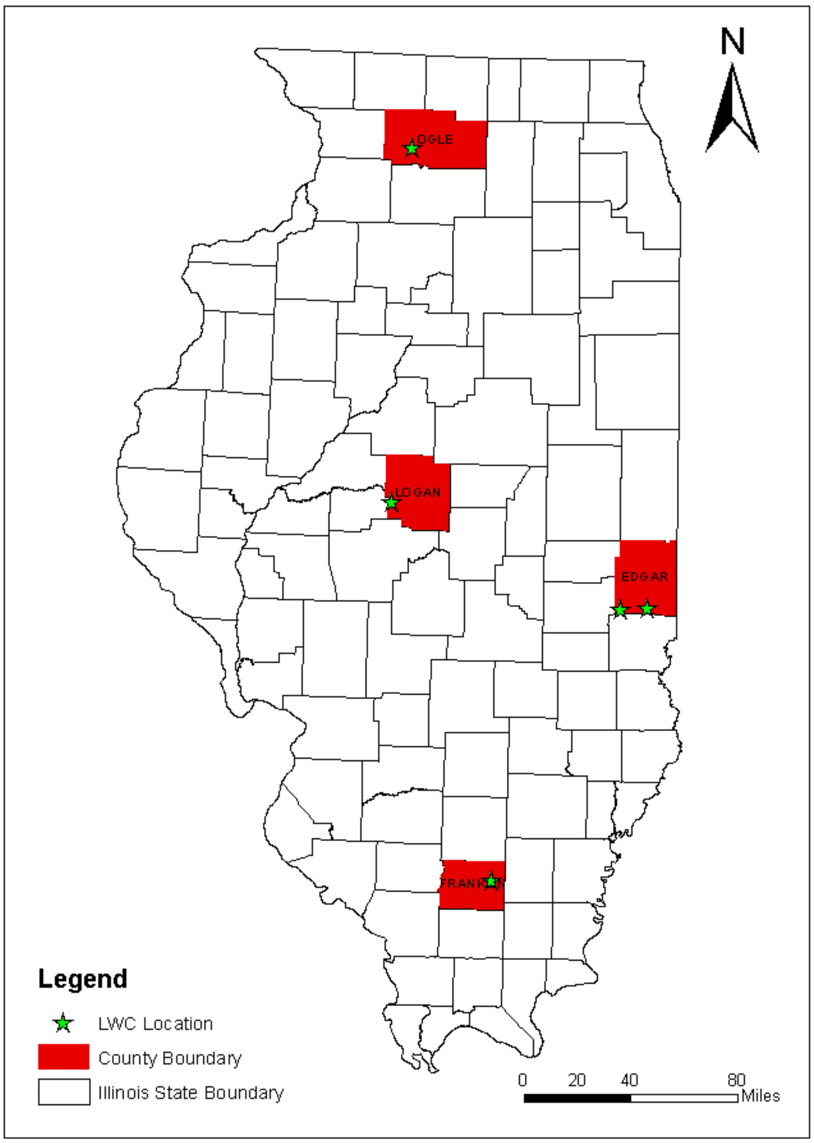

Figure 4.1: Location of LWCs selected in the study.

4.3 Logan LWC

The LWC selected for further study in Logan is centered at 40.0673N, 89.5468W and is located on 1025 - 275th Avenue (Figure 4.2). It lies in the middle of an agricultural area and mostly carries the water discharged by tile drains in the field. The crossing was constructed in the late 80's and has performed well, requiring minimum maintenance.

Figure 4.2: LWC (vented) in Logan County, Illinois.

4.3.1 Structure Details

Crossing history	The LWC was constructed as a Missouri crossing, and the pipes were provided to handle the normal flow.
Why was this structure selected?	The structure was chosen because of its performance and low maintenance requirements. The LWC is located on a low ADT road, with less than 25 vehicles per day.
Crossing details	**Structure:** The structure was designed with a 10-year flood event. However, the low flow culverts were sized to accommodate the low flow of the stream. The vented LWC has three concrete pipes of 2 ft diameters. The crossing is 27 ft wide and has a skew angle of 35 degrees (Figure 4.2). **Cost:** The structure was built in 1988 at the cost of $12,541.

	Safety: There have not been any serious accidents at this location. However, there have been complaints about the roughness of the approach grades. **Signage:** Warning signs saying *SLOW* and *DO NOT ENTER WHEN FLOODED* are present on the approach roads. **Alternative route:** When the road is flooded, the adverse travel is only 2 miles.
Flood and maintenance history	The crossing is closed for a few hours with each flood event. In case a larger flood event occurs (i.e. 50-year, 100-year), the road might be shut down for a day. The crossing requires maintenance after high flows, which are expected. The maintenance cost is very minimal.
Presence of aquatic species	This is a very small stream for most of the year, fed mostly by drainage tiles. There are few, if any, fish in this area. The low-flow culverts should allow passage of any aquatic species because these culverts are set along the flowline of the stream.
Public perception	The local citizens have not complained about the low water crossing.

4.3.2 Watershed

The vented LWC is placed across the stream, which is a tributary of the Salt Creek (Figure 4.3). The LWC area has a main channel slope of 20.33 ft/mi and the watershed upstream of the LWC has a drainage area of 3.13 mi^2 as per Illinois StreamStats (USGS, 2016). It lies within the Salt Creek of Sangamon River Watershed.

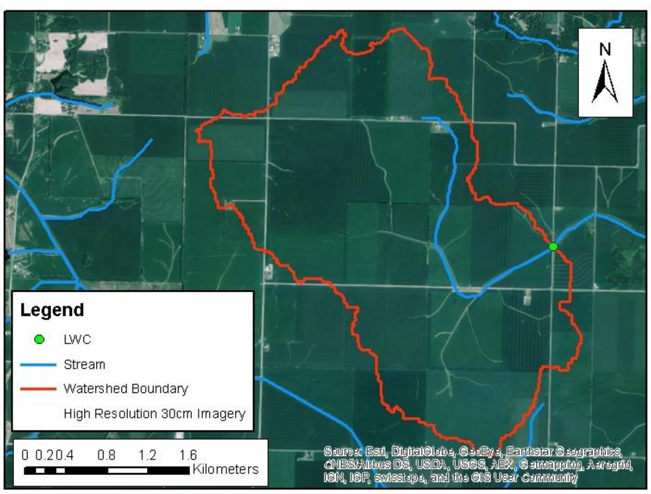

Figure 4.3: Stream network in the Logan LWC watershed.

The stream was classified using Chow (1959) to determine the appropriate Manning's n value of the reach. The value assigned for Manning's n was 0.03 within the channel and 0.035 for floodplain dominated by cultivated crops. The watershed is a predominately-agricultural area (96.7%), with a little bit of developed area (Figures 4.3 and 4.4; Table 4.2).

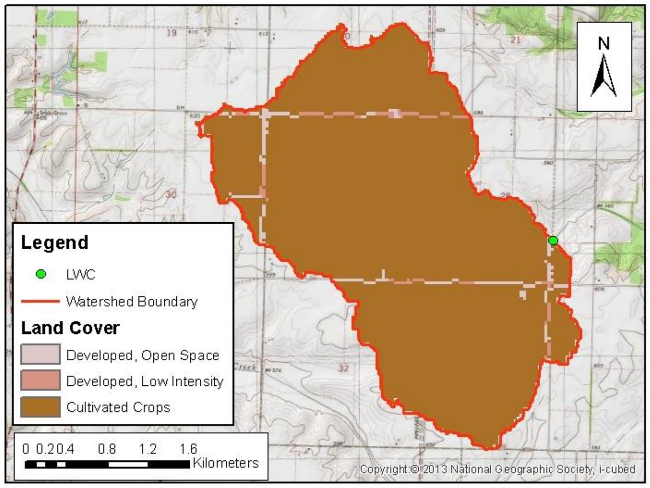

Figure 4.4: NLCD land cover in the watershed associated with the Logan LWC.

Table 4.2: NLCD Land Cover by Percent Area in Logan LWC Site

NLCD Land Cover	% Area
Developed, Open Space	2.11
Developed, Low Intensity	1.19
Cultivated Crops	96.70

4.3.3 Soil

The majority of the watershed has soils in the B or C/D hydrologic group (Figure 4.5, Table 4.3). Soil group B has a moderate infiltration rate when thoroughly wet. Soil groups with drainage characteristics affected by a high water table are indicated with a /D designation, where the letter preceding the slash indicates the hydrologic group of the soil under drained conditions. The main drainage way is comprised primarily of B soils affected by a high water table (B/D).

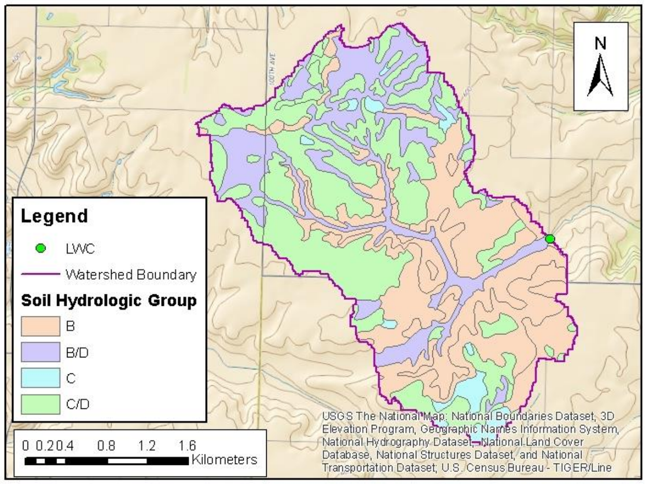

Figure 4.5: Hydrologic soil groups for watershed associated with Logan LWC.

Table 4.3: Hydrologic Soil Groups by Percent Area in Logan LWC Site

Hydrologic Soil Group	% Area
B	35.68
B/D	14.49
C	13.49
C/D	36.34

4.4 Edgar#1 LWC

This above-grade vented LWC in Edgar County is centered at 39.508N, 87.924W, and is located on N 200th Street (Figure 4.6). It lies across the North Fork Embarras River in the middle of an agricultural area. The LWC is located on low volume road and has been functioning well.

Figure 4.6: Edgar#1 LWC after a rainfall event (left), same LWC during dry months (right).

4.4.1 Structure Details

Why was this structure selected?	The structure was chosen because of its better functioning and low maintenance requirements. The LWC is located on a low ADT road, with less than 25 vehicles per day.
Crossing details	**Structure:** The LWC is an arched structure made up of corrugated metal with a span of 8.75 feet and a rise of 2.5 feet. The crossing is 20 feet in width and has a skew angle of 15 degrees. The LWC has suffered from scouring at the downstream end, and traps logs and branches of trees after heavy rainfall events (Figure 4.6). **Cost:** Not known. **Safety:** There have not been any severe safety issues in the crossing. **Signage:** None present.
Flood and maintenance history	The crossing is closed a couple of times a year, for up to 24 hours depending on the flood event. The crossing needs to be maintained after heavy rainfall events, the cost of which is very low.
Presence of aquatic species	None known.
Public perception	The local citizens feel the LWC is better than having no crossing at all, and the crossing is very useful for the vehicle movement.

4.4.2 Watershed

The LWC is located across the North Fork Embarras River, which is a major tributary to the Embarras River (Figure 4.7). The LWC area has a main channel slope of 29.86 feet per mile, and the watershed upstream of the LWC has a drainage area of 4.44 square miles as per Illinois StreamStats. It lies within the Embarras/Middle Wabash River Watershed.

The stream was classified using Chow (1959) to determine the appropriate Manning's n value of the reach. The value assigned for Manning's n was 0.04 within the channel, 0.1 for floodplain dominated by heavy timber stands and 0.05 for scattered brushes. The watershed is a predominantly agricultural area, with a little bit of pasture and forested area (Figures 4.7 and 4.8; Table 4.4).

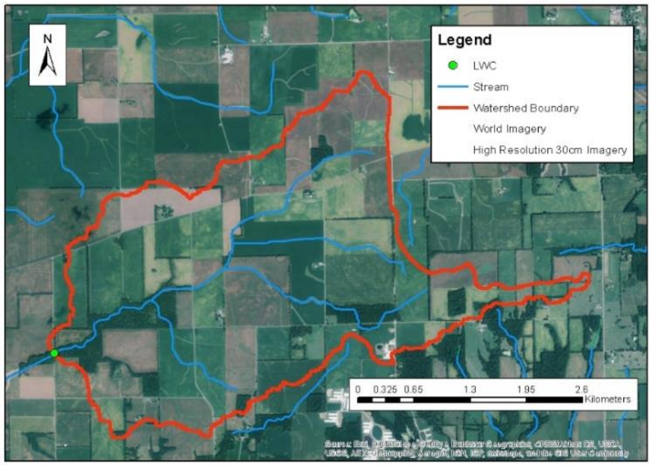

Figure 4.7: Stream network in the Edgar#1 LWC watershed.

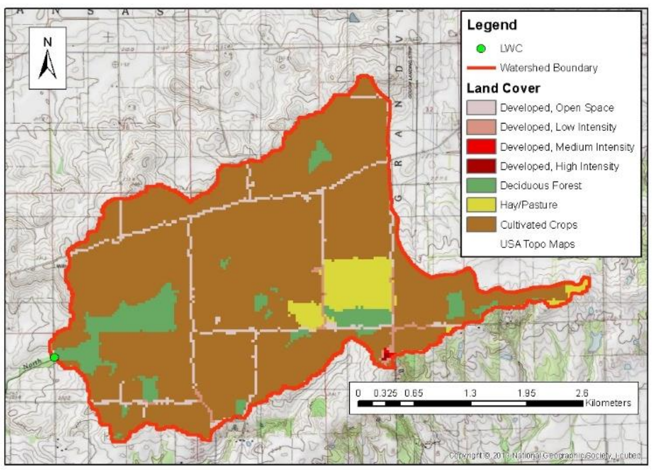

Figure 4.8: NLCD land cover in the watershed associated with the Edgar#1 LWC.

Table 4.4: NLCD Land Cover by percent area in Edgar#1 LWC site

NLCD Land Cover	% Area
Developed, Open Space	4.38
Developed, Low Intensity	0.96
Developed, Medium Intensity	0.05
Developed, High Intensity	0.05
Deciduous Forest	7.93
Hay/Pasture	5.28
Cultivated Crops	81.35

4.4.3 Soil

The majority of the watershed has soils in the “C” or “B/D” hydrologic group (Figure 4.9, Table 4.5). Soil group C has a slow infiltration rate when thoroughly wet. Soil groups with drainage characteristics affected by a high water table are indicated with a “/D” designation, where the letter preceding the slash indicates the hydrologic group of the soil under drained conditions. The main drainage way is comprised primarily of “B” soils affected by a high water table (“B/D”).

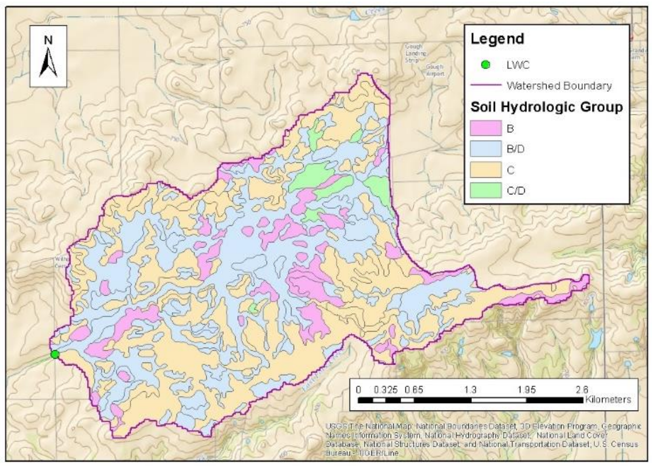

Figure 4.9: Hydrologic Soil Groups for watershed associated with Edgar#1 LWC.

Table 4.5: Hydrologic Soil Groups by percent area in Edgar#1 LWC site

Hydrologic Soil Group	% Area
B	19.59
B/D	35.00
C	41.80
C/D	3.61

4.5 Edgar#3 LWC Site

This at-grade vented LWC in Edgar County is centered at 39.5136N, 87.7297W, and is located on E 300th road (Figure 4.10). It lies across the East Fork Big Creek, in an agricultural region. The LWC is located on a low ADT road, with less than 25 vehicles per day.

Figure 4.10: Low water crossing and the approach, Sediment deposition upstream, Undercutting on downstream end (clockwise direction from top).

4.5.1 Structure Details

Why was this structure selected?	The structure was chosen because it is functioning poorly. An alternative design for the LWC has been provided, which is expected to perform better than the existing LWC.
Crossing details	**Structure:** The LWC has two one-ft diameter corrugated metal pipes. The crossing is 11.5 feet in width and has a skew angle of 15 degrees. There is sediment deposition on the upstream side of the LWC, and undercutting by water under the structure (Figure 4.10). **Cost:** Not known. **Safety:** There have been several incidences where people tried to cross when they shouldn't and were washed off. There have been deaths at the slabs, but it’s due to people trying to swim during high water, and not from cars being washed downstream.

	Signage: None present.
Flood and maintenance history	The crossing is closed a couple of times a year, for up to 24 hours depending on the flood event. The crossing must be maintained after heavy rainfall events.
Presence of aquatic species	None known.
Public perception	The local citizens feel that LWC is better than having no crossing at all. The LWC is good for a vehicle, but unreliable, as LWCs cannot always be depended upon.

4.5.2 Watershed

The LWC is placed across the East Fork Big Creek, which is a tributary of the Big Creek and eventually drains into the Wabash River (Figure 4.11). The LWC area has a main channel slope of 15.34 feet per mile, and the watershed upstream of the LWC has a drainage area of 13.64 square miles as per Illinois StreamStats. It lies within the Embarras/Middle Wabash River Watershed.

The stream was classified using Chow (1959) to determine the appropriate Manning's n value of the reach. The value assigned for Manning's n was 0.04 within the channel, 0.10 for floodplain dominated by heavy timber stands and 0.06 for light brushes and trees. The watershed is mostly dominated by agricultural area, with a little bit of pasture and forested area (Figures 4.11 and 4.12; Table 4.6).

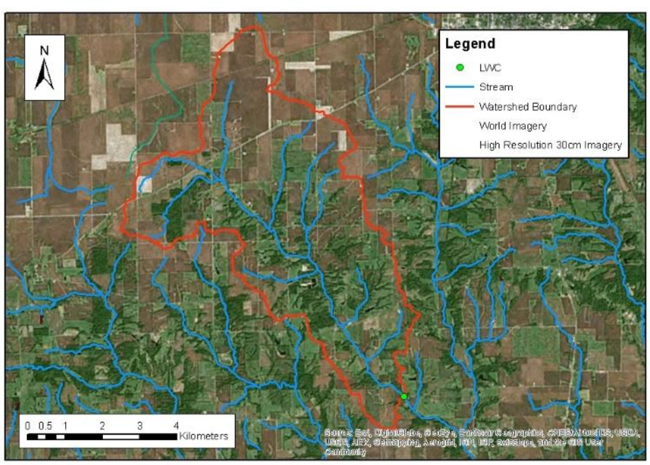

Figure 4.11: Stream network in the Edgar#3 LWC watershed.

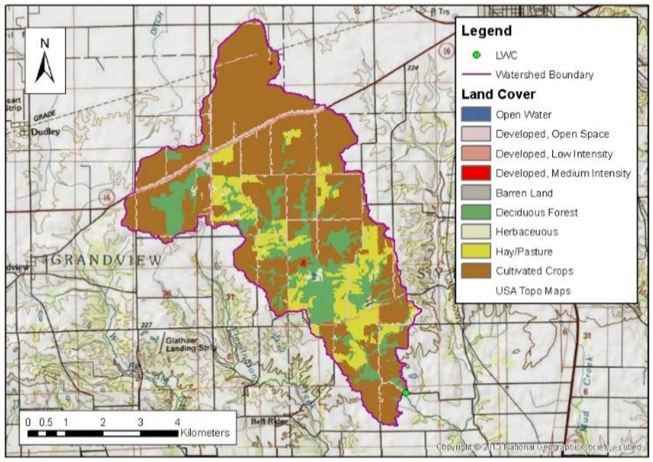

Figure 4.12: NLCD land cover in the watershed associated with the Edgar#3 LWC.

Table 4.6: NLCD Land Cover by percent area in Edgar#3 LWC site

NLCD Land Cover	% Area
Open Water	0.08
Developed, Open Space	4.98
Developed, Low Intensity	1.67
Developed, Medium Intensity	0.06
Barren Land	0.02
Deciduous Forest	16.93
Herbaceuous	0.89
Hay/Pasture	13.24
Cultivated Crops	62.13

4.5.3 Soil

The majority of the watershed has soils in the “B”, “C” or “B/D” hydrologic group (Figure 4.13, Table 4.7). Soil groups A, B and C have a high, moderate and slow infiltration rate respectively when thoroughly wet. Soil groups with drainage characteristics affected by a high water table are indicated with a “/D” designation, where the letter preceding the slash indicates the hydrologic group of the soil under drained conditions. The main drainage way is comprised primarily of “A” and “B/D” soils, affected by a high water table.

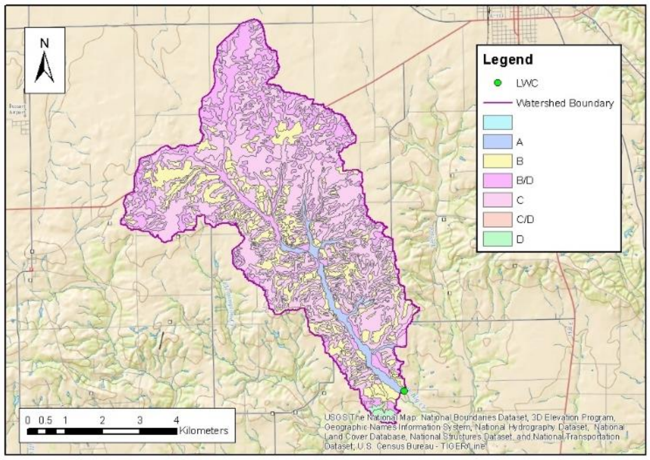

Figure 4.13: Hydrologic Soil Groups for watershed associated with Edgar#3 LWC.

Table 4.7: Hydrologic Soil Groups by percent area in Edgar#3 LWC site

Hydrologic Soil Group	% Area
Water	1.93
A	0.06
B	25.16
B/D	29.37
C	42.67
C/D	0.30
D	0.51

4.6 Franklin LWC

The at-grade unvented LWC is centered at 38.0171N, 88.7879W and is located on an old farm road (Figure 4.14). It is placed across a tributary to Akin Creek, and lies in the middle of an agricultural area, providing access to one house.

Figure 4.14: Franklin LWC site, crossing surface, sediment deposition downstream, signage (clockwise).

4.6.1 Structure Details

Crossing history	The unvented LWC was a replacement for the culvert pipes that were washed out. The road commissioner at the time pulled the pipes out, cut the bank back, and poured concrete in the bottom.
Why was this structure selected?	The unvented structure was chosen because of its better performance and low maintenance requirements. The flow of water over the structure is very low most of the time, and it is located on a low ADT road with less than 25 vehicles per day.
Crossing details	**Structure:** The crossing is 11.5 feet in width. The crossing has gravel approaches, and the crossing surface has worn out (Figure 4.14). **Cost:** Not known. **Safety:** There have not been any accidents at this location. **Signage:** Sign that reads "LOW WATER CROSSING" is present on the approaches, about 375 feet from the crossing.
Flood and maintenance history	The crossing is closed once or twice a year when the creek it flows into backs up during a heavy rain, during which it is impassable for about 12 hours. Not much maintenance has been performed on the crossing.
Presence of aquatic species	None known.
Public perception	The crossing provides service for one house. There is another road nearby for a detour when the water gets high.

4.6.2 Watershed

The LWC is placed on a gravel road across a tributary to Akin Creek, which is a tributary to the Middle Fork Big Muddy River (Figure 4.15). The LWC area has main channel slope of 21.055 feet per mile, and the watershed upstream of the LWC has a drainage area of 1.3 square miles as per Illinois StreamStats. It lies within the Big Muddy River Watershed.

The stream was classified using Chow (1959) to determine the appropriate Manning's n value of the reach. The value assigned for Manning's n was 0.035 within the channel and 0.035 for floodplain dominated by pasture and cultivated crops. The watershed is a predominantly agricultural area, along with some pastureland (Figures 4.15 and 4.16; Table 4.8).

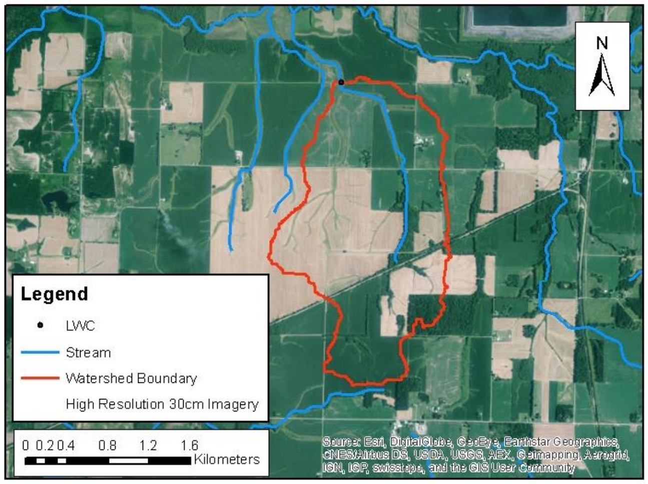

Figure 4.15: Stream network in the Franklin LWC watershed.

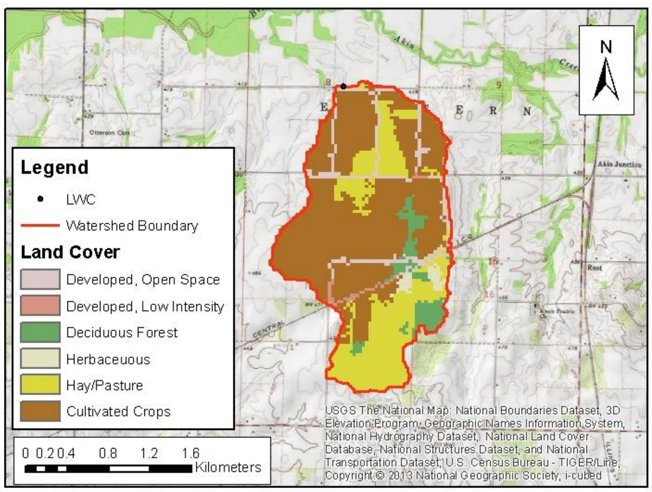

Figure 4.16: NLCD land cover in the watershed associated with the Franklin LWC.

Table 4.8: NLCD Land Cover by percent area in Franklin LWC site

NLCD Land Cover	% Area
Developed, Open Space	6.55
Developed, Low Intensity	1.06
Deciduous Forest	5.79
Herbaceuous	1.49
Hay/Pasture	26.12
Cultivated Crops	58.99

4.6.3 Soil

The majority of the watershed has soils in the "C" or "C/D" hydrologic group (Figure 4.17, Table 4.9). Soil group C has a low infiltration rate when thoroughly wet. Soil groups with drainage characteristics affected by a high water table are indicated with a "/D" designation, where the letter preceding the slash indicates the hydrologic group of the soil under drained conditions. The main drainage way is comprised primarily of "B/D" soils, which are affected by a high water table.

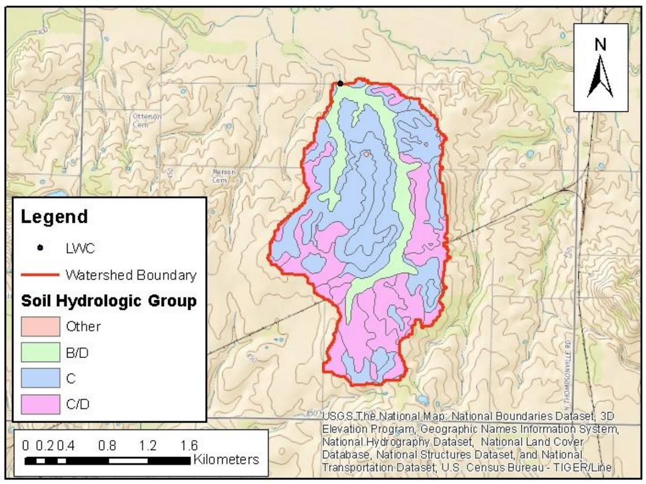

Figure 4.17: Hydrologic Soil Groups for watershed associated with Franklin LWC.

Table 4.9: Hydrologic Soil Groups by percent area in Franklin LWC site

Hydrologic Soil Group	% Area
Water	10.36
B/D	3.24
C	51.95
C/D	34.45

4.7 Ogle LWC

The at-grade unvented LWC is centered at 41.9924N, 89.4707W and is located on a park road in the White Pines Forest State Park (Figure 4.18). The hardened crossing lies across Pine Creek and is used by the visitors to get around the park. Another similar LWC is located about 750 feet downstream of the structure. The road is used seasonally and closed in the winter. Riprap on the streambanks have been used as a best management practice, and a weir is provided to dissipate the energy in the crossing.

Figure 4.18: Low water crossing under study and warning sign.

4.7.1 Structure Details

Crossing history	The original fords were constructed in 1927. Those were replaced with the current fords in 1955. Remnants of the original ford are still visible immediately downstream of the current ford.
Why was this structure selected?	The structure was chosen because of its performance, low maintenance, and popularity among users.
Crossing details	**Structure:** The ford is a concrete monolith founded on a grid of timber piles driven into the stream bed. The crossing is 16.5 feet in width, located on a low ADT road of about 50 vehicles per day. The crossing performs well during periods of normal flow. Normal flow results in an approximate water depth of 6 inches in the center. **Cost:** Not Known.

	Safety: The crossing is for vehicles only. Wading and swimming in the creek are prohibited. Pedestrians cross using the pedestrian bridge approximately 100 feet east of the ford. However, once in a while, a car with low clearance will get turned sideways by the current. **Signage:** Since the surface of the crossing is very slippery, there is a warning sign present on the entrance (Figure 4.18).
Flood and maintenance history	The ford can be closed for periods of a few days to several weeks depending on precipitation patterns. This is common in the spring with the snow melt and rain. The ford is closed during cold weather months due to ice. Even with the creek flowing normally in cold weather, the approaches become slippery from ice. The crossing is virtually maintenance free. The concrete surface has been patched in the past.
Presence of aquatic species	The fish population of the Creek includes bass, sunfish, crappie, carp, and suckers. Keeper size trout are released in the spring and fall. There are no known threatened and endangered species in the area of the park. Invasive zebra mussels are not present in Pine Creek.
Public perception	It is a very popular and unique novelty to most visitors. People like driving through the water.

4.7.2 Watershed

The LWC is placed across Pine Creek, which is a tributary to the Rock River (Figure 4.19). The LWC area has a main channel slope of 15.102 feet per mile, and the watershed upstream of the LWC has a drainage area of 44.94 square miles as per Illinois StreamStats. It lies within the Rock River Watershed.

The stream was classified using Chow (1959) to determine the appropriate Manning's n value of the reach. The value assigned for Manning's n was 0.04 within the channel and 0.10 for floodplain dominated by brush and timber stands. The watershed is a predominantly agricultural area, with a little bit of developed area and forests (Figures 4.19 and 4.20; Table 4.10).

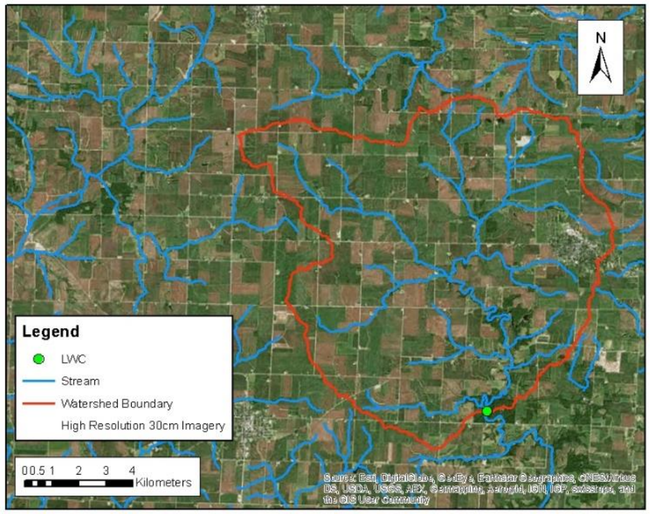

Figure 4.19: Stream network in the Ogle LWC watershed.

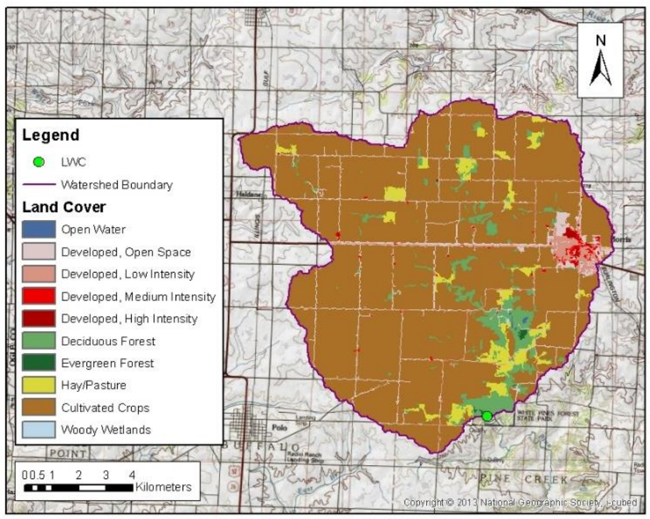

Figure 4.20: NLCD land cover in the watershed associated with the Ogle LWC.

Table 4.10: NLCD Land Cover by percent area in Ogle LWC site

NLCD Land Cover	% Area
Open Water	0.05
Developed, Open Space	4.74
Developed, Low Intensity	2.33
Developed, Medium Intensity	0.50
Developed, High Intensity	0.16
Deciduous Forest	5.58
Evergreen Forest	0.11
Hay/Pasture	4.15
Cultivated Crops	82.36
Woody Wetlands	0.02

4.7.3 Soil

The majority of the watershed has soils in the "B" or "B/D" hydrologic group (Figure 4.21, Table 4.11). Soil group B has a moderate infiltration rate when thoroughly wet. Soil groups with drainage characteristics affected by a high water table are indicated with a "/D" designation, where the letter preceding the slash indicates the hydrologic group of the soil under drained conditions. The main drainage way is comprised primarily of "B" soils affected by a high water table ("B/D").

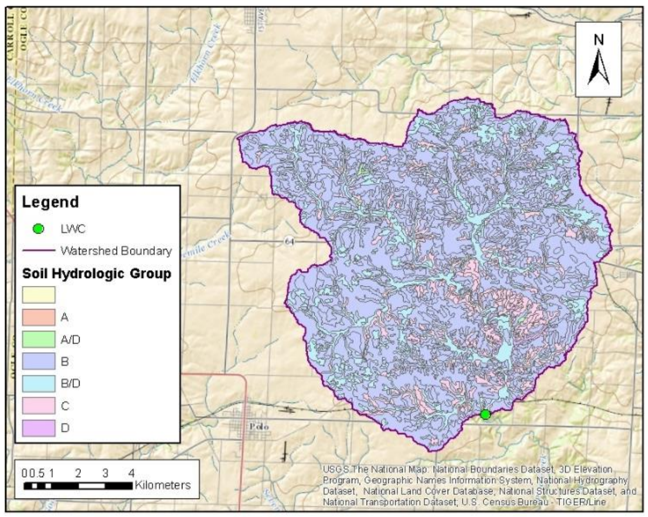

Figure 4.21: Hydrologic Soil Groups for watershed associated with Ogle LWC.

Table 4.11: Hydrologic Soil Groups by percent area in Ogle LWC site

Hydrologic Soil Group	% Area
Water	1.01
A	0.35
A/D	0.22
B	62.60
B/D	17.05
C	17.85
D	0.92

4.8 Site Hydrology

The important factors in LWC design are expected high flow and normal (or base) flow. The high design flow dictates the expected water level above the LWC structure as well as the length of road surface that will be under the water—and indicates the need for special protection techniques such as bank stabilization and reinforcement. The normal or base flow will help the user decide which LWC to construct at the site. In the case of vented fords, the size of the pipes necessary to convey the flow through the structure depends on the low or normal flow.

Although LWCs are designed to be overtopped by higher flows, it is not desirable that LWCs flood most of the year. The experience of local highway officials with LWCs, as well as the literature (Motayed et al. 1982), suggests that the favorable condition for a LWC is when average annual flooding is less than two times a year, whereas it is undesirable to use an LWC when flooding is more than ten times a year. Thus, the LWC should be designed such that it is functional at least 95% of the time in a year.

There are two approaches to obtain design flow used in the design of the LWCs:

- Use flow-duration data to estimate closure time of the LWC (number of days in a year during which the LWC may be closed to traffic) and the capacity of the LWC (pipes in case of a vented ford).
- Use flood-frequency data to estimate high design flow for the design of the LWC structure at full capacity and refer to local knowledge about base flow in the stream to determine the type of LWC and the size of pipe in the case of a vented ford.

Flow for different exceedance probabilities as obtained from regional flow duration curves equation (Over et al., 2014) is given in Table 4.12. The regional flow duration curves for Illinois is explained in detail in Appendix C.

Table 4.12: Flow for selected LWC sites for exceedance probability of 1 percent

LWC	Region	Drainage area (sq. miles)	Flow (ft^3/s)
Logan	II	3.13	28.71
Edgar#1	III	4.44	72.56
Edgar#3	III	13.64	192.04
Franklin	III	1.3	25.00
Ogle	I	44.94	298.18

The 1-year flow values for LWC sites were obtained from partial duration series (PDS) regional equations (Soong et al., 2004) and are provided in Table 4.13.

Table 4.13: 1-year flow from PDS equations for selected LWC sites

LWC	Hydrological Region	Drainage area (sq. mi)	Main channel slope (ft/mi)	%water	Flow (ft^3/s)	Standard error of estimate (%)
Logan	III	3.13	20.332	0	186.01	45.9
Edgar#1	III	4.44	29.864	0.078	231.18	39.6
Edgar#3	III	13.64	15.34	0.071	477.16	39.6
Franklin	VI	1.3	21.055	0	174.27	39.6
Ogle	I	44.94	15.162	0.05	1403.83	44.1

The partial duration series (PDS) regional equations for different regions in Illinois is explained in detail in Appendix D. The Table below (Table 4.14) contains the parameters used to obtain the flow values in Table 4.13 and were obtained from Soong et al. (2004). The parameters and values in Table 4.13 were used in conjunction with PDS regional equations to obtain the 1-year flow.

Table 4.14: Parameters used to obtain 1-years flow values from PDS equations

LWC	Hydrological Region	a	b	c	d	Flow (ft^3/s)
Logan	III	207.1	0.645	-0.524	NA	186.01
Edgar#1	III	207.1	0.645	-0.524	NA	231.18
Edgar#3	III	207.1	0.645	-0.524	NA	477.16
Franklin	VI	87.4	0.822	0.405	-0.472	174.27
Ogle	I	52.6	0.755	0.458	-0.515	1403.83

Peak flow statistics for the sites, obtained from Illinois StreamStats, which includes flows for a return period of 25 years are given in Table 4.15.

Table 4.15: 25-Year Peak flow for selected LWC sites

LWC Site	Flow (ft^3/s)	Prediction error (percent)	Equivalent years of record
Logan	970	44	4.7
Edgar#1	1890	44	4.7
Edgar#3	3170	44	4.7
Franklin	854	NA	NA
Ogle	4700	44	4.7

4.9 Datasets Required

The input essential for steady-state flow modeling of LWC in HEC-RAS includes geometry properties, steady flow rate, and flow regime. Table 4.16 lists the datasets required for modeling a LWC in HEC-RAS and the source from which they were obtained.

Table 4.16: Datasets Required for Hydraulic Modeling in HEC-RAS

Datasets	Source
Elevation	LiDAR DTM (ISGS Clearinghouse)
	Site Survey
Land Use	NLCD Database
Design Flood rates	USGS StreamStats
	USGS empirical equations

4.9.1 Topography and Survey

Site surveys were performed to obtain the cross-section data, and LiDAR elevation data was used to get the elevation of points that were missed during the survey. Cross-sectional surveys for the LWC sites in Edgar and Logan counties were conducted in the fall of 2015 and for the LWCs in Franklin and Ogle counties in the spring of 2016. Cross-sectional data was surveyed with the help of surveying staffs from the US Army Corps of Engineers (USACE) and is consistent with FEMA mapping protocol. Surveys included structure measurement, channel topography up to the top of the bank, and other relevant data to characterize the channel and near overbank geometry. Minimum of four cross sections per site, two cross sections upstream and downstream of the structure, were obtained and then processed.

Cross-sectional geometry in the non-surveyed overbank area was obtained using the topographic data derived from light detection and ranging (LiDAR) and combined with the surveyed channel cross section. LiDAR data was obtained from the Illinois State Geological Survey (ISGS) clearinghouse, which was acquired by the ISGS as a part of the Illinois Height Modernization Program (ILHMP). A digital terrain model (DTM) was developed based on the topographic data from LiDAR and survey, and elevation information was extracted for use in HEC-RAS modeling.

4.9.2 Channel Roughness

The main channel and overbank roughness characteristics were determined from the photographs taken during the field surveys. The photographs were combined with information from aerial photography and land use data to assign the Manning's roughness coefficients (n) along the modeled stream length. Manning's n for the main channel was determined following Chow (1959) and for overbank floodplain regions were determined following HEC-RAS manual (Brunner 2010b; Chow 1959).

Land use data for the selected counties was obtained from USDA Geospatial Data Gateway (USDA 2016) and the surrounding area was clipped. Land use data was used to account for the variation of Manning's roughness (n) along the cross section. Manning's n represents the surface roughness which provides the resistance to flow. Different values of Manning's n used for various land cover conditions are as given in Table 4.17.

Table 4.17: Manning's n Values for Different Land Cover (adapted after Kalyanapu et al. 2010)

NLCD (2011) Code	Land Cover	Manning's n
21	Developed, Open Space	0.0404
41	Deciduous Forest	0.36
82	Cultivated Crops	0.035

Since no gage sites are present on any of the LWC locations, design flow rates for different return periods to use in the HEC-RAS modeling was obtained from USGS StreamStats application for Illinois (USGS, 2016) and USGS empirical equations, and their values are given in Tables 4.2 through 4.5.

4.10 Data Processing and HEC-RAS Modeling

The data processing and modeling of the LWCs was completed using HEC-RAS, ArcGIS, and Microsoft Excel. In the study, the steady flow simulation was performed in HEC-RAS for different flow rates. Smaller flow rates such as 1% exceedance flow and 1-year flow were used to evaluate the adequacy of the existing structures in conveying the design flow through them, and 25-year flow model run was utilized in the flood inundation study. HEC-RAS requires the geometry data, steady flow data and flow regime to perform the steady flow simulation.

The elevation data used for the cross sections in this study is based on survey data as well as LiDAR data. LiDAR data for different counties were acquired by the ISGS in different years: Edgar (2012), Franklin (2014), Logan (2013), Ogle (2009), and they were available in the Illinois state plane coordinate system. The survey was done following the UTM projection. Thus, the first step in the data processing was clipping the area surrounding the LWC from the LiDAR DTM and projecting them into NAD 1983 – UTM – Zone 16N, and subsequently, the elevation was also converted into meter. After this, Triangulated Irregular Network (TIN) files were created from the digital terrain files (DTM) for all the LWC locations.

Numerical modeling of LWCs can be divided into three steps: HEC-RAS preprocessing, HEC-RAS processing, and HEC-RAS postprocessing, as shown in Figure 4.22.

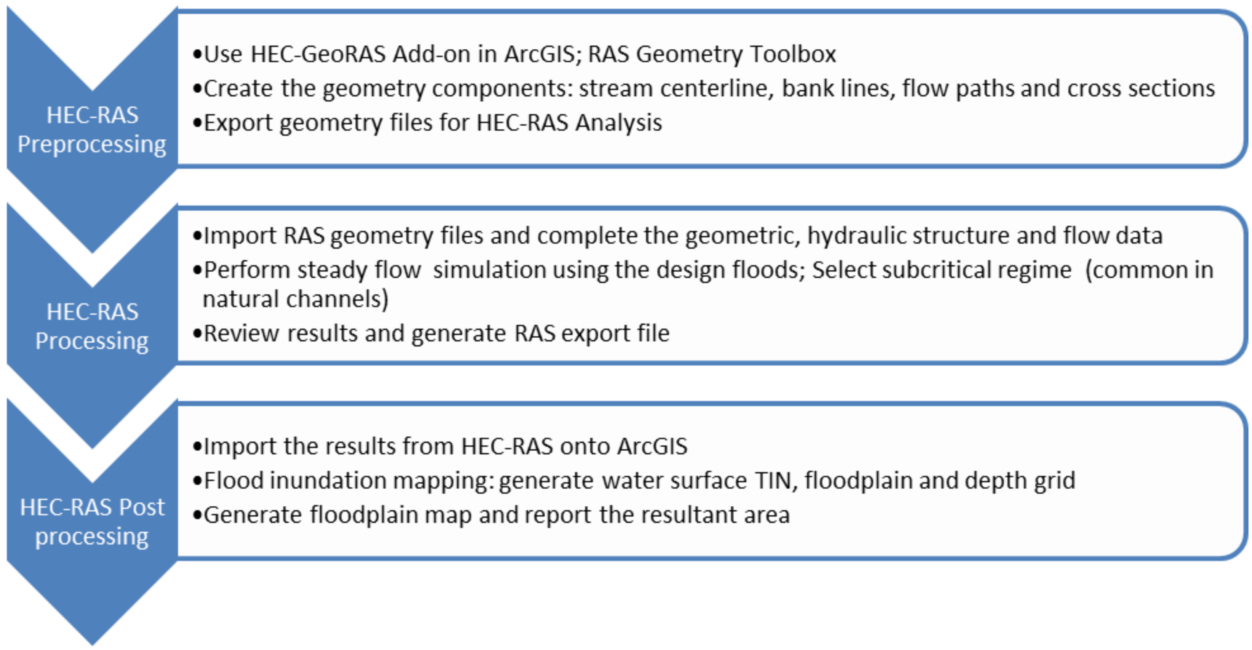

Figure 4.22: Steps in HEC-RAS modeling.

4.10.1 HEC-RAS Preprocessing

HEC-RAS preprocessing was completed in ArcGIS using HEC-GeoRAS add-on. Creation of geometry files is the first step in the analysis. The HEC-GeoRAS add-on tools allow the user to create cross-sections along the geometric terrain, as well as assign bank stations, the structure and appropriate Manning's roughness values (n). The RAS geometry toolbox was used to create and digitize the geometry components such as stream, banks, flow paths, and cross section using the triangulated irregular network (TIN) files. After the digitization was completed, the geodatabases and geometry files were exported into HEC-RAS format using the *Export RAS Data* option.

The HEC-GeoRAS toolbar has four menus (RAS Geometry, RAS Mapping, ApUtilities, and Help) and seven tools/buttons (Assign RiverCode/ReachCode, Assign FromStation/ToStation, Assign LineType, Construct XS Cutlines, Plot Cross Section, Assign Levee Elevation, and Import RAS SDF file).

The RAS Geometry tool within the HEC-GeoRAS toolbar was used for all the pre-processing required. The first step in HEC-RAS analysis was the creation of the geometry files that the model required. A geodatabase file was created for each LWC and orientation. Firstly, from the RAS geometry menu, the Create RAS Layers tool was used to create empty GIS layers for the geometry components: stream centerline, bank lines, flow paths, and XS cut lines (cross-section lines). The layer files were automatically generated as feature classes within the current LWC geodatabase.

The terrain TIN for the LWC site and the basemap aerial imagery were added to guide the digitization process. Digitization of the geometry files was done in the following order: River centerline, river banks, flow paths and finally cross sections. The river centerline was digitized by editing the River feature class, approximately following the center of the river/creek and aligned in the direction of flow i.e. from upstream to downstream direction. After the digitization of the stream centerline had been completed, the river name and the reach were assigned using the *Assign River Code and Reach Code* tool. Then, the complete attributes of the River feature class were populated by selecting the *Stream Centerline Attributes* option under RAS Geometry. This function also creates the 3D version of River centerline called River3D.

Next, the banks were created, which are used to distinguish the main channel from the overbank (floodplain) region. The bank lines were then digitized by considering the bank full terrain elevation. Since there are no specific guidelines about the bank lines orientation, they were digitized along the flow direction, similar to the river centerline. Digitization was done starting from the upstream end and looking downstream; the left bank was digitized first followed by the right bank.

The flow path centerlines were created by selecting Flow Path Centerlines from the Create RAS Layers menu. Three flow path lines are needed: left, right, and channel. The stream centerline was used to create the flow path centerline. The left and right flow paths were digitized by editing the Flowpaths feature class. The left flow path was digitized first looking downstream followed by the right flow path. These flow path lines are used to compute the distances between cross-sections in the over bank areas. After digitizing the flow paths, the *Assign Line Type Attributes* tool was used to assign the line type: left, channel, and right to the respective flow path lines.

The next component created using HEC-GeoRAS was the cross-section feature class, which was the key input to HEC-RAS. For the HEC-RAS modeling of LWCs, at least four cross sections, two each in upstream and downstream regions, are required. For the study, at least six cross sections were surveyed for each LWC site. The cross sections were drawn along the surveyed cross sections, which was layered on top of the TIN and aerial imagery. The cross-sections were digitized from left to right, moving from upstream to downstream with respect to the downstream direction.

Aerial imagery from ArcGIS was used as a guidance so that the cross sections do not cross the roadway. It was also ensured that no two cross sections intersected each other. The cross sections were drawn 20 - 30 times the width of the stream so that all the variability in the entire area is captured in the modeling. In some cases, it was impractical to extend boundary cross-sections to a greater extent as it would intersect with a present road or a segment of the same stream, which would cause a divided flow and HEC-RAS errors. The cross -section cutlines are 2D lines and have no elevation information associated with them. The 2D cutlines were converted into 3D by using *XS Cut Line Attributes* tool, which creates XSCutLines3D and contains the elevation information as well.

Although assigning Manning's n and creating the LWC structure can be done within HEC-GeoRAS, these things were done in HEC-RAS in this study as it is easier it HEC-RAS. Once the digitization of the layers mentioned above was completed, the *Layer Setup* option was selected from RAS Geometry menu to verify that all the required layers were ready for exporting. The stream centerline attributes, cross-section cutline attributes, bank lines, and flow path centerlines were selected and exported for HEC-RAS analysis using the *Export RAS Data* option.

4.10.2 HEC-RAS Processing

In the HEC-RAS processing, the geometry files created using ArcGIS were imported and the location of the LWC was identified and added. Required cross sections were interpolated, and the required input of Manning's n values, ineffective flow areas, contraction and expansion coefficients, and loss coefficients were provided. Then, steady flow analysis was performed using the design flood in the subcritical regime.

For the study, different project files were created corresponding to the each LWC. Separate project files were created for the LWC-free (baseline) scenario and alternative design as well. First, the geometry data exported from HEC-GeoRAS was imported into HEC-RAS. The river and reach data were verified along with the cross section data and importing was completed. The modeling part in HEC-RAS was done is US Customary units. The data was populated into the HEC-RAS geometry editor, which was then saved. Some of the cross sections had more than 500 elevation points (HEC-RAS limits the elevation points to 500), and in such cases, the *Cross Section Points Filter* tool was used from Geometric data editor. Wherever necessary, *graphical cross-section editor* was used to adjust the bank stations.

Manning's n values for the river channel, left over bank and right over bank were assigned. The variation in the Manning's n values was determined using the Land Cover information, aerial imagery and photographs taken during the survey. The channel Manning's n values were selected based on the stream characteristics, and the assigned values ranged from 0.03 to 0.04. The surrounding area land cover information derived from the 2011 NLCD data was used to determine the appropriate Manning's n value, which ranged from the values 0.035 to 0.1.

Then, the LWC was added at the identified location. *Edit/ or Create bridges and culverts* tool was used to add vented ford, and *Edit/ or Create Inline structures* tool was used to add unvented ford. Once the vented ford was inserted, *Deck/Roadway data editor* was opened and information such as deck width, a distance of deck to upstream cross section, weir coefficient, high chords for upstream and downstream stations, etc. were entered. In the weir crest shape, broad crested was selected, and weir coefficient of 2.54 was assigned. Ineffective flow areas were placed at cross sections upstream and downstream of crossings, assuming a contraction ratio of 1:1 and an expansion ratio of 1.5:1. Contraction and expansion coefficients were increased to 0.3 and 0.5, respectively, at cross sections adjacent to structures.

Similarly, in the case of unvented fords, Inline structure information such as deck width, a distance of deck to upstream cross section, weir coefficient, station and elevation coordinates for the top of the weir, etc. were entered. Franklin LWC was modeled as broad crested weir whereas Ogle LWC was modeled as Ogee weir. Unvented fords do not have ineffective flow areas, and the default contraction and expansion coefficients of 0.1 and 0.3 were used.

After the geometry data was completed, steady flow data for different flow profiles were added. The flow data were obtained from different sources which are discussed in section 4.7. Then, normal depth was selected as steady flow boundary conditions and the downstream slope for normal depth computation was entered. Next, subcritical flow regime was selected to perform a steady flow analysis, and the water surface profiles were computed.

Cross section outputs were analyzed for any errors and warnings, and necessary modification in the model input was made until the results were error free. All of the HEC-RAS models were reviewed by USACE-CERL engineers to verify roughness values, bank stations, ineffective flow areas, hydraulic structures, boundary conditions, and hydrologic model output.

The final results of the calculation were exported using the *Export GIS Data* option, and the water surface profiles to export for further processing were selected.

4.10.3 HEC-RAS Postprocessing

The final phase was the HEC-RAS post processing, in which the results from HEC-RAS simulation were imported into ArcGIS and the results of flood simulation were displayed using HEC-GeoRAS. The aerial extent of the flood as per the limiting depth was calculated, and the resultant area was reported.

HEC-GeoRAS was once again employed in the postprocessing. First, *Import RAS SDF file* option was selected, and the SDF file was converted into an XML file. Then, *RAS Mapping Layer Setup* option was selected to create a new analysis, and input rasterization cell size. Next, *Import RAS Data* option was selected from RAS Mapping, which created a bounding polygon by connecting the endpoints of XS cut lines.

For the water surface generation, *RAS Mapping/ Inundation Mapping/ Water Surface Generation* option was selected for the profile associated with the 25-year flow. Next, *RAS Mapping/ Inundation Mapping/ Floodplain Delineation using Rasters* was selected which resulted in the flood inundation polygon. Then, the depth raster was reclassified to find the area with flooding depth greater than 6 inches, and the resultant areas were noted down.

The following steps were carried out to obtain the 25-year flood inundation map:

1. The flood depth grid file was imported from HEC-RAS into ArcGIS
2. Binary raster calculation was performed to identify the area with flood depth of 6 inches or more
3. Converted the resultant binary calculation layer into a shapefile
4. Calculated the sum of the flooded area in the shapefile's attribute table
5. Reported resultant areas in the results section of the report

The flood inundation maps under different scenarios for all the LWCs are included in the results section.

4.11 Flood Extent Analysis

For the flood extent analysis of the LWC sites, a steady-state run of the HEC-RAS model was performed, and flood inundation analysis was done. The HEC-RAS models were run for the following flow rates:

1. 1% exceedance flow
2. 1-year return flow
3. 25-year return flow

1% exceedance flow and 1-year return flows were run to find out if the present pipes were adequate to convey the flow. Based on the site conditions and nature of the stream, these flows are used to select and design the crossing. In this analysis, 1% exceedance flow was taken as the main parameter to see if the present LWC is adequate or not. 1% flow is expected to pass through the pipes in case of vented LWCs and the overtopping depth in case of unvented LWCs is expected to be below 6 inches in some cases, 1-year return flow was used to make additional observations.

In this analysis, 25-year flow was used for the flood extent analysis. The LWC components apart from the pipes, such as crossing surface, approach roads, riprap, etc. should be designed for these high flows so that they are not significantly damaged even when the flows overtop the crossing. This flow to use for analysis depends upon the desired life of the crossing.

4.12 Sediment Transport Analysis

Sediment transport modeling in HEC-RAS v 4.1 assumes quasi-unsteady flow, in which the flow is constant for a part of the flow series, making it easier to compute sediment transport (Brunner 2010b). The major data requirements for the sediment modeling include sediment data (bed gradation) and quasi-unsteady flow data (flow series data and water temperature). Because there were no flow-gaging stations in any of the study areas, in addition to the lack of information about the bed gradation, this approach could not be utilized in the sediment transport modeling.

Thus, a simplified approach was adopted for the sediment transport analysis. Two critical cross sections in the immediate vicinity of the crossing were taken into consideration, and the change in bed shear was computed. Values for bed shear stress (lb/ ft^2) were obtained from the HEC-RAS model runs for the different scenarios. Channel bed shear represents the sediment transport capacity of the stream. Change in bed shear between the present condition and LWC free scenario was computed to see how the installation of the LWC has affected the sediment transport capacity in the stream.

4.13 Aquatic Organism Passage

There is no information about the presence of aquatic species in most of the LWCs modeled in the study. In the Ogle LWC located across Pine Creek, fish population such as bass, sunfish, carp, etc. are present, and no threatened or endangered species are reported in the stream. The LWC is an unvented one, which provided very minimal obstruction to the aquatic organism passage. Due to the lack of flow-gaging stations in any of the study areas, flow hydrographs could not be obtained for further analysis of AOP in FishXing. Average stream velocity for the two critical cross sections (immediate upstream and downstream) of the LWC are reported in the results section, and the change in the velocity due to the LWC construction compared to the LWC free scenario is analyzed. This helps in understanding how the LWCs are impacting the movement of aquatic species. Apart from the stream velocity, minimum depth of water in the stream and the LWC also affects the AOP.

CHAPTER 5: RESULTS AND DISCUSSION

5.1 Logan LWC

5.1.1 Results from HEC-RAS Analysis

The existing vented LWC in Logan County was modeled in HEC-RAS using the design flows of 1% exceedance (E1), 1-year flood (P1) and 25-year flood (P25). Similarly, the model was run for the same flows for the natural conditions (LWC free). The results of the analyses are presented as water surface elevation maps in Figure 5.1 and 5.2. From the analysis, the design of the existing LWC is found to be adequate to pass the design flows.

For the existing LWC, the 1% exceedance flow of 28.71 ft^3/s passes through the structure. This flow will be exceeded four days in a year, during which the LWC might be impassable. The 1-year flow of 186.01 ft^3/s passes over the structure with an overtopping depth of 1 ft, which has a probability of occurring once a year, and during this time the LWC will be closed for public use.

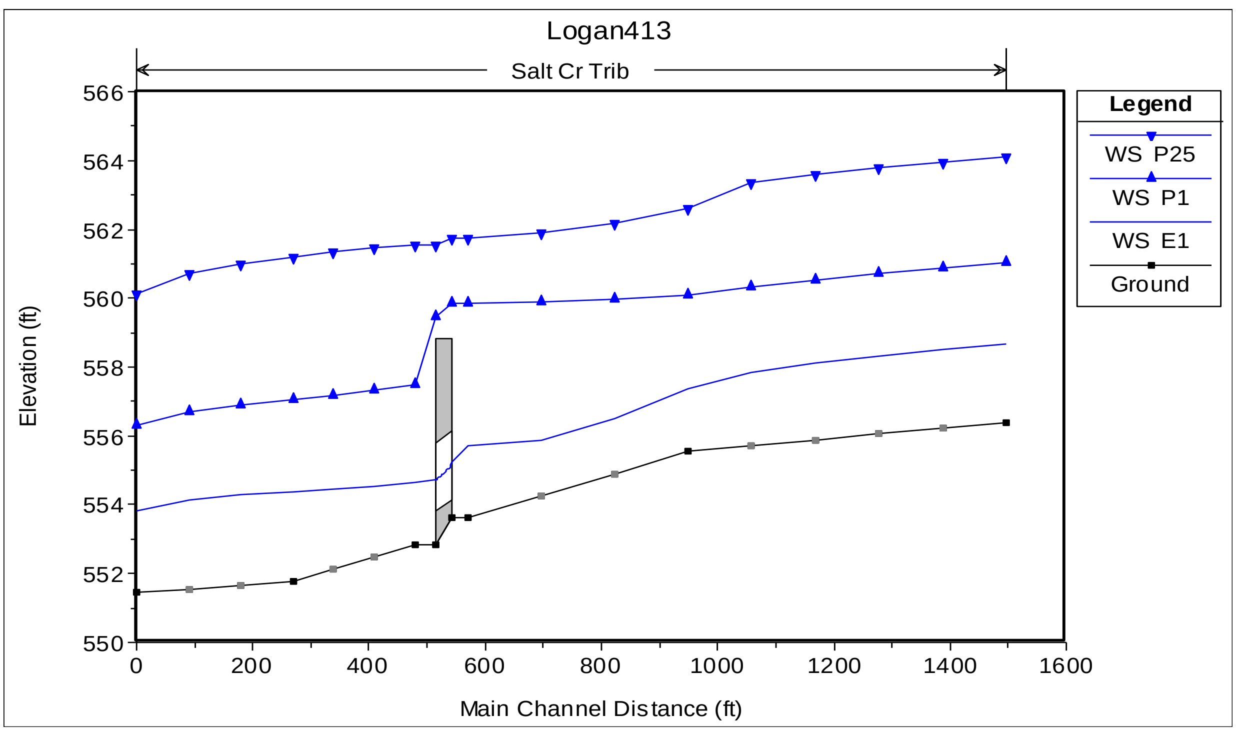

Figure 5.1: Water surface elevation for design flows in Logan LWC.

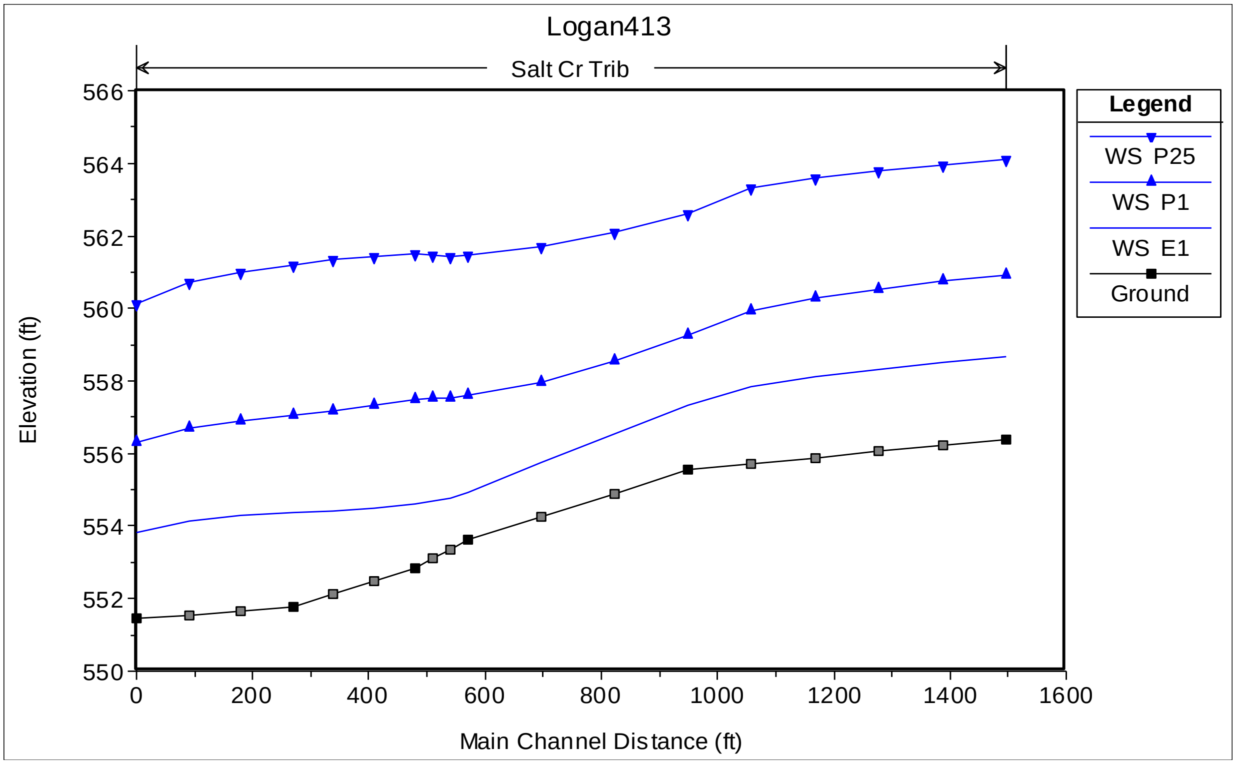

Figure 5.2: Water surface elevation for design flows in LWC free scenario.

The HEC-RAS analysis results were used to compute the flood depth in the LWC area. The results of the flood inundation study for the LWC site revealed that for the 25-year flow rate of 970 ft^3/s, the floodplain extent was minimally affected by the presence of the LWC (Table 5.1). In fact, it was found that there is a decrease in the inundated area in the present condition with the LWC compared to the LWC free scenario. With the present LWC, the area flooded with a depth of 6 inches or greater totaled 4.64 acres whereas it is 4.79 acres in LWC free condition. The decrease in inundated area ($d > 6$ in) is due to the backing up of water more in depth caused by the obstruction to the flow, which results in less area for the same volume of water.

The floodplain map of the inundated area due to the 25-year flood in the present condition is as shown in Figure 5.3. A portion of the approach roadway and the surrounding agricultural area is affected by this flood, which was found to be acceptable.

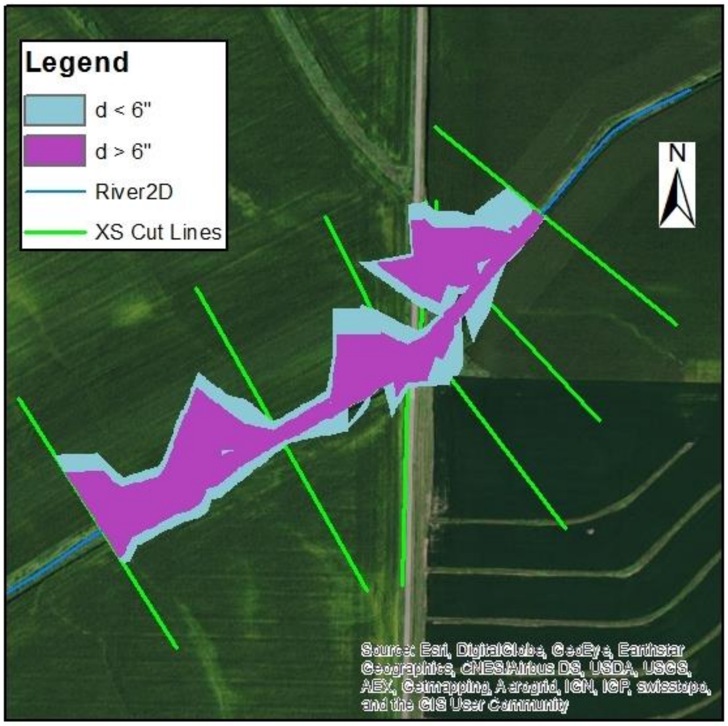

Figure 5.3: Flood inundation map for 25-year flood in Logan LWC.

Table 5.1: Results of the HEC-RAS 25-Year Flood Inundation for Logan LWC

LWC	Scenario	Inundated area (acres)		Percent change
		Present Scenario	LWC free scenario	
Logan	Total area	7.37	7.37	0.00
	Area with d > 6 in.	4.64	4.79	-3.23

For sediment transport capacity in the stream, the streambed shear stress output from HEC-RAS analysis was utilized. The results in Table 5.2 are average bed shear stress for the cross sections upstream and downstream of the LWC at the 1% exceedance flow of 28.71 ft^3/s.

Table 5.2: Results for Bed Shear Stress for Critical Cross Sections in Logan LWC

Scenario	Shear Stress (lb/ ft^2)	
	U/S Section	D/S Section
LWC	0.08	0.14
LWC Free	0.34	0.15

There is minimal change in the shear stress in the downstream cross section, before and after the LWC. However, in the upstream cross section, the shear stress decreases from 0.34 lb/ ft^2 in the LWC free scenario to 0.08 lb/ ft^2 in the present scenario. This means that the LWC is restricting the sediment transport in the downstream direction, which may lead to sediment deposition.

For the AOP in the stream, the average velocity output from HEC-RAS analysis was analyzed. The results in Table 5.3 are average velocities for the cross sections upstream and downstream of the LWC at the 1% exceedance flow of 28.71 ft^3/s.

Table 5.3: Results for average velocity for critical cross sections in Logan LWC

	Velocity (ft/s)	
Scenario	**U/S Section**	**D/S Section**
LWC	1.84	2.43
LWC Free	3.58	2.43

There is no change in the average velocity in the downstream cross section, before and after the LWC installation. However, in the upstream cross section, the velocity decreases from 3.58 ft/s in the LWC free scenario to 1.84 ft/s in the present scenario. The LWC is restricting the movement of water, backing up the water and reducing the velocity. The abrupt change in the velocity from upstream to downstream affects the movement of fishes and aquatic species.

5.1.2 Summary and Discussion

The Logan LWC lies in a very small stream fed mostly by drainage tiles. It is an example of a cost-effective LWC for locations with agricultural drainage and low road use. It is a good design option as there is an alternative travel route 2 miles away when the crossing is impassable. Maintenance is required across the structure after high flows.

5.2 Edgar#1 LWC

5.2.1 Results from HEC-RAS Analysis

The vented LWC in Edgar County (Edgar#1) was modeled in HEC-RAS using the design flows of 1% exceedance (E1), 1-year flood (P1) and the 25-year flood (P25). The model was also run for the same flows for the natural condition (LWC free). Results of the analyses are presented as water surface elevation maps in Figure 5.4 and 5.5. From the analysis, the design of the existing LWC is found to be adequate to pass the design flows.

For the existing LWC, the 1% exceedance flow of 72.56 ft^3/s passes through the structure. This flow is expected to be exceeded four days in a year, during which the LWC might be impassable. The 1-year flow of 231.18 ft^3/s passes over the structure with an overtopping depth of 1 ft, which has a probability of occurring once a year, and during this time the LWC will be closed for public use.

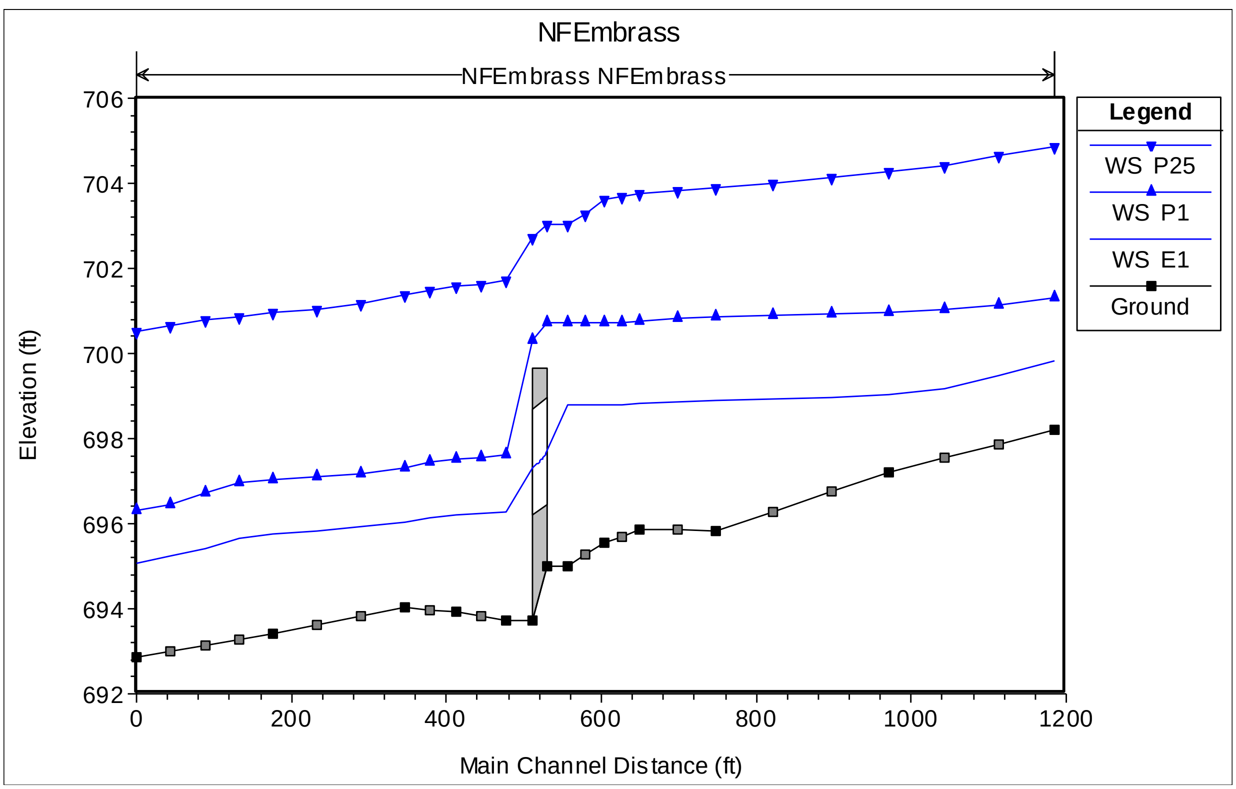

Figure 5.4: Water surface elevation for design flows in Edgar#1 LWC.

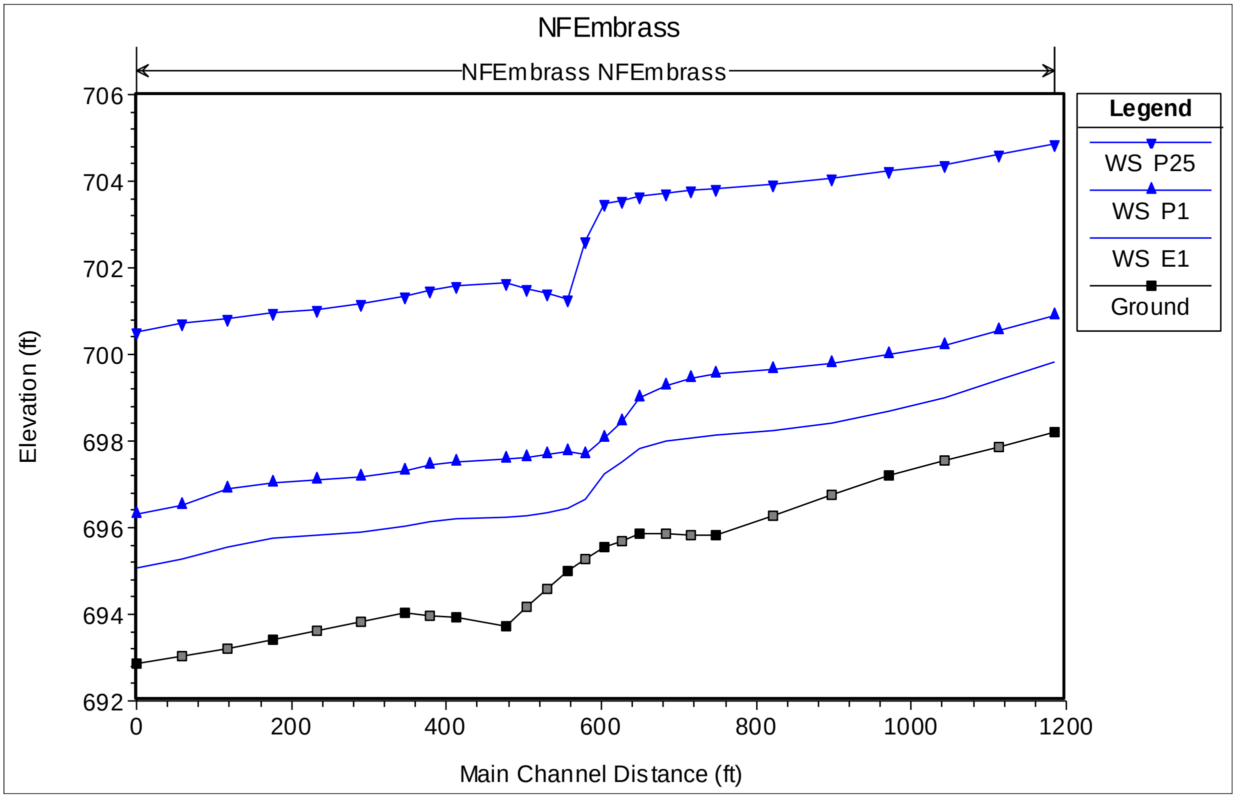

Figure 5.5: Water surface elevation for design flows in LWC free scenario.

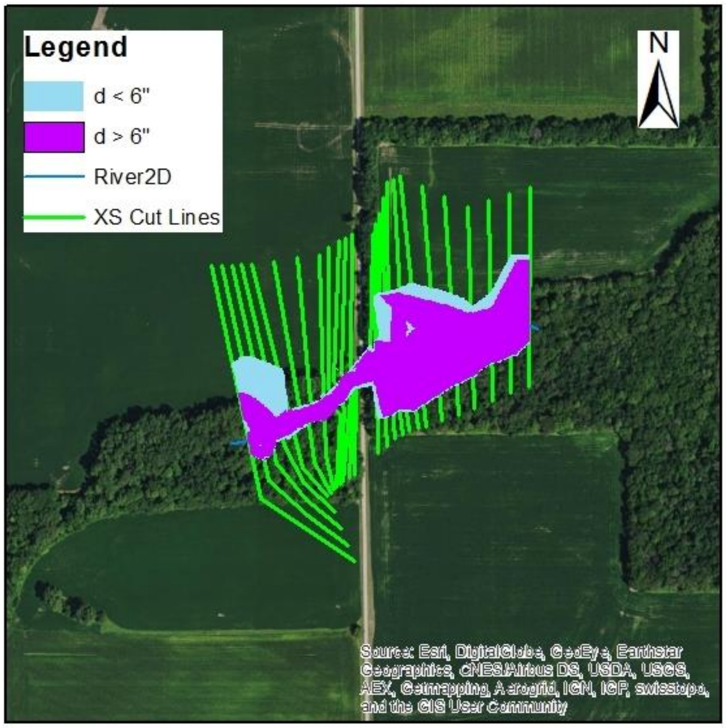

Figure 5.6: Flood inundation map for 25-year flood in Edgar#1 LWC.

The HEC-RAS analysis results were used to compute the flood depth in the LWC area. The results of the flood inundation study for the LWC site revealed that for the 25-year flow rate of 1890 ft^3/s, the floodplain extent was minimally affected by the presence of the LWC (Table 5.4). It was found that there is an increase in the inundated area by 5% in the present condition with the LWC compared to the LWC free scenario. With the present LWC, the area flooded with a depth of 6” or greater totaled 5.13 acres whereas it is 4.85 acres in the LWC free condition.

The floodplain map of the inundated area due to the 25-year flood in present condition is as shown in Figure 5.6. The area inundated by this flood includes the surrounding forested area and a small portion of farmland, which was found to be acceptable.

Table 5.4: Results of the HEC-RAS 25-year flood inundation for Edgar#1 LWC

LWC	**Scenario**	**Inundated area (acres)**		**Percent change**
		Present Scenario	**LWC free scenario**	
Edgar#1	Total area	6.76	6.47	4.48
	Area with d > 6”	5.13	4.85	5.77

For sediment transport capacity in the stream, the streambed shear stress output from HEC-RAS analysis was taken into consideration. The results in Table 5.5 are average bed shear stress for the cross sections upstream and downstream of the LWC at the 1% exceedance flow of 72.56 ft^3/s.

Table 5.5: Results for shear stress for critical cross sections in Edgar#1 LWC

	Shear Stress (lb/ ft^2)	
Scenario	**U/S Section**	**D/S Section**
LWC	0.02	0.11
LWC Free	0.42	0.11

There is minimal change in the shear stress in the downstream cross section, before and after the LWC. However, in the upstream cross section, the shear stress decreases from 0.42 lb/ ft^2 in the LWC free scenario to 0.02 lb/ ft^2 in the present scenario. This means that the LWC is restricting the sediment transport in the downstream direction, which may lead to sediment deposition.

For the AOP in the stream, the average velocity output from HEC-RAS analysis was analyzed. The results in Table 5.6 are average velocities for the cross sections upstream and downstream of the LWC at the 1% exceedance flow of 72.56 ft^3/s.

Table 5.6: Results for average velocity for critical cross sections in Edgar#1 LWC

	Velocity (ft/s)	
Scenario	**U/S Section**	**D/S Section**
LWC	0.82	1.71
LWC Free	3.08	1.71

There is no change in the average velocity in the downstream cross section, before and after the LWC installation. However, in the upstream cross section, the velocity decreases from 3.08 ft/s in the LWC free scenario to 0.82 ft/s in the present scenario. The LWC is restricting the movement of water, backing up the water and reducing the velocity. The LWC is controlling the abrupt change in the velocity from upstream to downstream and is more conductive to the movement of fishes and aquatic species compared to LWC free condition.

5.2.2 Summary and Discussion

This vented LWC is a good choice for the site located in a small drainage watershed. The crossing is functioning well, and there is very little effect on the environment.

Periodic maintenance is required in the LWC, as the pipes tend to get clogged by the logs and branches of trees that tend to get trapped at the crossing. The vented ford poses less restriction to the movement of aquatic species.

5.3 Edgar#3 LWC Site

5.3.1 Results from HEC-RAS Analysis

The existing vented LWC in Edgar County (Edgar#3) was modeled in HEC-RAS using the design flows of 1% exceedance (E1), 1-year flood (P1) and 25-year flood (P25). Similarly, the model was run for the same flows for the LWC free scenario and alternate design. The results of these analyses are presented as water surface elevation maps in Figure 5.7, 5.8 and 5.9. From the analysis, the design of the existing LWC is found to be inadequate to pass the design flows. Thus, another alternative design has been presented.

For the existing LWC, the 1% exceedance flow of 192.04 ft^3/s passes over the structure with an overtopping depth of 1.5 ft and 1-year flow of 477.16 ft^3/s has an overtopping depth of 2.5 ft Since the 1% exceedance flow is expected to flow under the road surface through pipes, the design is found to be insufficient.

The alternative design for this LWC is a single cell rectangular concrete box culvert with a span of 8 ft and rise of 4 ft with a cover of 8 inches. The culvert was centered at the middle of the roadway, and the minimum high chord of the road was raised to an elevation of 606.9 ft from the current 602.1 ft in the design. 1% exceedance flow in this case passes through the box culvert whereas the 1-year flood has an overtopping depth of 11 inches. With this design, the LWC is expected to be impassable for less than four days in a year, during which the LWC will be closed for public use.

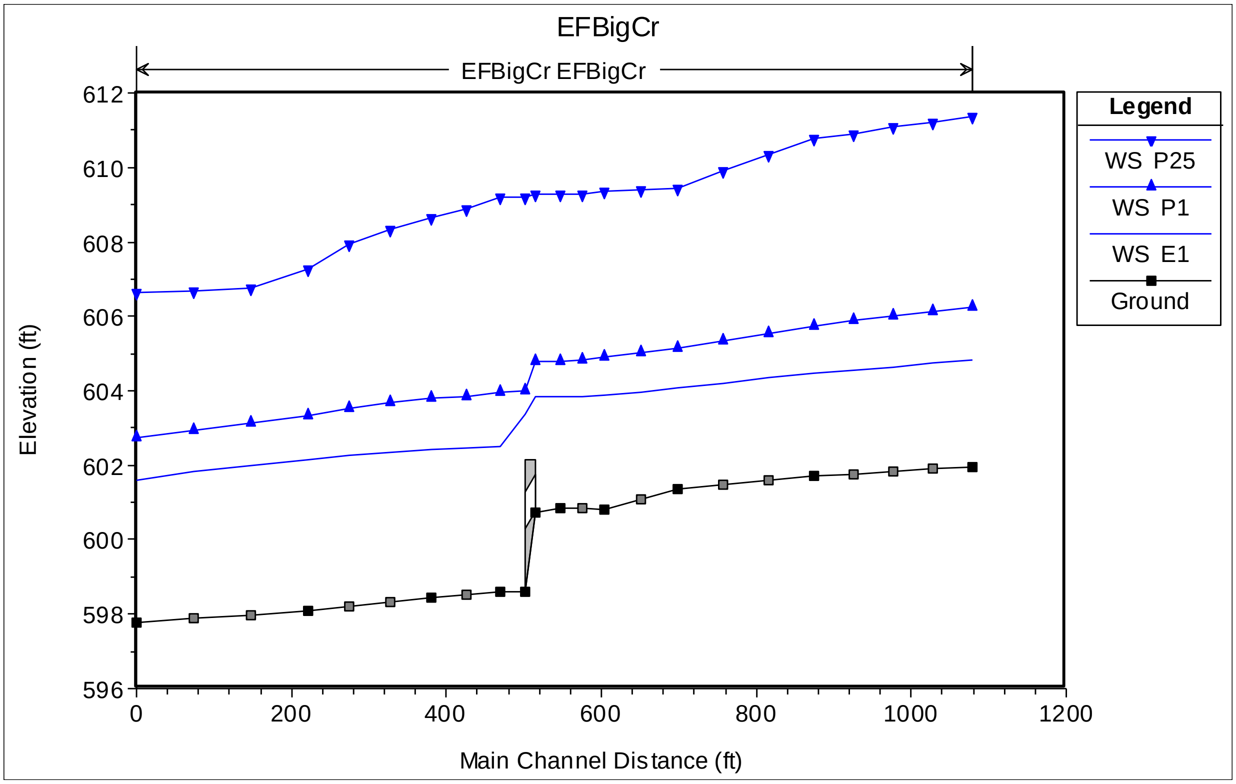

Figure 5.7: Water surface elevation for design flows in Edgar#3 LWC.

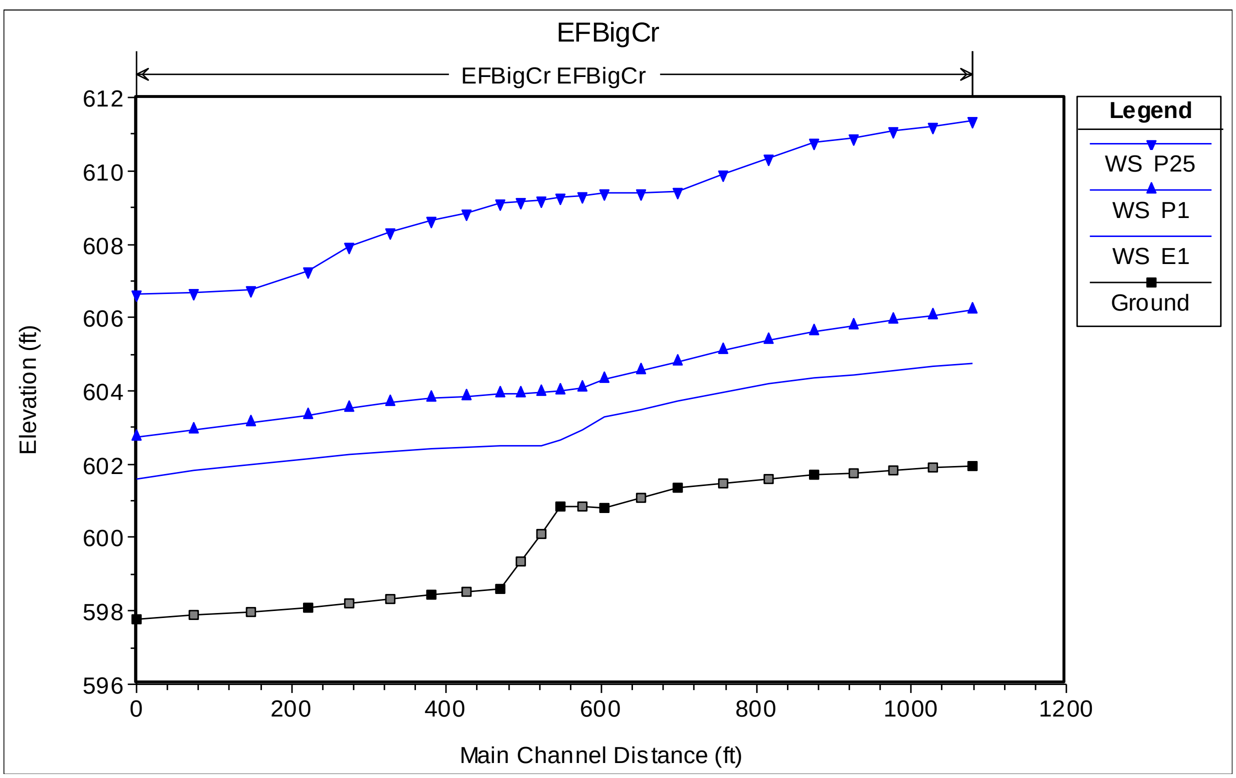

Figure 5.8: Water surface elevation for design flows in LWC free scenario.

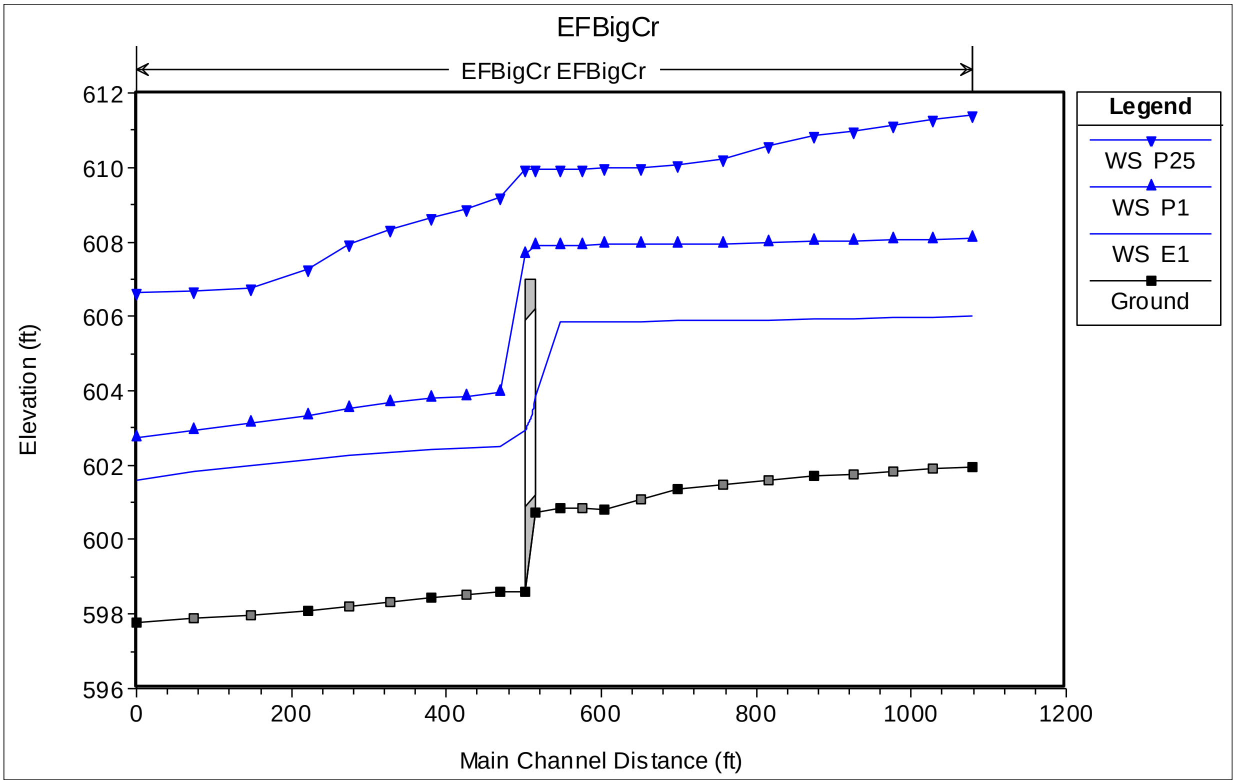

Figure 5.9: Water surface elevation for design flows in Edgar#3 LWC alternate design.

The HEC-RAS analysis results were used to compute the flood depth in the LWC area. The results of the flood inundation study for the LWC site revealed that for the 25-year flow rate of 3170 ft^3/s, the floodplain extent was minimally affected by the presence of the current LWC (Table 5.7). It was found that there is a decrease in the inundated area in the present condition with the LWC compared to the LWC free scenario. With the present LWC, the area flooded with a depth of 6” or greater totaled 2.92 acres whereas it is 3.12 acres with the alternative design.

The floodplain map of the inundated area due to the 25-year flood in present condition and alternate design is as shown in Figures 5.10 and 5.11. The area inundated by this flood includes the surrounding forested area, which was found to be acceptable.

Table 5.7: Results of the HEC-RAS 25-year flood inundation for Edgar#3 LWC

LWC	Scenario	Inundated area (acres)		Percent change
		Present Scenario	LWC free scenario	
Edgar#3	Total area	3.72	3.72	0.00
	Area with d > 6”	2.92	2.92	0.00
Edgar#3 Alternative	Total area	4.15	3.72	11.55
	Area with d > 6”	3.12	2.92	7.19

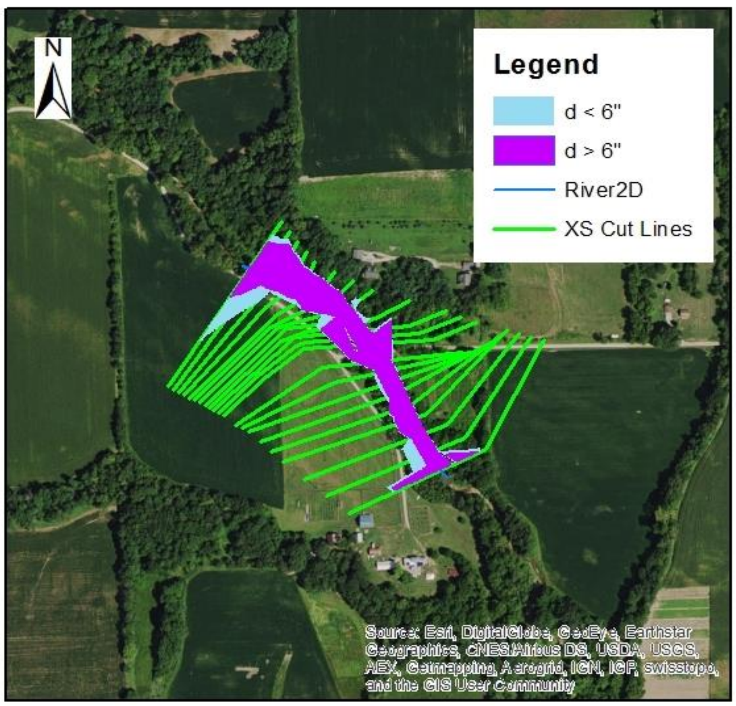

Figure 5.10: Flood inundation map for 25-year flood in Edgar#3 LWC.

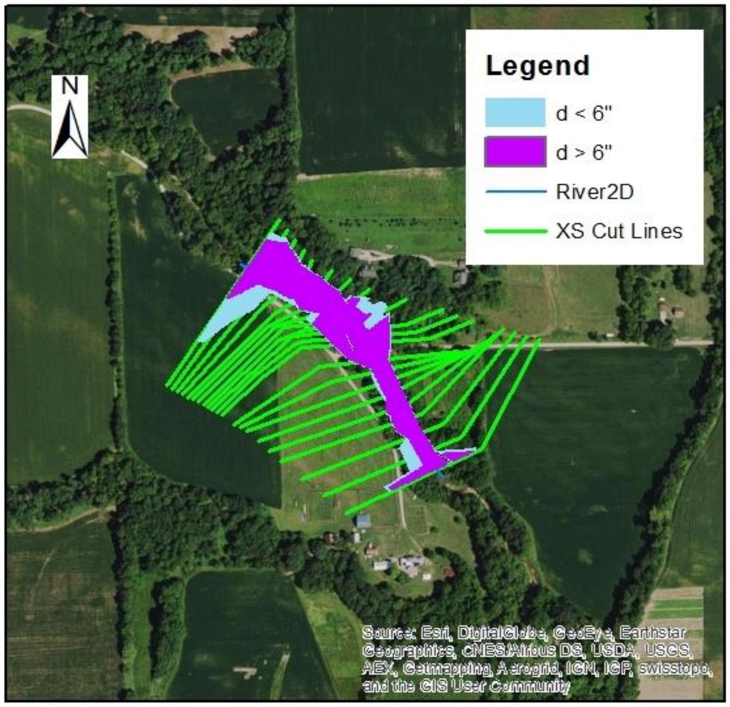

Figure 5.11: Flood inundation map for 25-year flood in Edgar#3 LWC alternate design.

For sediment transport capacity in the stream, the streambed shear stress output from the HEC-RAS analysis was utilized. The results in Table 5.8 are average bed shear stress for the cross sections upstream and downstream of the LWC at the 1% exceedance flow of 477.16 ft^3/s.

Table 5.8: Results for shear stress for critical cross sections in Edgar#3 LWC

	Shear Stress (lb/ ft^2)	
Scenario	**U/S Section**	**D/S Section**
LWC	0.11	0.08
LWC Free	0.67	0.08
Alt Design	0.02	0.08

There is minimal change in the shear stress in the downstream cross section, before and after the LWC and in the alternate design. However, in the upstream cross section, the shear stress decreases from 0.67 lb/ ft^2 in the LWC free scenario to 0.11 lb/ ft^2 in the present scenario and 0.02 lb/ ft^2 in the alternate design. This means that the LWC is restricting the sediment transport in the downstream direction, which leads to sediment deposition in the upstream region and scouring in the sediment starved downstream region.

For the AOP in the stream, the average velocity output from HEC-RAS analysis was analyzed. The results in Table 5.9 are average velocities for the cross sections upstream and downstream of the LWC at the 1% exceedance flow of 477.16 ft^3/s.

Table 5.9: Results for average velocity for critical cross sections in Edgar#3 LWC

	Velocity (ft/s)	
Scenario	**U/S Section**	**D/S Section**
LWC	1.74	1.54
LWC Free	3.84	1.54
Alt Design	0.85	1.54

There is no change in the average velocity in the downstream cross section, before and after the LWC installation. However, in the upstream cross section, the velocity decreases from 3.58 ft/s in the LWC free scenario to 1.74 ft/s in the present scenario and 0.85 ft/s in the alternate design. The LWC is controlling the abrupt change in the velocity from upstream to downstream. Alternate design more conductive to the movement of fishes and aquatic species compared to LWC free condition.

5.3.2 Summary and Discussion

This LWC in Edgar County is an example of a poorly designed LWC. Although the crossing is located on a low ADT road, it is not big enough to pass the 1% exceedance flow through it. The crossing also acts as a sediment trap. There is scouring underneath the structure which needs to be addressed.

Alternate design analysis has been done using the concrete box culverts. Possible improvement in the crossing might include replacement by a suitable alternative LWC, as the drainage area of the watershed is relatively large giving a significant amount of flow. This will also facilitate easier AOP.

5.4 Franklin LWC

5.4.1 Results from HEC-RAS Analysis

The unvented LWC in Franklin County was modeled in HEC-RAS using the design flows of 1% exceedance (E1), 1-year flood (P1) and 25-year flood (P25). Similarly, the model was run for the same flows for the LWC free scenario. The results of this analysis are presented as water surface elevation maps in Figure 5.12 and 5.13. From the analysis, the design of the existing LWC is found to be adequate to pass the design flows.

For the existing LWC, the 1% exceedance flow of 25 ft^3/s passes over the structure with an overtopping depth of 6 inches. This flow is expected to be exceeded four days in a year, during which the LWC might be impassable. The 1-year flow of 174.27 ft^3/s has an overtopping depth of 2 ft, which has a probability of occurring once a year, and during this time the LWC will be closed for public use.

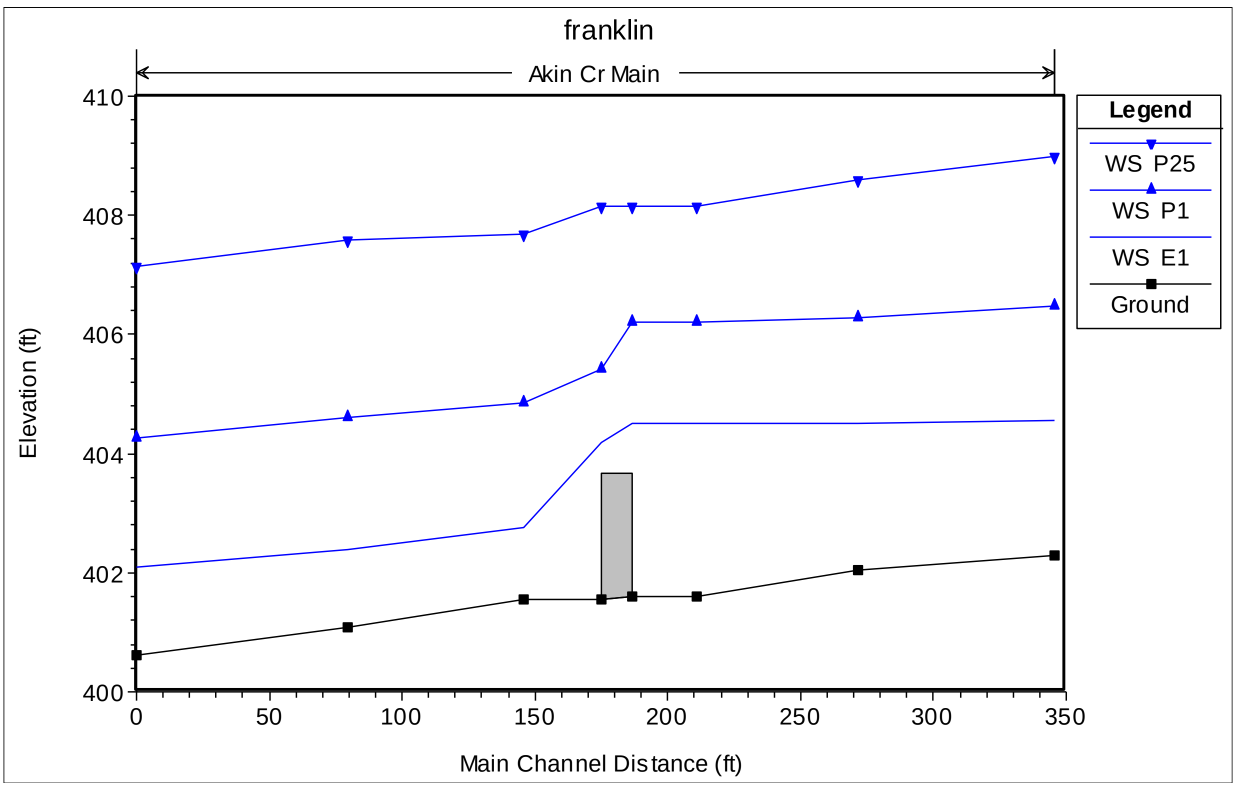

Figure 5.12: Water surface elevation for design flows in Franklin LWC.

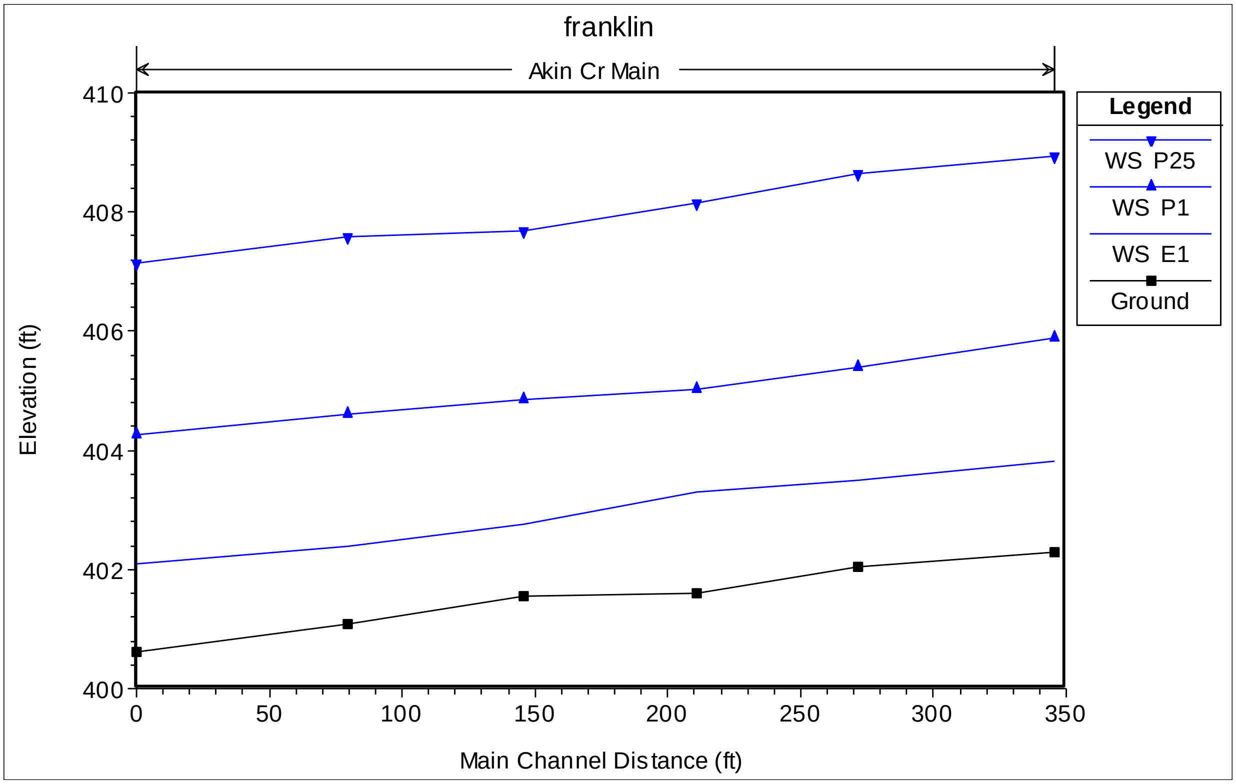

Figure 5.13: Water surface elevation for design flows in LWC free scenario.

The HEC-RAS analysis results were used to compute the flood depth in the LWC area. The results of the flood inundation study for the LWC site revealed that for the 25-year flow rate of 854 ft^3/s, the floodplain extent was minimally affected by the presence of the LWC (Table 5.10). It was found that there is a slight increase in the inundated area in the present condition with the LWC compared to the LWC free scenario. With the present LWC, the area flooded with a depth of 6" or greater totaled 1.30 acres, same as the LWC free condition.

The floodplain map of the inundated area due to the 25-year flood in present condition is as shown in Figure 5.14. A portion of the approach roadway and the surrounding agricultural area is affected by this flood, which was found to be acceptable.

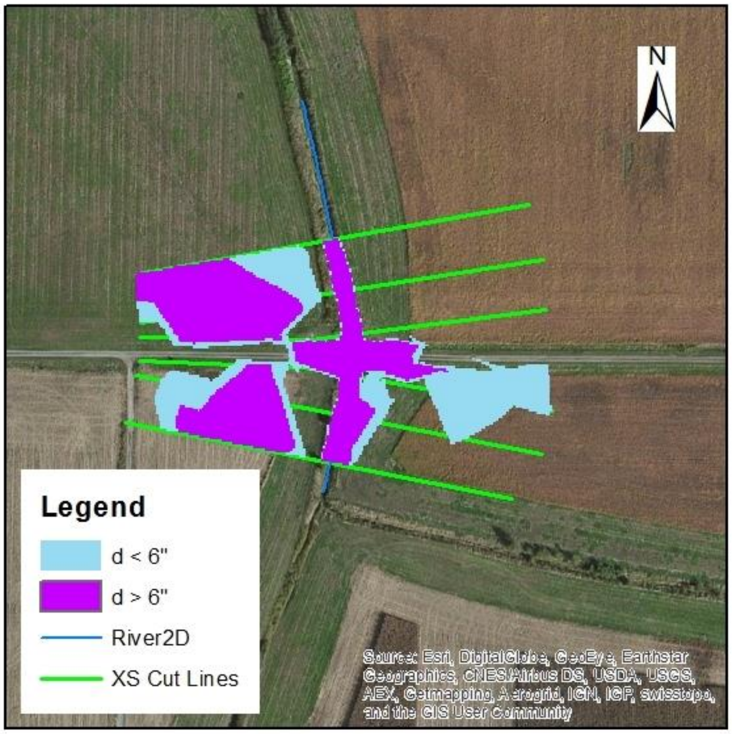

Figure 5.14: Flood inundation map for 25-year flood in Franklin LWC.

Table 5.10: Results of the HEC-RAS 25-year flood inundation for Franklin LWC

LWC	Scenario	Inundated area (acres)		Percent change
		Present Scenario	LWC free scenario	
Franklin	Total area	2.20	2.24	-1.78
	Area with d > 6”	1.30	1.30	0.00

For sediment transport capacity in the stream, the streambed shear stress output from HEC-RAS analysis was utilized. The results in Table 5.11 are average bed shear stress for the cross sections upstream and downstream of the LWC at the 1% exceedance flow of 25 ft^3/s.

Table 5.11: Results for shear stress for critical cross sections in Franklin LWC

	Shear Stress (lb/ ft^2)	
Scenario	U/S Section	D/S Section
LWC	0.03	0.61
LWC Free	0.17	0.61

There is no change in the shear stress in the downstream cross section, before and after the LWC. However, in the upstream cross section, the shear stress decreases from 0.17 lb/ ft^2 in the LWC free scenario to 0.03 lb/ ft^2 in the present scenario. The results indicate that the LWC is restricting the sediment transport in the downstream direction, which may lead to sediment deposition.

For the AOP in the stream, the average velocity output from HEC-RAS analysis was analyzed. The results in Table 5.12 are average velocities for the cross sections upstream and downstream of the LWC at the 1% exceedance flow of 25 ft^3/s.

Table 5.12: Results for average velocity for critical cross sections in Franklin LWC

	Velocity (ft/s)	
Scenario	**U/S Section**	**D/S Section**
LWC	0.95	3.71
LWC Free	2.23	3.71

There is no change in the average velocity in the downstream cross section, before and after the LWC installation. However, in the upstream cross section, the velocity decreases from 2.23 ft/s in the LWC free scenario to 0.95 ft/s in the present scenario. The LWC is controlling the abrupt change in the velocity from upstream to downstream.

5.4.2 Summary and Discussion

Franklin LWC is located on a road that serves only one residence. The crossing is located in an ephemeral stream with the very low flow most of the time. If the water gets high, there is another road nearby for a detour. This unvented ford is a good example where the local highway authority does not have enough money to construct a culvert or other alternatives.

Unvented fords are easy to maintain, and the surface of the crossing requires periodic maintenance after larger precipitation events.

5.5 Ogle LWC

5.5.1 Results from HEC-RAS Analysis

The unvented LWC in Ogle County was modeled in HEC-RAS using the design flows of 1% exceedance (E1), 1-year flood (P1) and 25-year flood (P25). Similarly, the model was run for the same flows for the LWC free scenario. The results of this analysis are presented as water surface elevation maps in Figure 5.15 and 5.16. From the analysis, the design of the existing LWC is found to be adequate to pass the design flows.

For the existing LWC, the 1% exceedance flow of 298.18 ft3/s passes over the structure with an overtopping depth of 1 ft. This flow is expected to be exceeded 4 days in a year, during which the LWC might be impassable. The 1-year flow of 1403.83 ft3/s has an overtopping depth of 2 ft, which has a probability of occurring once a year, and during this time the LWC will be closed for public use.

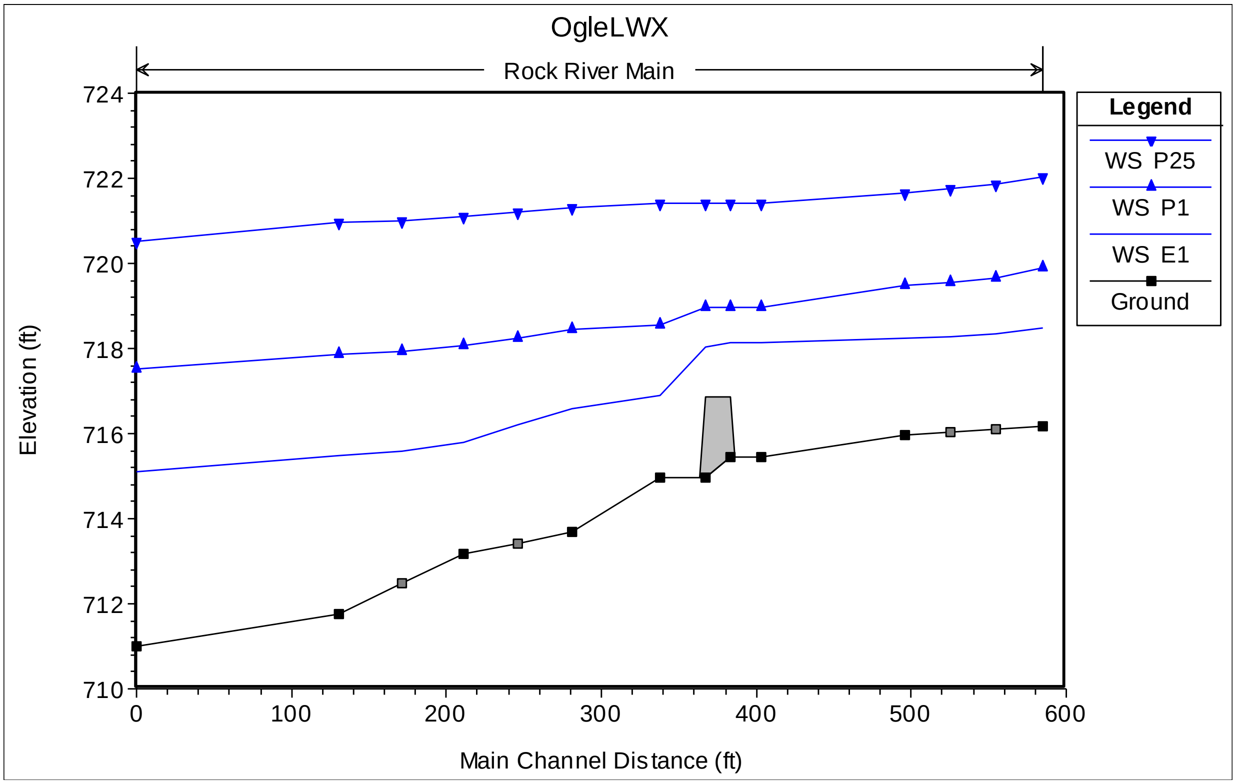

Figure 5.15: Water surface elevation for design flows in Ogle LWC.

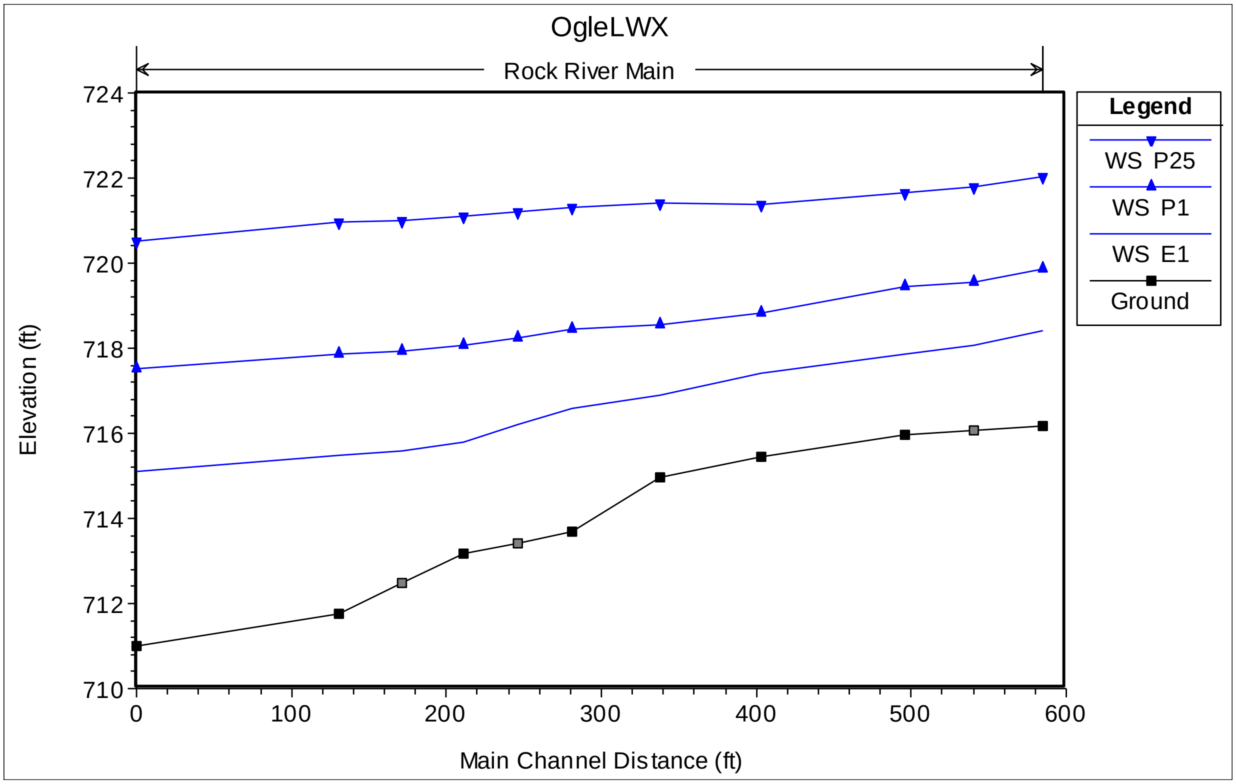

Figure 5.16: Water surface elevation for design flows in LWC free scenario.

The HEC-RAS analysis results were used to compute the flood depth in the LWC area. The results of the flood inundation study for the LWC site revealed that for the 25-year flow rate of 4700 ft^3/s, the floodplain extent was minimally affected by the presence of the LWC (Table 5.13). It was found that there is an increase in the inundated area with a depth of more than 6" in the present condition with the LWC compared to the LWC free scenario. With the present LWC, the area flooded with a depth of 6" or greater totaled 5.51 acres whereas it is 4.92 acres in the LWC free condition.

Table 5.13: Results of the HEC-RAS 25-year flood inundation for Ogle LWC

LWC	Scenario	Inundated area (acres)		Percent change
		Present Scenario	LWC free scenario	
Ogle	Total area	6.24	6.21	0.48
	Area with d > 6"	5.51	4.92	11.99

The floodplain map of the inundated area due to the 25-year flood in present condition is as shown in Figure 5.17. A significant portion of the approach roadway and the parking lot area is affected by this flood.

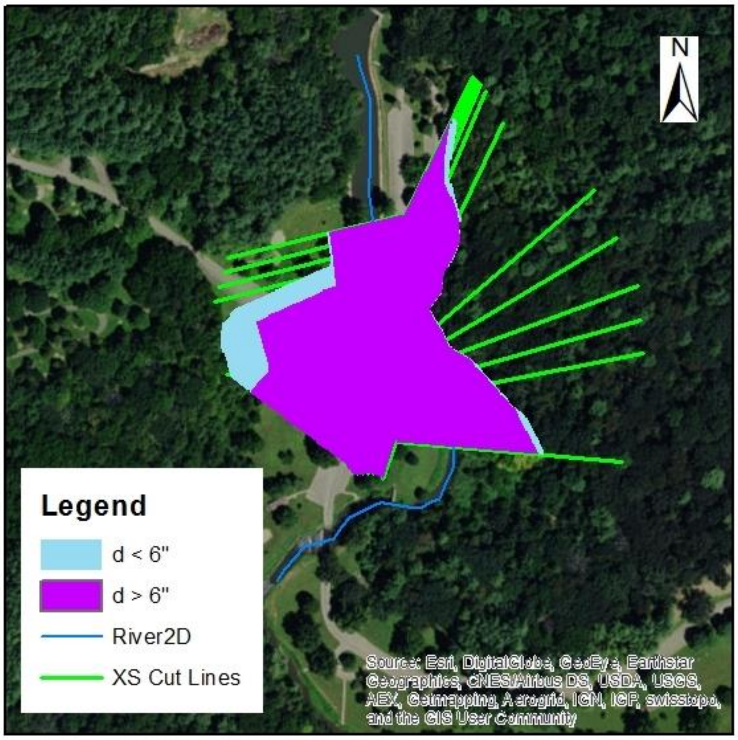

Figure 5.17: Flood inundation map for 25-year flood in Ogle LWC.

For sediment transport capacity in the stream, the streambed shear stress output from HEC-RAS analysis was utilized. The results in Table 5.14 are average bed shear stress in the main channel for the cross sections upstream and downstream of the LWC at the 1% exceedance flow of 298.18 ft^3/s.

Table 5.14: Results for shear stress for critical cross sections in Ogle LWC

	Shear Stress (lb/ ft^2)	
Scenario	**U/S Section**	**D/S Section**
LWC	0.11	0.66
LWC Free	0.36	0.66

There is no change in the shear stress in the downstream cross section, before and after the LWC. However, in the upstream cross section, the shear stress decreases from 0.36 lb/ ft^2 in the

LWC free scenario to 0.11 lb/ ft^2 in the present scenario. This means that the LWC is restricting the sediment transport in the downstream direction, which may lead to sediment deposition in the upstream region.

For the AOP in the stream, the average velocity output from HEC-RAS analysis was analyzed. The results in Table 5.15 are average velocities for the cross sections upstream and downstream of the LWC at the 1% exceedance flow of 298.18 ft^3/s.

Table 5.15: Results for average velocity for critical cross sections in Ogle LWC

Scenario	Velocity (ft/s)	
	U/S Section	**D/S Section**
LWC	1.44	3.77
LWC Free	2.89	3.77

There is no change in the average velocity in the downstream cross section, before and after the LWC installation. However, in the upstream cross section, the velocity decreases from 2.89 ft/s in the LWC free scenario to 1.44 ft/s in the present scenario. The LWC is restricting the movement of water, backing up the water and reducing the velocity. The velocity downstream is higher due to the ogee shaped weir and affects the movement of fishes and aquatic species across the LWC.

5.5.2 Summary and Discussion

The low water crossing performs well during periods of normal flow and is serving its intended purpose. Normal flow results in an approximate water depth of 6 inches in the center. It has required very little maintenance and has had a negligible impact on the aquatic environment.

Since the LWC is closed in the events of heavy precipitation during spring and over winter, the LWC is functioning adequately. This is one case where the LWC might be closed for more than 25% of the time in a year. The crossing is located in a park, and the roads are closed to the public when there is no staff in the park, which makes flash flooding and the safety of the public less of an issue.

CHAPTER 6: CONCLUSION

Low water crossings have been used in rural, low ADT routes in Illinois as an alternative to culverts and bridges for a long time. However, with the lack of design guidelines specific to LWCs, there is no standard practice or design among these LWCs in Illinois. Historically, LWCs have been selected, designed, and constructed based on the experience of highway department officials.

Within the past decade, studies have been performed, selection and design criteria have been established for LWCs by different agencies, and reports have been published (Barnard et al. 2013; Bates and Kirn 2009; Clarkin et al. 2006; Howard et al. 2011; Lohnes et al. 2001). This study includes findings from the previous studies incorporated with a LWC survey, case studies, and other information specific to Illinois.

From the case studies, it was found that most of the modeled LWCs were able to pass the design flow (4 LWCs out of 5), but are not conductive to the sediment transport and aquatic organism movement. Results from the flood inundation studies show that the change in the inundated area compared to the baseline scenario is within 5% in most of the cases. There is a significant decrease in the shear stress and velocity in the cross section upstream of the crossing, restricting the sediment transport. LWCs are acting as a sediment trap, which over the long period of time will modify the channel characteristics and affect the stream dynamics. LWCs provide a restriction to the flow of water and increase inundation under higher flows but allow smooth and safe movement of vehicles across the streams.

It is a challenge for an engineer to design a LWC in an economical way that has minimum effects on the aquatic organism passage in the stream. Various factors such as stream type, hydrology, channel conditions, road use, economics, and aquatic organism passage should be considered during the selection, design, and construction of LWCs. Installation of a LWC in a particular site involves a compromise between human needs and the environment. Efforts should be directed at posing minimum disturbance to the surrounding environment when constructing these crossings.

6.1 Recommendations for Future Studies

This study provides a clear methodology on how HEC-RAS can be used in hydrological modeling of LWCs. The difference in the modeling approach for unvented fords, vented fords and low water bridges are also discussed. However, the lack of all the required data hindered the sediment transport modeling in HEC-RAS. Model verification could not be done due to the lack of gaging stations nearby. With the increasing popularity of LWCs in rural low volume roads, the hydrological modeling and performance evaluation of LWCs, even before their construction, is more beneficial.

The following additional data can be collected and utilized in order to improve upon the HEC-RAS analysis:

- Stream flow data, which can be used to obtain the flow hydrographs
- Streambed gradation at each LWC cross-sections
- More detailed bathymetric survey data for the LWC area, if possible.
- Continuous monitoring of the stream velocity and depth of flow for AOP

With the addition of above mentioned data in the modeling, the reliability of the HEC-RAS results can be improved, and it helps further in understanding the impacts of LWCs on the environment.

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APPENDIX A: SURVEY QUESTIONNAIRE FOR LWCS IN ILLINOIS

Development of Low-Water Crossing Design Guidelines for Very Low ADT Routes in Illinois

Survey Questionnaire

Responses Due: February 20, 2015

This survey is to assist the Illinois Department of Transportation (IDOT) and local public agencies in determining a safe, cost-effective, and environmentally friendly design of Low-Water Crossings (LWCs) for very low average daily traffic (ADT) routes in Illinois. The focus of this work is to develop guidelines that can be used to determine appropriate locations and an optimal design of LWCs to meet traffic needs, while maintaining natural channel function.

As part of this project, we are surveying IDOT, county, and municipal engineers on the current status of LWCs in Illinois. The survey will provide critical information to effectively determine optimal design, current practices, and potential design issues impacting natural channel flow and safety.

For Questions, please contact either Niels Svendsen (niels.g.svendsen@usace.army.mil) 217-373-3448 or Heidi Howard (heidi.r.howard@usace.army.mil) 217-373-5865.

Definition: The Natural Resources Conservation Service (NRCS) defines a LWC as "a stabilized area or structure constructed across a stream to provide a travel way for people, livestock, equipment or vehicles." These LWCs may consist of an unvented ford, a vented ford, or bridges and culverts designed to be overtopped by high flows during flooding conditions. For purposes of this survey, as there are currently no design standards in Illinois, please provide locations which appear to meet the functional definition of a LWC.

LWC: Unvented ford

LWC: Vented ford

Respondent Information

- Name:
- Organization:
- Telephone:
- E-mail:

General LWC Questions

1. Please indicate the number of LWCs meeting the above definition within your jurisdiction:
2. Please indicate the number of LWCs proposed for development in 2015 and 2016:
3. Do you have your LWCs located within an available GIS layer (location, name, etc)?

Individual LWC Questions (See attached spreadsheet for multiple LWCs)

1. Please number the LWCs, and indicate the NBIS Structure Number if applicable. (We will then assign a tracking number for our database and for future reference):
2. Please indicate the LWC location: (Latitude and Longitude are preferred, or an approximate location (street address/junction))
3. Please indicate the stream or body of water the LWC is on:
4. Please clearly label and include any photographs of the LWC: (Insert here or attach)
5. Please indicate the LWC type: (*Examples; at grade structure (vented/unvented), above grade structure (vented/unvented), culvert, low water bridge (pier and pillar), etc:*
6. Please indicate the design specifications used for this LWC if applicable:
7. Please indicate the storm event the LWC was designed for, if applicable: (i.e. 15 year, 30 year, etc.)
8. Please indicate the function and intended use of this LWC:
9. Please indicate the Average Daily Traffic (ADT):
10. Please estimate the number of over-toppings per year:
11. Does this LWC have advance warning signs?
    a. If Yes, what type(s):
    b. Please insert any photographs of the warning signs:

12. Do you experience any maintenance issues with this LWC:

13. Have you experienced any safety issues with this LWC:

14. What is the local public perception of this LWC:

Completed Surveys: Completed surveys may be returned to heidi.r.howard@us.army.mil

OR

ERDC-CERL
ATTN: H. Howard
P.O. Box 9005
Champaign, IL 61826

APPENDIX B: ILLINOIS LWC SURVEY RESULTS

Illinois LWC Survey

A survey was conducted as a part of the research to obtain an overview of the distribution of LWCs in Illinois, along with county engineers' experiences with LWCs pertaining to design, construction, and maintenance. The survey questionnaire consisted of a document file with 14 questions (Appendix A) and a spreadsheet to document information on multiple LWCs. In the first phase, survey responses were obtained from only 32 out of 102 counties in Illinois. Hence, the survey questionnaire was sent again, and survey responses were received from 23 additional counties. Even though some of the counties' personnel did not respond to the survey, the survey responses from the Illinois Department of Natural Resources (IDNR) and the Illinois Department of Transportation (IDOT) contained the details on LWCs in various counties and were included in our survey response summary. Figure B.1 shows the participation of all the agencies in the survey.

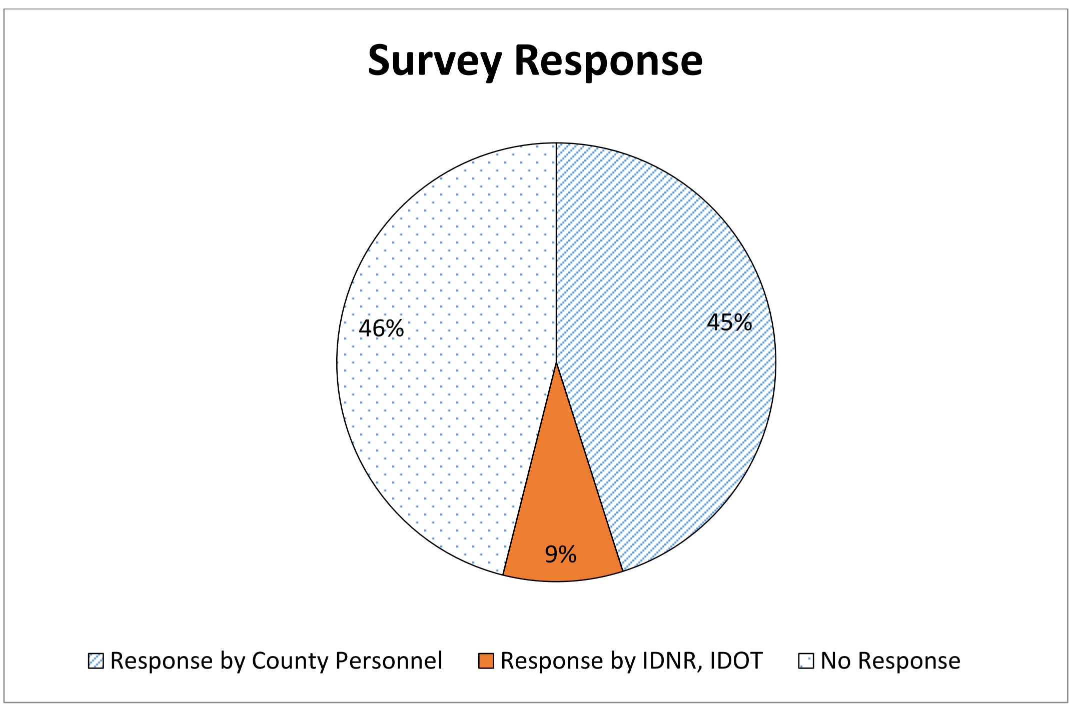

Figure B.1: Responses for Illinois LWCs survey.

The survey responses are divided into three categories:

- Counties that did not respond (47)
- Responding counties that have LWCs (37)
- Responding counties that have no LWCs (18)

The responses from different counties and the distribution and location of LWC structures within the counties are shown in Figure B.2. The counties with orange dots in the background did not respond to the survey, the counties with a hatched background do not have any LWC structures, and the counties with a white background have LWCs in the location marked by purple dots. Based on the responses received, it can be seen that the southern part of Illinois has more LWC density compared to the northern part.

Out of the 55 counties in Illinois for which we received information about LWCs, 18 counties indicated they do not have any LWCs. A total of 155 LWCs were identified in the remaining counties, which include unvented fords, vented fords, and bridges. Figure B.3 is the chart showing the number of low-water crossing structures in each category.

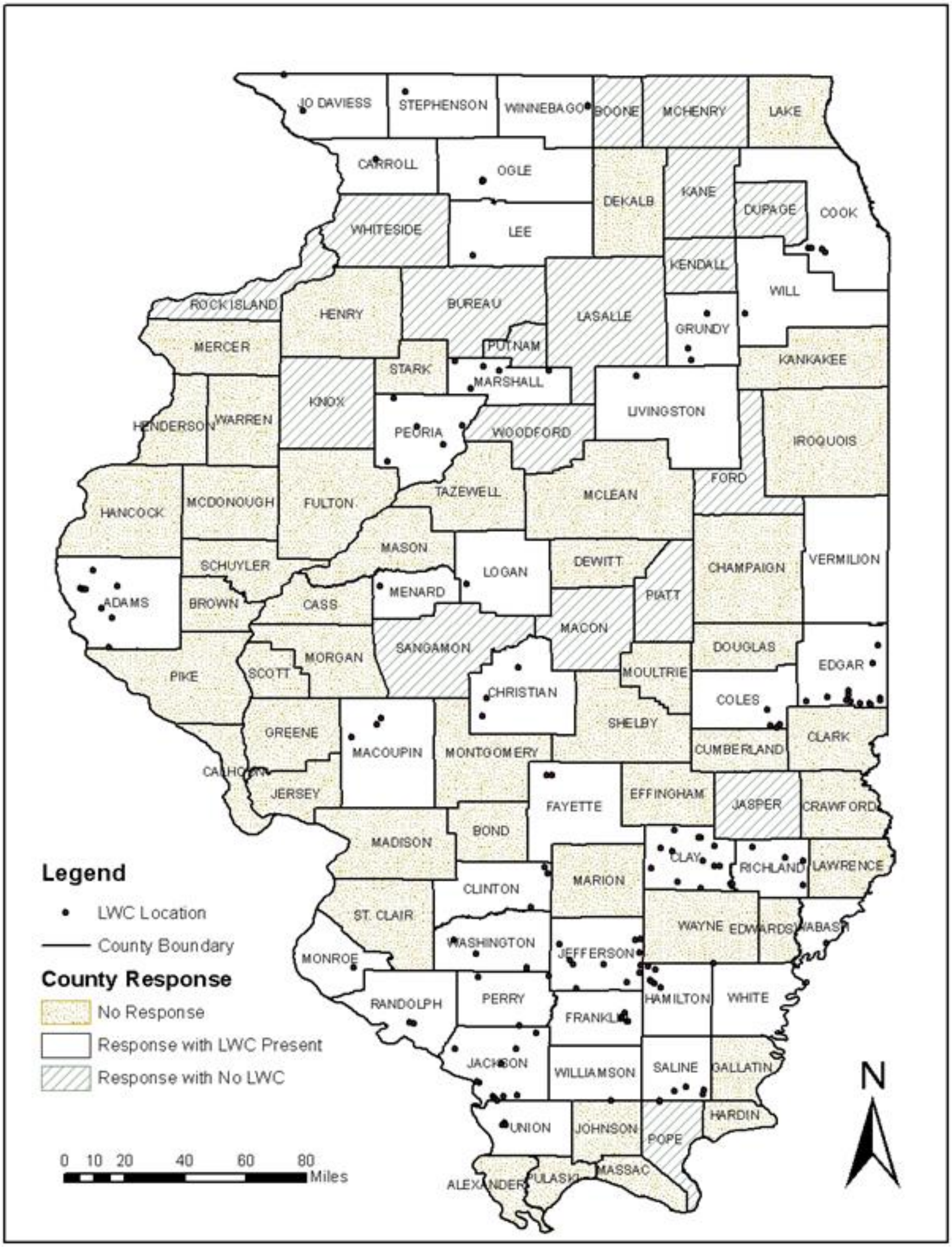

Figure B.2: Location of LWCs in the Illinois counties.

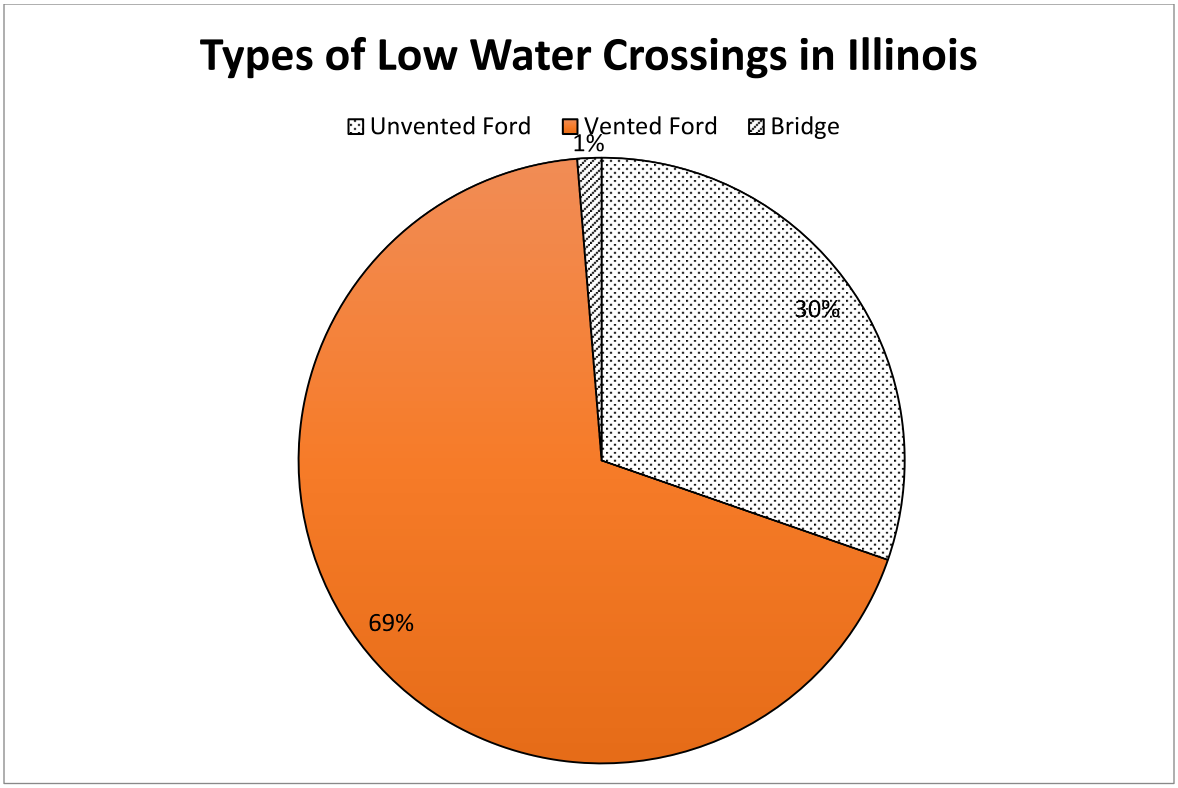

Figure B.3: Types of LWCs in Illinois.

Vented fords are the most popular LWCs currently being used, with 106 vented fords. Of the 47 unvented fords, 33 are at-grade structures.

Most of the county highway departments do not have information about the design storm and design specifications that were used in the construction of these LWCs.

These low-water crossings are used for a variety of purposes: farmland access, residence access roads, park roads, forest roads, drainage, a roadway for general traffic, etc. The bar graph in Figure B.4 shows the intended uses of the LWCs in Illinois.

Adams County is planning to construct a LWC in the Ellington Road District over Little Mill Creek, with a 10-year return period as the design storm. Jo Daviess County and Christian County also have plans to build one and two new LWCs, respectively.

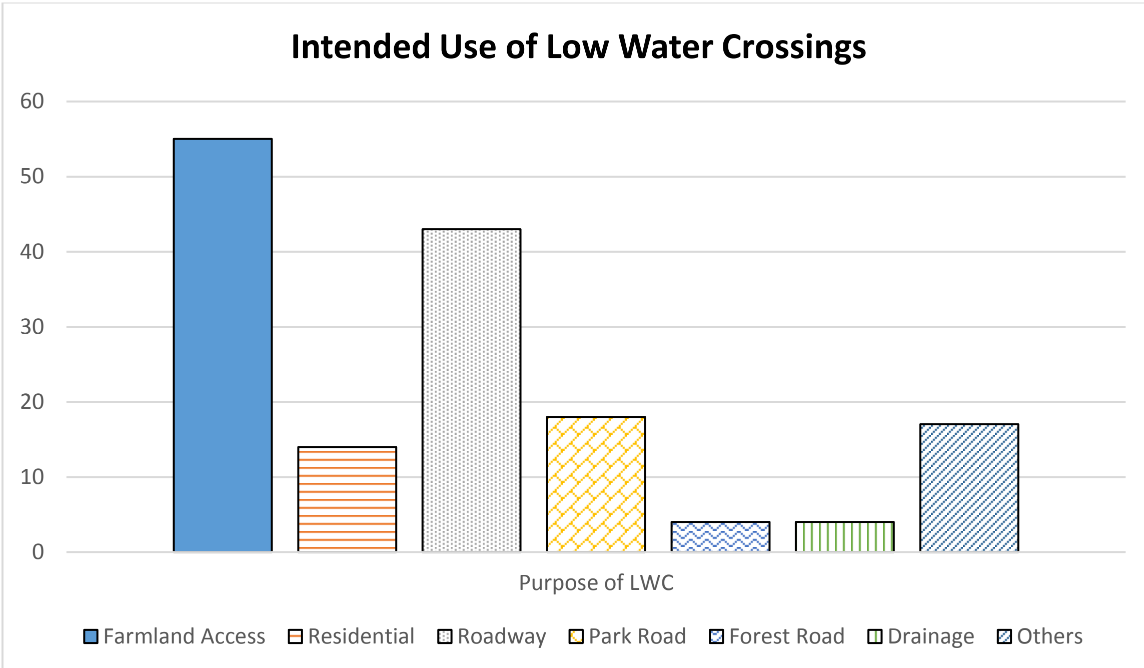

Figure B.4: Use of LWCs in Illinois.

The following tables summarize the average daily traffic (ADT) over the crossings (Table B.1) and the frequency of overtopping of the LWCs (Table B.2), as reported by the county engineers.

Table B.1: Average Daily Traffic (ADT) Over LWCs

Average Daily Traffic (ADT)	No. of LWCs
100 to 200	13
25 to 100	19
25 or less	78
Unknown	45

Table B.2: Frequency of Overtopping of LWCs

No. of Overtoppings per Year	No. of LWCs
250 or more	11
100 to 250	8
25 to 100	3
10 to 25	26
Less than 10	44
Unknown	63

Thirty-six of the LWCs have warning and information signs present, whereas the majority of the existing LWCs (119) have no warning signs on the approach road.

Of the 155 LWCs, 78 are functioning smoothly, with no safety and maintenance issues, whereas 77 of them are facing some issues. The most prevalent maintenance issues include the following:

- deposition of sediment and debris on the upstream side
- blockage of pipes/vent by sand and debris
- scouring of the crossing surface
- scouring of the downstream end
- washing out of riprap
- aging of the structure

Public perception about most of the LWCs is positive, but it is believed that some LWCs are narrow and inadequate. Hence, users want LWCs of adequate capacity to be installed and repaired in a timely manner.

APPENDIX C: REGIONAL FLOW DURATION CURVES FOR ILLINOIS

Flow-Duration Approach

The flow-duration curve (FDC) is a plot that indicates the percentage of time that the flow in a stream of interest is equaled or exceeded. The exceedance probability (e) can be used to determine the number of times per year a LWC will be closed. For example, a 5% exceedance probability means that the crossing will be closed, on average, for 18 days a year (5% time of a year) and 2% exceedance probability gives the closing time at 7 days in a year. During those days, the design discharge is equaled or exceeded, and the LWC is overtopped.

A flow-duration curve for gauged streams can be prepared based on the available daily streamflow data. It is recommended to use long-term data because extreme values are averaged out more over a longer time period. The steps to obtain the FDC are as follows:

Step 1: Sort the daily discharge values for the period of record from the largest to the smallest value.

Step 2: Assign a rank to each of the discharge values, starting with the one that has the largest daily discharge value.

Step 3: Compute the exceedance probability (P) using the following formula:

$$P = \frac{m}{n+1} * 100$$

P = probability that a given flow will be equaled or exceeded (% of time)

m = ranked position on the list

n = total number of events in record

Step 4: To obtain the FDC, plot the discharge vs. percentage of time that a particular discharge was equaled or exceeded.

The FDC for ungauged catchments in Illinois is discussed in a USGS report, *Estimation of Regional Flow-Duration Curves for Indiana and Illinois* (Over et al. 2014). The study encompasses most of the area in Illinois, dividing the state into three different regions. Two methods are discussed in the study: drainage area–only equations and multiple regression equations.

The drainage-area ratio (DAR) method is more applicable for LWCs because the only parameter required in this method is drainage area in square miles. The equation used in DAR method is

$$\log_{10} Q = i + a_1 \log_{10} DA$$

Q = discharge (cfs)

DA = drainage area (mi^2)

i and a_1 = coefficients

On solving, we get

$$Q = 10^i (DA)^{a_1}$$

The equation can be simplified as

$$Q = b\ (DA)^{a_1}$$

$$where, b = 10^i$$

Intercepts (i) and coefficients (a_1) for different regions and the corresponding flow-duration-area curves are provided in Figures C.1 through C.4 and Tables C.1 through C.3.

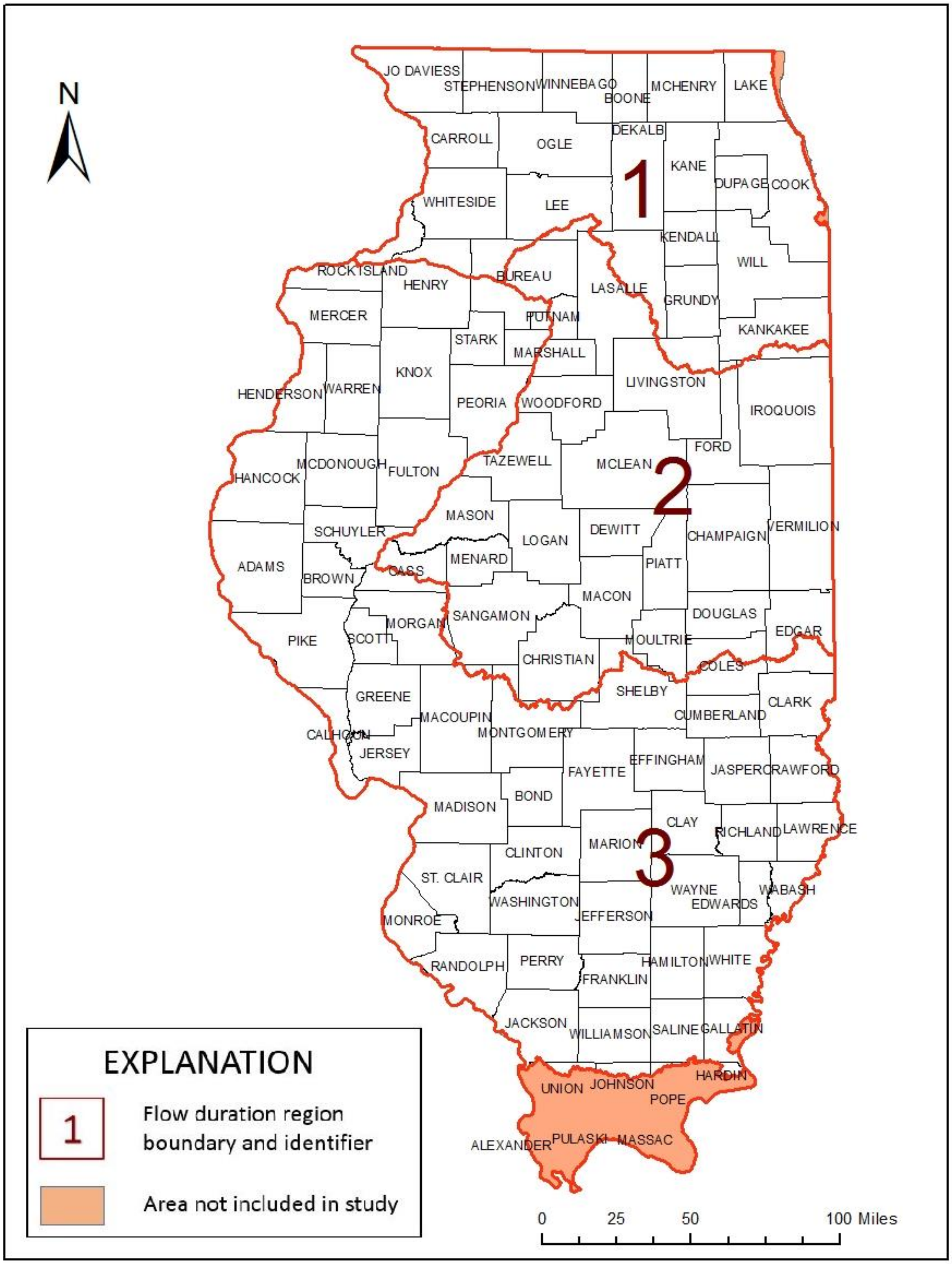

Figure C.1: Regions in Illinois used to obtain FDC (reproduced after Over et. al 2014).

Region 1

Table C.1: Parameters to Estimate FDC for Region 1 in Illinois (from Over et. al 2014)

Exceedance Probability (e)	Intercept (i)	$b=10^i$	a_1
99.9	–3.131	0.0007	1.381
99	–1.836	0.0146	0.938
95	–1.272	0.0535	0.957
90	–1.029	0.0935	0.927
75	–0.686	0.2061	0.915
50	–0.419	0.3811	0.970
25	–0.0897	0.8134	0.976
10	0.214	1.6368	0.986
5	0.455	2.8510	0.961
2	0.786	6.1094	0.914
1	1.04	10.9648	0.868
0.5	1.261	18.2390	0.826
0.1	1.72	52.4807	0.729

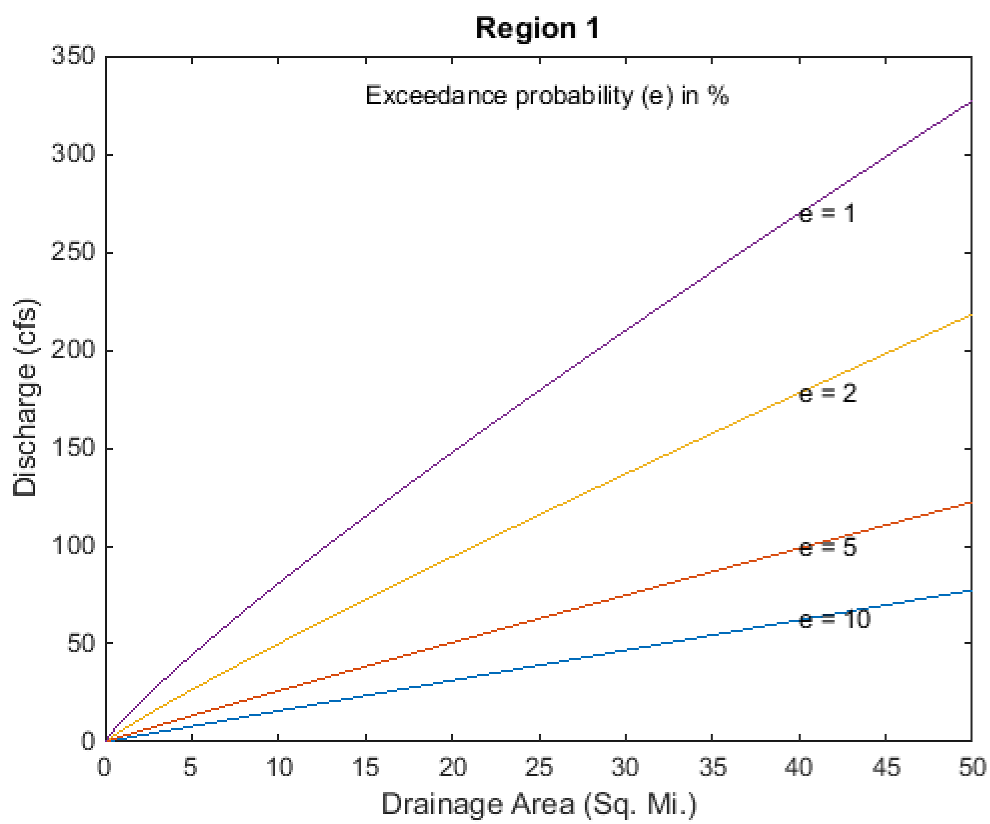

Figure C.2: FDC for different exceedance probabilities for Region 1 in Illinois.

Region 2

Table C.2: Parameters to Estimate FDC for Region 2 in Illinois (from Over et. al 2014)

Exceedance Probability (e)	Intercept (i)	$b = 10^i$	a_1
99.9	–7.310	0.00000005	2.483
99	–5.637	0.00000231	2.108
95	–4.892	0.00001283	2.017
90	–3.799	0.00015873	1.725
75	–2.166	0.00682193	1.327
50	–0.816	0.15259946	1.082
25	–0.259	0.55090146	1.051
10	0.150	1.41187448	1.041
5	0.428	2.67832454	1.016
2	0.761	5.76666409	0.975
1	0.991	9.79854005	0.942
0.5	1.217	16.48763297	0.906
0.1	1.630	42.69043253	0.839

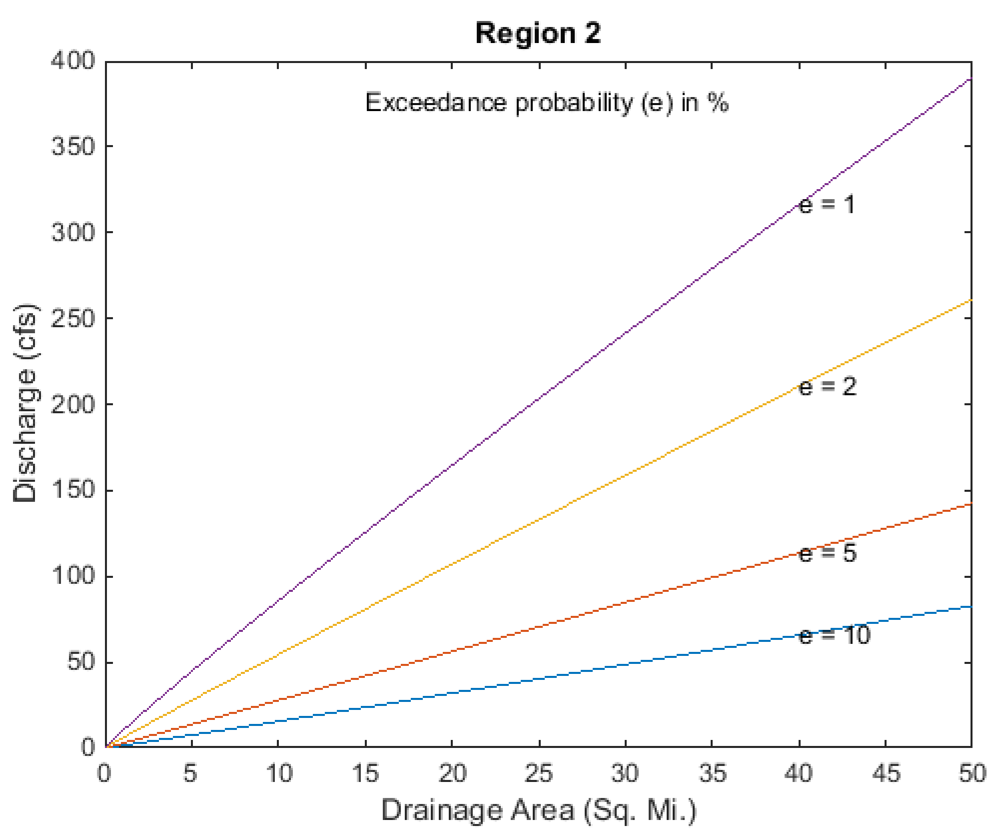

Figure C.3: FDC for different exceedance probabilities for Region 2 in Illinois.

Region 3

Table C.3: Parameters to Estimate FDC for Region 3 in Illinois (from Over et. al 2014)

Exceedance Probability (e)	Intercept (i)	$b = 10^i$	a_1
99.9	–8.251	0.00000001	2.589
99	–6.492	0.00000032	2.276
95	–4.962	0.00001092	1.965
90	–3.9682	0.00010759	1.719
75	–2.611	0.00244873	1.432
50	–1.237	0.05793078	1.161
25	–0.653	0.22226750	1.154
10	–0.052	0.88655626	1.113
5	0.401	2.51737781	1.046
2	0.964	9.21411610	0.933
1	1.299	19.91891095	0.867
0.5	1.558	36.11465579	0.820
0.1	1.948	88.66374680	0.761

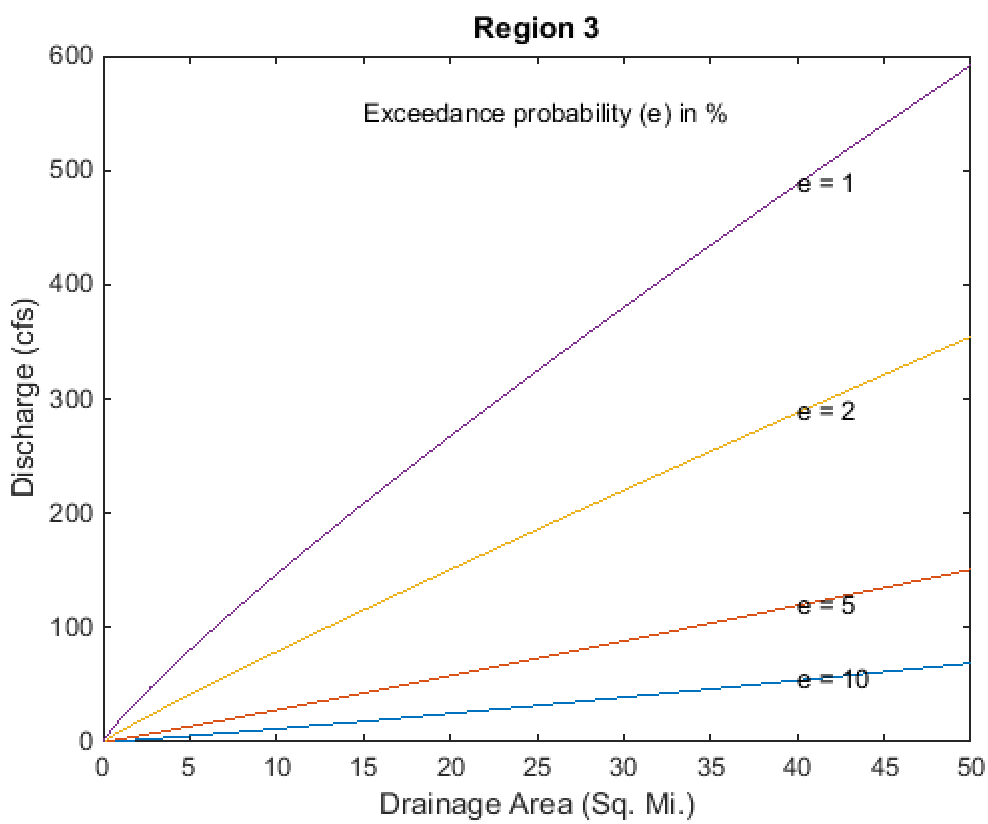

Figure C.4: FDC for different exceedance probabilities for Region 3 in Illinois.

APPENDIX D: PARTIAL DURATION SERIES REGIONAL EQUATIONS

Partial Duration Series (PDS) Regional Equations

The lowest peak discharge that StreamStats gives is Q2, which has a return period of 2 years. In certain areas, using this discharge to design the structure may result in a larger structure than required. In such cases, partial duration equations can be employed to obtain design discharge of return periods of 0.8 years, 1 year, 1.5 years, etc.

More information on PDS regional equations can be found in the USGS report, *Estimating Flood-Peak Discharge Magnitudes and Frequencies for Rural Streams in Illinois* (Soong et al. 2004). The seven hydrologic regions used for the PDS regional equations are given in Figure D.1. The regional equations are as follows:

$Q_T = a(TDA)^b(MCS)^c(\%water + 5)^d$ [Region 1]

$Q_T = a(TDA)^b(BL)^c(PermAvg)^d$ [Region 2]

$Q_T = a(TDA)^b(\%water + 5)^c$ [Region 3]

$Q_T = a(TDA)^b(MCS)^c(BL)^d$ [Region 4]

$Q_T = a_N(TDA)^{b_N}(MCS)^c(\%water + 5)^d$ [Region 5, 6, 7]

Values for the parameters TDA, MCS, %water, PermAvg, and BL can be obtained from StreamStats. A table of parameters a, b, c, and d used in different regions can be found in Soong et al. (2004).

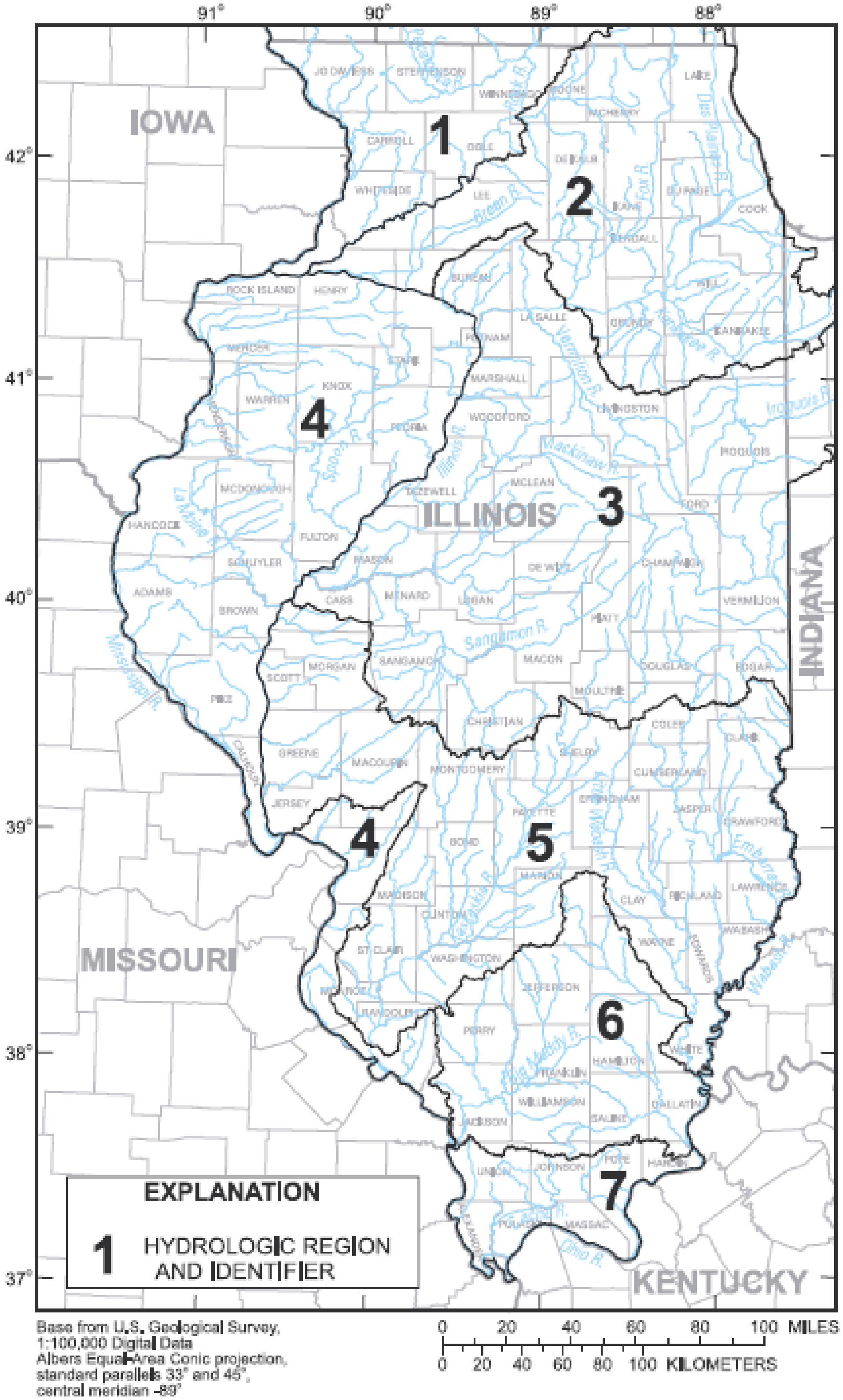

Figure D.1: Hydrologic regions for PDS regional equations (adapted from Soong et. al 2004).