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

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16. Abstract The Illinois Department of Transportation (IDOT) and local agencies monitor and regulates the 146,764 mi 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 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 oversized or undersized. 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 from a survey conducted among the county engineers in Illinois about their experience with LWCs are presented, along with commonly used LWCs, site considerations, selection criteria, and signage requirements. Design criteria and procedure for the LWCs design, construction, and best management practices are also discussed. Additionally, case studies, design examples, and permitting requirements for LWCs are included in the report. Implementation of LWC guidelines could save local agencies significant funding, due to lower construction and maintenance costs, less channel and flood plain blockage, and better adaptability and storm-proofing characteristics, as well as reduced impacts to aquatic organism passage.					
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EXECUTIVE SUMMARY

This document provides guidelines for the structure selection and design of low-water crossings (LWCs) at rural sites in Illinois with low average daily traffic (ADT). The materials in this document are based on case studies done by researchers at the University of Illinois at Urbana-Champaign, previous reports from USDA-CERL, USDA Forest Service, Federal Highway Administration, Iowa State University, Texas Transportation Institute, and various other organizations as listed in the references section.

A comprehensive survey was conducted across Illinois to determine use and status of LWCs. The survey was given to county engineers, and it provided a wealth of information on LWC utilization and history for use in preparing of the design guidelines.

The Illinois Department of Transportation (IDOT) has been implementing best management practices (BMPs) for environmental sustainability. To minimize the effects of roadway construction, IDOT requested guidance on LWC design impacts to stream hydrology, human safety, aquatic health, and organism passage.

To make this document more approachable to county engineers, theoretical background and complex design principles are provided in their respective sections and references.

CONTENTS

CHAPTER 1: INTRODUCTION.....	1
1.1 BACKGROUND	1
1.2 OBJECTIVES	2
1.3 LOW-WATER CROSSING TYPES.....	3
1.4 CURRENT STATUS OF LWC DESIGN GUIDELINES.....	8
CHAPTER 2: ILLINOIS LWC SURVEY	10
2.1 KEY FINDINGS	14
CHAPTER 3: LWC SITE CONSIDERATIONS	15
3.1 LWCS AND ENVIRONMENT	15
3.2 AQUATIC ORGANISM PASSAGE.....	16
3.3 SUITABILITY OF LWC	18
3.4 SELECTION OF LWC TYPE.....	21
3.5 LWC SIGNAGE.....	21
3.6 LWC ECONOMICS.....	31
CHAPTER 4: LWC DESIGN GUIDELINES	33
4.1 GENERAL DESIGN GUIDELINES	33
4.2 SITE HYDROLOGY.....	35
4.3 HYDRAULICS.....	43
4.4 DESIGN OF UNVENTED FORDS	44
4.5 DESIGN OF VENTED FORD	46
4.6 DESIGN OF LOW-WATER BRIDGES.....	54
4.7 MATERIALS SELECTION AND CONSTRUCTION	56
4.8 BEST MANAGEMENT PRACTICES	58
CHAPTER 5: LWC CASE STUDIES.....	61
5.1 STUDY SITES	61
5.2 HEC-RAS IN LWC STUDY	62
5.3 LOGAN LWC.....	66
5.4 OTHER CASE STUDIES.....	73

CHAPTER 6: SUMMARY74

REFERENCES75

APPENDIX A: SURVEY QUESTIONNAIRE FOR LWCS IN ILLINOIS.....80

APPENDIX B: LOW-WATER CROSSINGS—INITIAL SITE ASSESSMENT SUMMARY83

APPENDIX C: INITIAL SITE ASSESSMENT FOR FEASIBILITY OF LWC CONSTRUCTION.....85

APPENDIX D: MANNING’S N VALUES86

APPENDIX E: ILLINOIS ENDANGERED AND THREATENED SPECIES LIST87

APPENDIX F: ILLINOIS LOW-WATER CROSSINGS PERMITTING90

APPENDIX G: ADDITIONAL CASE STUDIES96

APPENDIX H: DESIGN EXAMPLES132

APPENDIX I: LWC INVENTORY AND INSPECTION CRITERIA137

CHAPTER 1: INTRODUCTION

1.1 BACKGROUND

Illinois has 146,764 mi of road; the Illinois Department of Transportation (IDOT) maintains 15,978 mi, and local agencies maintain the rest (IDOT 2016). Combined, 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, that 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.

According to the Federal Highway Administration (FHWA), low-water crossings include public road waterway crossings other than bridges where construction improvements have been made in the stream, river, or lake bed to provide a firm surface for vehicles to travel across the watercourse (FHWA 1987).

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 construction of a LWC.

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).

1.2 OBJECTIVES

The overall objective of this research is 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 project will assist IDOT and local agencies in determining safe, cost-effective, and environmentally friendly design of LWCs for low ADT routes in the state. In addition to the geomorphic, biological, and engineering considerations at each crossing, this research will also take into account the upstream watershed conditions such as land use/land cover, topography, and soil conditions, which, as a whole, will determine discharges and high-water elevations.

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. Using LiDAR and field data from the selected LWCs, conduct case studies using the analytical tools Illinois StreamStats, flow-duration curves (FDC) and HEC-RAS; and evaluate LWC performance (with consideration of effects from soil conditions, geography, and environmental factors when applicable).
5. Based on the literature review, survey results, and case studies, develop design guidelines for applications specific to Illinois.
6. Identify suitable best management practices (BMPs) for installation/construction and utilization of LWCs, and provide short- and long-term maintenance recommendations for selection and implementation of identified BMPs.

1.3 LOW-WATER CROSSING TYPES

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 1.1 through 1.4).

1.3.1 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 in. 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 1.1).
- *Improved fords* have a stable driving surface of rock, concrete, asphalt, concrete blocks, concrete planks, gabions, geocells, or a combination of materials.



Figure 1.1: Unvented ford across Big Creek in Hamilton County, Illinois.

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

1.3.2 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 due to the fact that the vented ford is overtopped by higher flows. 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 1.2), 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 1.2: Vented ford in Jackson County, Illinois.

1.3.3 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 are able to 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 1.3).



Figure 1.3: Low-water bridge at Montgomery County, North Carolina (adapted from Filer, 2008).

1.3.4 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:

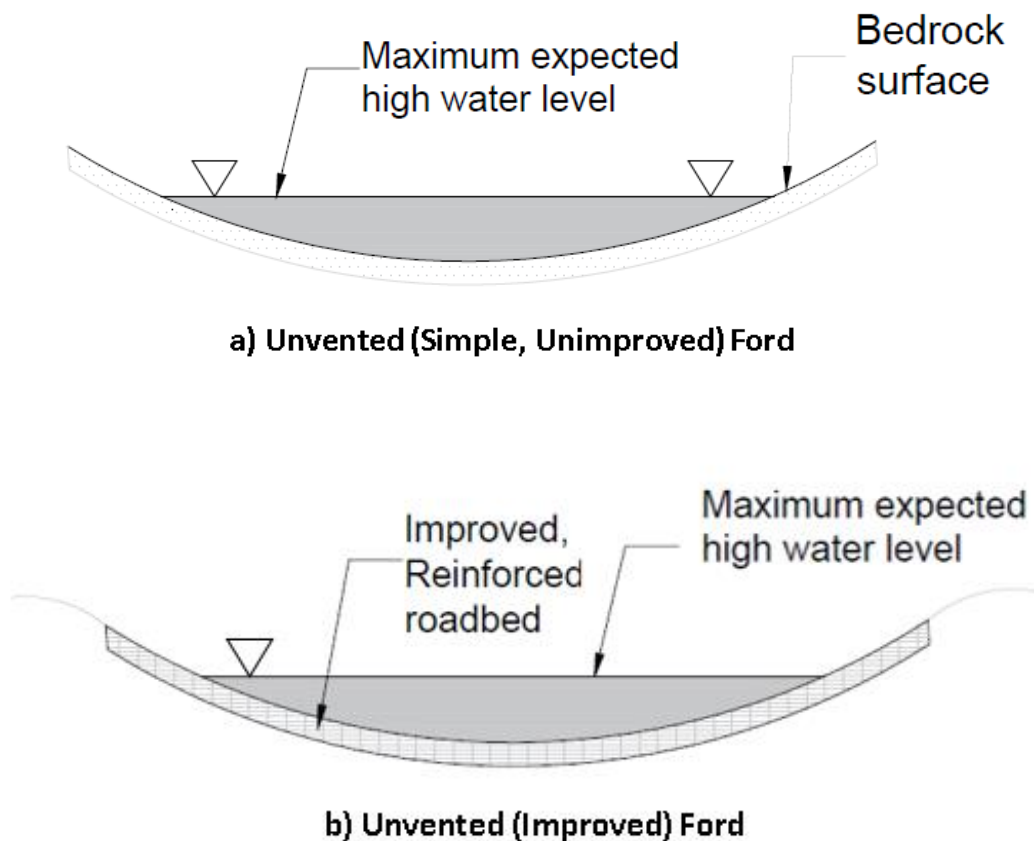
- Structures designed for overtopping, hence less damage during overtopping
- Less likely than culverts to be plugged and damaged by debris or vegetation
- Less expensive than culverts or bridges
- Less susceptible than other structures to failing during flows higher than the design flow

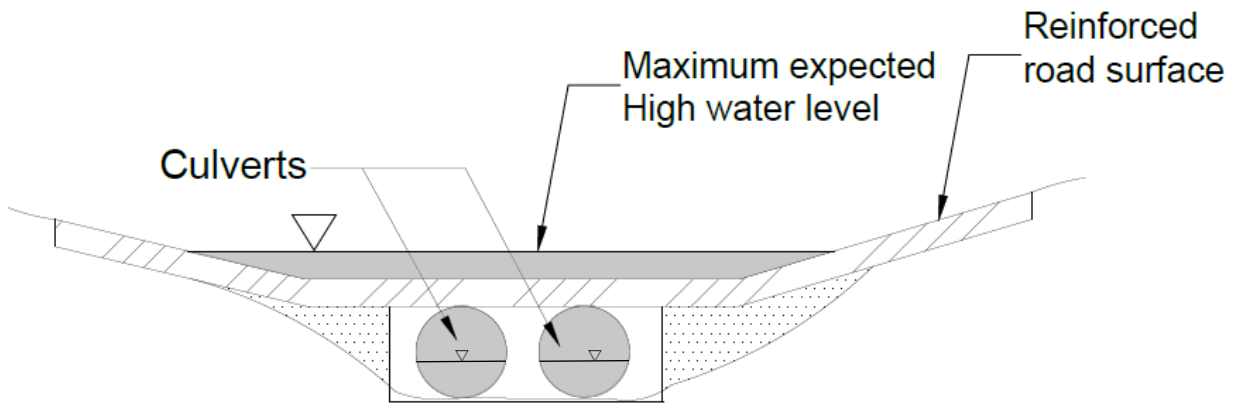
- Good for storm proofing roads where large amounts of sediment and debris are expected, such as after a large storm event or forest fire
- Readily available materials and fast construction

Some of the disadvantages of LWCs are as follows:

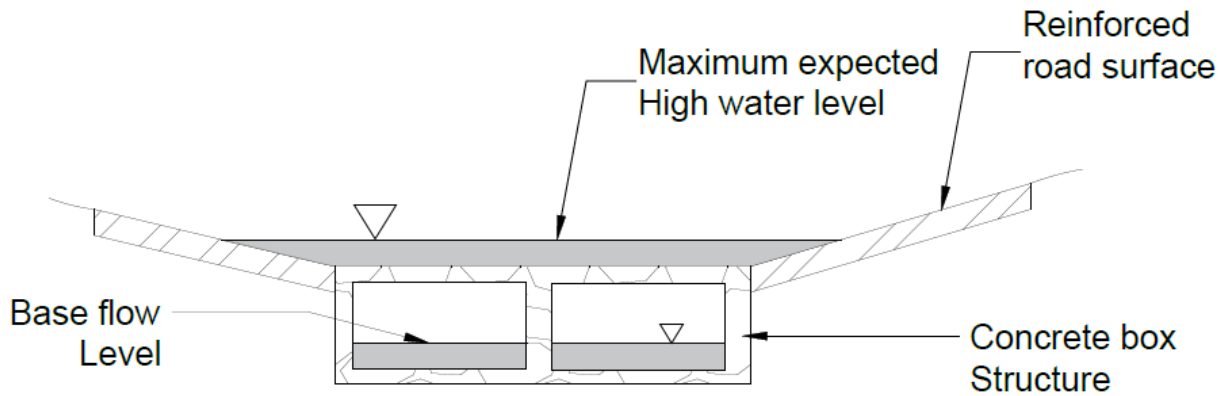
- Periodic or occasional traffic delays during high-flow periods
- Not well-suited to deeply incised channels
- Not desirable for high use or high-speed roads
- Can be difficult to design for aquatic organism passage
- Can be dangerous to traffic during high-flow periods
- Periodic maintenance is required

Figure 1.4 is a schematic of different LWC types:

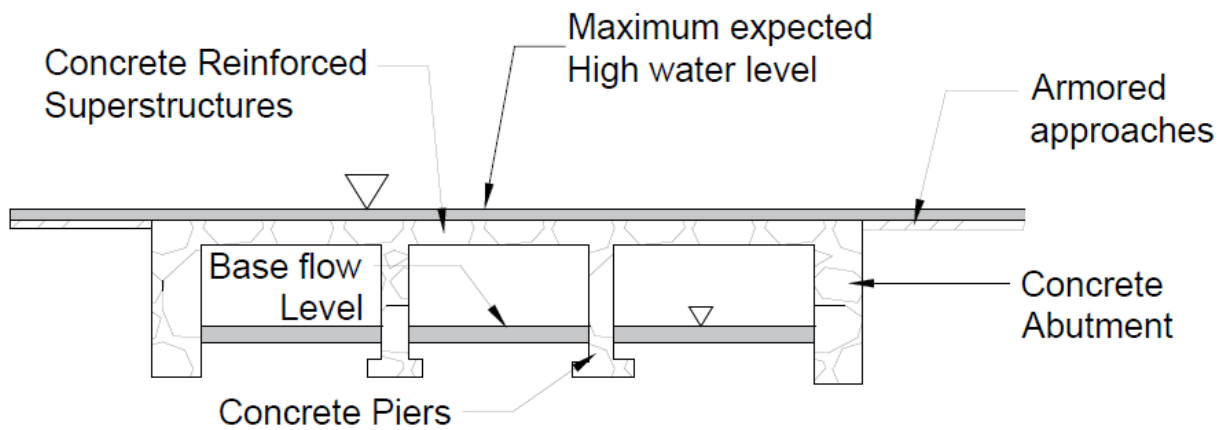




c) Vented Ford with Traditional Culverts (Low VAR)



d) Vented Ford with Concrete Box Culverts (High VAR)



e) Low Water Bridge

Figure 1.4 (a–e): Commonly used LWCs (redrawn after Clarkin et al. 2006).

1.4 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 on 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 University of Illinois at Urbana-Champaign (Svendsen et al. 2006) in which design and testing of LWCs for military operations is 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 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 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 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 on 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.

CHAPTER 2: ILLINOIS LWC SURVEY

A survey was conducted as a part of the current research project 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 2.1 shows the participation of all the agencies in the survey.

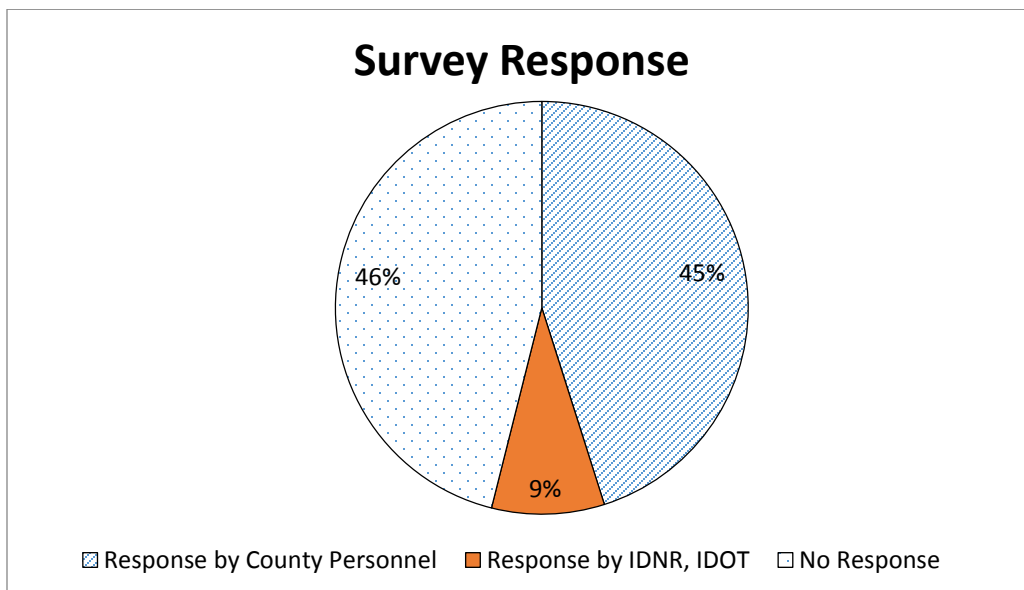


Figure 2.1: Responses for Illinois LWCs survey.

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 2.2. 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.

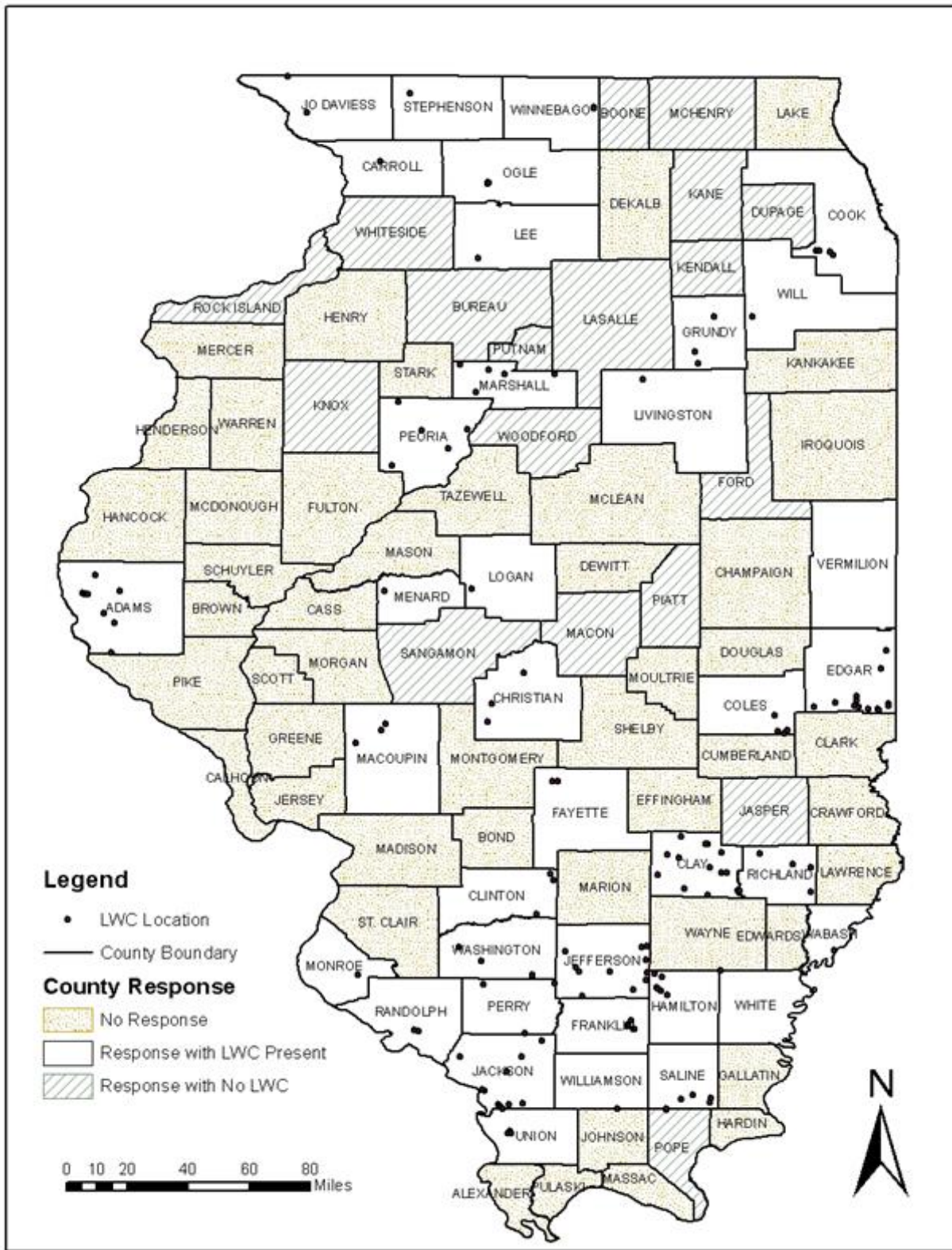


Figure 2.2: Location of LWCs in the Illinois counties.

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 2.3 is the chart showing the number of low-water crossing structures in each category.

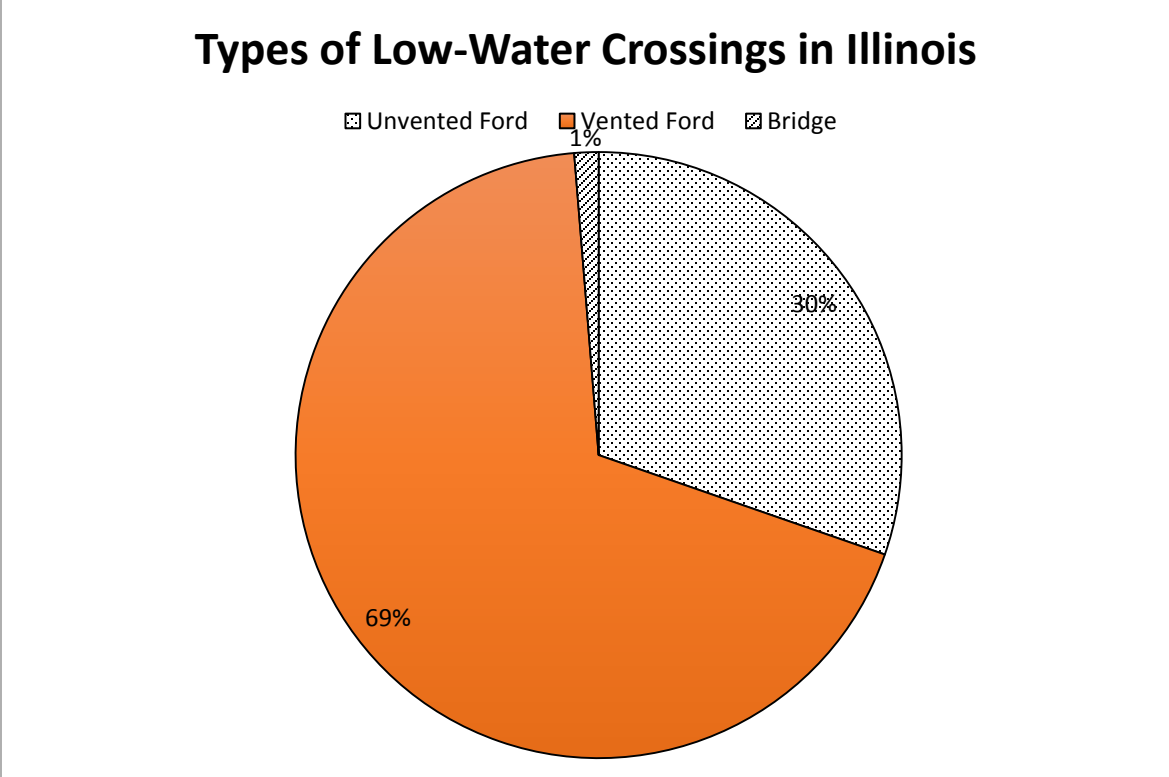


Figure 2.3: Types of LWCs in Illinois.

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, roadway for general traffic, etc. The bar graph in Figure 2.4 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.

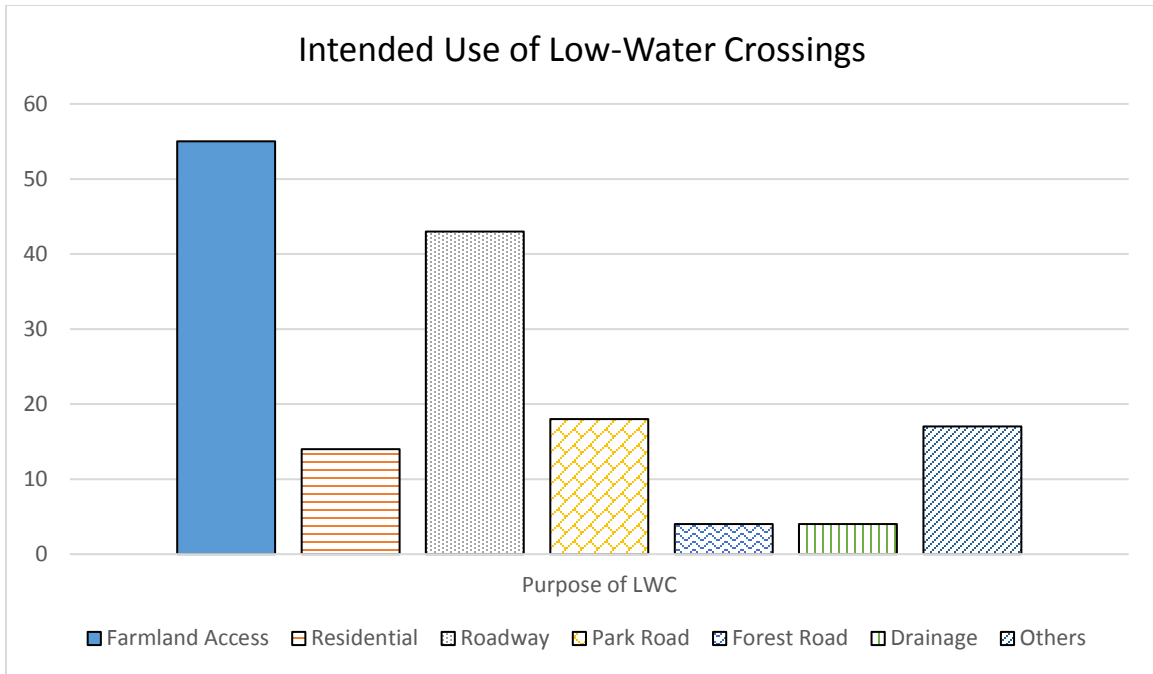


Figure 2.4: Use of LWCs in Illinois.

The following tables summarize the average daily traffic (ADT) over the crossings (Table 2.1) and the frequency of overtopping of the LWCs (Table 2.2), as reported by the county engineers.

Table 2.1: 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 2.2: 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 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.

2.1 KEY FINDINGS

- 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 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.

CHAPTER 3: LWC SITE CONSIDERATIONS

3.1 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 generally 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. 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 subject to runoff and gullies at the ingress and egress of the crossings. When heavy vehicles ford 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 further geomorphological instability of the stream.

Vented fords keep vehicle tires dry during base flow conditions, keeping soils and other pollutants from vehicles from 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 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 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 can 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.

3.2 AQUATIC ORGANISM PASSAGE

Culverts and LWCs, if not designed and installed properly, can act as barrier to fish passage. Installation of LWCs at any site disturbs the natural regime of the channel. Common ways in which LWCs create 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, changes in distribution of the aquatic organisms in the stream, etc.

LWCs may have 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) looked into movement of fish for 21 different species in seven families through 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 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 3.1). 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 same
- Crossing has dry banks for wildlife passage



Figure 3.1: Culvert for AOP, which mimics the natural bedstream (from Barnard et al. 2013).

Fords with slots or small channel to allow AOP during very low flows provide 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 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. Lists of endangered and threatened species of fishes, amphibians, and reptiles are provided in Appendix E. A current version of the list, which can be obtained from Illinois Department of Natural Resources (IDNR) should be consulted.

The following alternatives are also recommended to be consulted:

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).

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.

3.3 SUITABILITY OF LWC

While deciding whether to use a LWC, factors such as site conditions, associated cost of construction, channel characteristics, flow characteristics, AOP needs, and other factors should be taken into account. LWCs are not appropriate on roads that access essential public facilities or that serve as the only public route to an area. Many states restrict construction of LWCs on school bus routes and on

roads required for national defense. Typically, LWCs are not desirable for accessing an area with permanent residences.

Fords with low vent-area ratio (VAR) are undesirable because they act as low dams when the flow in the channel exceeds the vents' capacity, and the backwater may lead to bank erosion and channel widening. It is recommended to use a high VAR ford when a vented ford is selected as a viable option. Bankfull flow is the flow that just overtops the stream banks and begins to flow over the floodplain.

Entrenchment ratio (ER) is defined as the ratio of flood-prone width to the bankfull width. In entrenched channels, ER is usually less than 1.4 because the channel is incised deeply enough that high flows do not overflow the valley floor. It is recommended not to install LWCs in entrenched channels because the flow obstruction in the channel may lead to aggradation and bank erosion.

The suitability of a LWC at a particular site depends on several factors, such as traffic volume, flow conditions, channel stability, etc. The following questions should be considered and taken into account when deciding whether a LWC is a feasible option in the stream (Clarkin et al. 2006; Howard et al. 2011).

- How frequently is the road used? Are there any alternative routes nearby? Is traffic volume or type likely to change in the near future?
- What kind of area does the road serve? What is the speed limit? What size vehicles are expected on the road?
- Is the crossing located near unstable landforms such as alluvial fans or landslide prone areas? Is the site in an active flood plain, or is the channel entrenched?
- Is there a current LWC? If yes, how is it affecting the stream (deposition, aggradation, stream bank erosion)?
- Does the stream have meandering or curved geometry? Is the channel locally stable? What kind of materials does the stream bottom and substrate primarily consist of—rock, cobble, gravel, sand, silt, or clay?
- Is the channel stable at a watershed scale, and are there any planned changes upstream or downstream of the crossing site that might affect its stability?
- What kind of flow is predominant in the stream—base flows or peak flows? Is the flow sediment- and debris-laden? Are flows in the stream highly variable? Are flows flashy?
- What hydrologic changes are likely to occur, and how might those changes affect characteristics such as flow quantity and duration?
- Should the crossing be designed for passage of special aquatic animal species? If so, what are their needs?
- Are there any constraints such as private property, threatened and endangered species, sediment or total maximum daily load (TMDL) requirements, or archeological sites?

- What is the accident history of the road? Do sight distance, geometrics, and design speed permit safe movement on the road?
- What is the public’s opinion? Do they want a LWC?
- What are the maintenance requirements—sediment deposition, ice, logs, or crops?
- What are the scour and erosion control requirements?
- What is the possible cost of construction? What are the permitting requirements?

When deciding whether a LWC is a best fit for the given site and hydrology conditions, it is recommended to use the initial site assessment form given in Appendix C to record the information and to consult Tables 3.1 and 3.2. A selection decision can be made about which LWC is best for the site based on the engineer’s judgment of site conditions and channel cross section on a LWC.

The general selection criteria for LWCs as suggested by Clarkin et al. (2006) are provided in Table 3.1.

Table 3.1: General Selection Criteria for LWCs

	Most Conducive	Least Conducive
Access priority	Low	High
Alternative route	Available	Not available
Traffic speed	Low	High
Average daily traffic	Low	High
Flow variability	High	Low
High-flow duration	Short (hours)	Long (days)
High-flow frequency	Seldom (rare closure)	Often (frequent closure)
Debris loading	High	Low
Channel entrenchment	Shallow	Deep

The selection criteria prepared from a survey of transportation engineers from several states as reported by Motayed et al. (1982) are presented in Table 3.2.

Table 3.2: LWC Selection Criteria

Criteria	Most Favorable for LWC	Least Favorable for LWC
Average daily traffic (ADT)	Less than five vehicles	More than 200 vehicles
Average annual flooding	Less than two times	More than 10 times
Average duration of traffic interruption per overtopping event	Less than 24 hours	More than 3 days
Extra travel time for alternate route	Less than 1 hour	More than 2 hours
Possibility of danger to human life	Less than 1 in 1 billion	More than 1 in 100,000
Property damage, US Dollars	None	1 million
Frequency of use as an emergency route	None	Frequent

3.4 SELECTION OF LWC TYPE

Selection of a LWC for a particular location depends on several factors such as traffic volume, stream channel conditions, anticipated flow in the stream, cost of construction and maintenance, etc. Some of the LWC types suggested for different conditions are as follows (Clarkin et al. 2006):

- An unimproved ford is suitable for an area that has an ephemeral stream with low base flow and where AOP is an important consideration.
- If the channel bottom is unstable and erodible, an improved ford with a hardened crossing is preferred.
- If the stream carries a large amount of sediment and debris, an unimproved or improved unvented ford is suitable.
- If the depth of water above the LWC is higher than 6 in., or driving through the water is not desired, a vented ford should be selected.
- For an incised channel, a vented box culvert is a suitable option.
- For a broad channel or stream associated with large base flow and high flood peaks, a low-water bridge should be considered.
- If a barrier is needed to exclude exotic species, an improved unvented ford with a raised platform or a raised vented ford with a perched outlet should be considered.

After the appropriate LWC type is selected for a particular site, required permits should be obtained from the proper authorities (USACE, Illinois DNR, and Illinois EPA). Details on the required permits are provided in Appendix F.

3.5 LWC SIGNAGE

3.5.1 Liability

Although LWCs provide an economical alternative to the replacement of an existing culvert, bridge, or other type of crossings, in some cases there might be potential liability incurred with LWCs that restrain their use (Carstens and Woo 1981). While a LWC does provide a safe alternative to bridges and culverts most of the time, it becomes unsafe to use in instances when the water level rises above the tolerable limit. Flashy streams, which are a common occurrence at a LWC site, can cause loss of life and damage to property. Most commonly used cars and SUV-sized vehicles get carried away by 2 ft of water, or even less in some cases (Balke et al. 2011).

LWCs can be of considerable concern to engineers and governing bodies because of perceived potential legal liability. Tort liability has been the primary concern with LWCs. From a study of serious accidents in LWCs and actual tort claims, caused primarily by LWC use during flood events, Carstens and Woo (1981) concluded that liability is minimized when LWCs are prevented from being used while flooded. With adequate warning, the potential for accidents and subsequent tort claims may actually be decreased by using LWCs rather than deficient and obsolete bridges (Carstens and Woo 1981).

A local agency's tort liability from use of LWCs can be minimized by adopting reasonable selection and design, and providing adequate warning of these structures to the road users.

3.5.2 Safety

Safety of road users is a primary concern in installing a LWC at any site. Static warning signs on the approach to a LWC, markers along the edge of hardened surfaces, and water depth gauges are required for the safety of those using a crossing (Balke et al. 2011; Howard et al. 2011; Lohnes et al. 2001).

Many drivers have trouble judging the speed and depth of water over the road in a LWC and enter the flooded roadway. Drivers who have been using the dry LWC may develop a false sense of confidence. Low visibility at night and muddy water during flooding conditions pose additional problems to drivers in making judgments under these conditions. In the absence of any informative signs, drivers set their own criteria to determine whether the road is passable, which might lead to loss of life and property.

LWCs should be treated similar to flood-prone roadways, and proper signage should be provided for the safety of the road users. Flood advisories and flash flood warnings should be issued to public via radio stations and messaging services so that the drivers can be alert to the floodwater situation.

3.5.3 Signage

General signage practices suggested at the LWC sites are as follows (Balke et al. 2011):

- Provide a water depth gauge and an advance warning sign at the LWC location
- Use advance signs such as *ROAD MAY FLOOD* or *FLOOD AREA AHEAD*
- At the crossing, use *DO NOT CROSS WHEN FLOODED* sign
- Provide stopping sight distance to maximum water height of the LWC
- Use yellow flood gauge and yellow flashers with an active warning system at crossings that are frequently flooded
- If possible, provide another gauge in addition to the flood gauge at the low point of the crossing so that at least one of the flood gauges is clearly visible to the driver
- Use *HIGH WATER/ROAD CLOSED TO THRU TRAFFIC* with flasher assembly for roadways with multiple flood-prone crossings
- Provide markers along the edge of hardened surfaces
- Provide alternative route information at the crossing site in case the crossing is not passable and a detour is necessary

For the safety of those using LWCs, it is recommended to use two warning signs (*FLOOD AREA AHEAD* and *IMPASSABLE DURING HIGH WATER*) and one regulatory sign (*DO NOT ENTER WHEN FLOODED*) in advance of LWCs (Carstens and Woo 1981; Lohnes et al. 2001; Motayed et al. 1982).

The current *Manual on Uniform Traffic Control Devices* (MUTCD) (FHWA 2012) does not have specifications for signs used in LWC areas. It is recommended to add specifications for these signs in the Illinois supplement to the MUTCD.

3.5.3.1 FLOOD AREA AHEAD Sign

The *FLOOD AREA AHEAD* sign (Figure 3.2) is a 30 in. x 30 in. diamond-shaped warning sign with a yellow background, black border, and black lettering. It is recommended to place this sign 750 ft in advance of the low-water crossing or at the last turnaround location for vehicles, whichever is greater in distance. In cases where the low-water crossing is not visible from a distance of 1,000 ft, it is advised to use an additional distance advisory sign placed below the *FLOOD AREA AHEAD* sign. An advisory speed sign can also be placed if the recommended crossing speed is less than the speed limit for the approach roadway.



Figure 3.2: *FLOOD AREA AHEAD* sign.

3.5.3.2 IMPASSABLE DURING HIGH WATER Sign

The *IMPASSABLE DURING HIGH WATER* sign (Figure 3.3) is a 30 in. x 30 in. diamond-shaped warning sign with a yellow background, black border, and black lettering. This sign should be placed about 450 ft in advance of the low-water crossing. It is recommended to use a gauge board sign that displays the depth of water at the deepest point on the roadway as a supplement to the *IMPASSABLE DURING HIGH WATER* sign at the low-water crossing.



Figure 3.3: *IMPASSABLE DURING HIGH WATER* sign (MoDOT 2016a).

3.5.3.3 DO NOT ENTER WHEN FLOODED Sign

The *DO NOT ENTER WHEN FLOODED* sign (Figure 3.4) consists of a 24 in. x 30 in. rectangular sign with black lettering and a black border on a white background. Because this is a regulatory sign, installation and enforcement require an appropriate resolution by the board of supervisors or city council. This sign should be installed about 200 ft in advance of the actual low-water crossing.



Figure 3.4: *DO NOT ENTER WHEN FLOODED* sign (NDOR 2009).

3.5.3.4 LOW WATER CROSSING Sign

Additionally, a *LOW WATER CROSSING* sign (Figure 3.5) may be used to warn people of a low-water crossing ahead. It is a diamond-shaped sign with yellow background, black border, and black lettering. The sign should be placed 500 to 700 ft in advance of the low-water crossing.



Figure 3.5: *LOW WATER CROSSING* sign (MoDOT 2016b).

3.5.4 Dynamic Signs

Apart from static signage, dynamic message signs displaying messages such as *CAUTION—WATER ON ROAD* or *ROAD CLOSED AHEAD DUE TO FLOODING* may be installed along with flashing lights, based on the use of the road, the budget of the public agency, and the frequency of flooding at the LWC site.

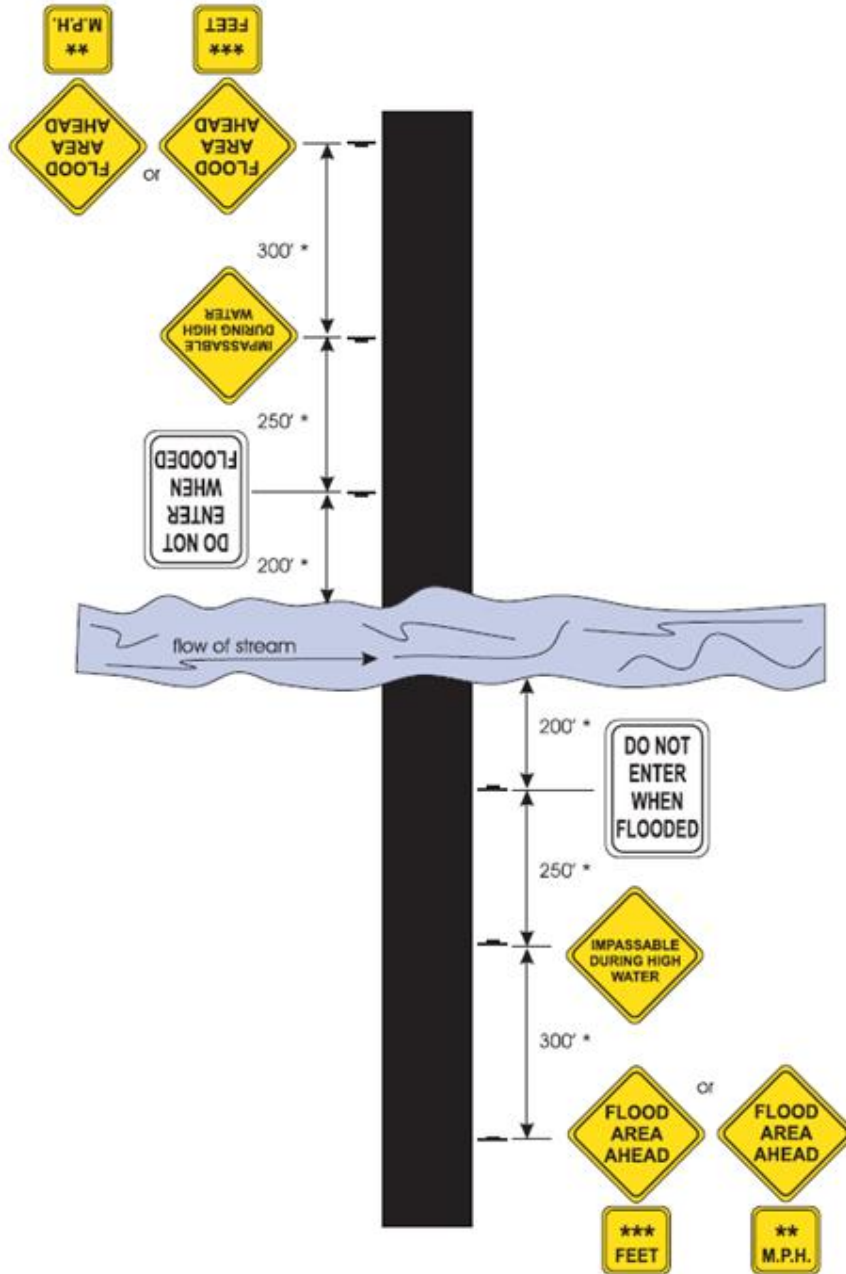
3.5.5 Additional Practices

Depending on necessity, flood control systems such as Floodway® Roadway gate (Floodbreak 2016) may be installed. It is a passive automatic flood mitigation system that is primarily useful on roads with continuous and heavy use. The system works without human intervention because it uses the power of rising floodwater to raise the structure and provide protection. During normal conditions, it allows for normal uninterrupted flow of traffic. Systems such as this might be costlier to install in the beginning, but in the long run they may be cheaper and provide additional benefits when it comes to the safety of humans and property.

A schematic of low-water crossing signage is illustrated in Figure 3.6. Signage in a LWC area should start at least 750 ft ahead of the crossing on both sides of the crossing. The *FLOOD AREA AHEAD* sign should be placed along with the posted speed. The *IMPASSABLE DURING HIGH WATER* sign should be

placed 450 ft ahead, and the *DO NOT ENTER WHEN FLOODED* sign at 200 ft, as shown in the schematic in Figure 3.6.

In areas where water can back up to more than 750 ft on the either side of the structure during the flooding events, it is recommended to place the *FLOOD AREA AHEAD* sign more than 750 ft from the structure, in addition to the minimum requirement discussed previously.



* Nominal distance (other distance may be used if engineering study indicates).

Figure 3.6: Layout of the signage for LWC (adapted from Lohnes et al. 2001).

3.5.6 Driver Preference and Comprehension Survey

Balke et al. (2011) provided focus group preferences and driver comprehension survey results for static and active warning signs, and for devices associated with LWCs and flood-prone roadways. Focus group participants were Texas drivers selected from the general public. Participants indicated that they would decide whether to cross a water-covered road based on criteria such as the depth and speed/movement of water, road condition, and their own familiarity with the road, as well as personal factors such as the type of vehicle they were driving. Most participants wanted to know the depth of water over the road and whether the road is closed or too dangerous to proceed (Balke et al. 2011). Participants had difficulty estimating the depth of water seen in photographs of flooded roads, and their definitions of the depth of water considered potentially dangerous varied widely. They wanted advance warning of flood sites and detour information to avoid last-minute turnarounds, at least at the intersection that precedes the site. On average, just over half of participants stated that they would not continue on a visibly dry road if an active sign was on, citing concern for an unseen flood farther ahead (Balke et al. 2011). On the other hand, static warning signs did not significantly impact the decisions of participants to continue along a road; participants stated that they did not tend to pay much attention to permanent warning signs. However, the presence of a static warning sign just before a flood-prone site was correlated with an increase in participants' estimates of risk. Additionally, participants wanted to see a date stamp or some other indicator of when a sign was posted for instances when the road was temporarily closed (Balke et al. 2011).

Drivers wanted positive guidance about whether they should attempt to cross water over a roadway. Flood gauges that are easy to see and read, signs with directive messages, and active elements such as flashing lights were among the devices suggested most often during focus groups (Balke et al. 2011). Common flood gauge colors are white, yellow, and red (Figure 3.7). At low water levels, participants were significantly less likely to make a crossing when a standard white or yellow flood gauge was used than when a gauge with red and white threshold levels was used. Participants in driver comprehension surveys strongly preferred the color-coded red/white flood gauge over the single-color white or yellow gauges and understood the implied "white is safe, red is dangerous" message. However, the arbitrary "safe" level designated by the gauge overrode the actual water depth as a cue that several of the participants would otherwise have used to make decisions on crossing; the authors believed this response to the color-coded gauge was a reason to be cautious about using it (Balke et al. 2011).

The Texas focus group participants were concerned with the credibility of warning signs leading up to LWCs. They were aware of possible time lags related to placing, activating, and deactivating signs, both at the onset of a flooding hazard and afterward (Balke et al. 2011). They felt signs should be used to let the driver know that water is present on the road ahead, and predominantly convey messages of "caution" or "slow" and "water." In terms of color choice, participants felt that the conspicuity of standard yellow warning signs may not be sufficient, particularly at night and in bad weather, to convey the appropriate level of warning at these crossings (Balke et al. 2011). They preferred permanent signs with flashing lights that activate to let a driver know when water levels are dangerous, though the particular color of a warning light was shown not to matter for driver comprehension (Balke et al. 2011). The devices that participants felt gave the most useful and credible information and served as the best deterrents were dynamic message signs (DMS),

permanent warning signs with automatically activated flashing lights, and gates or barricades with flashing lights that extend across an entire road.

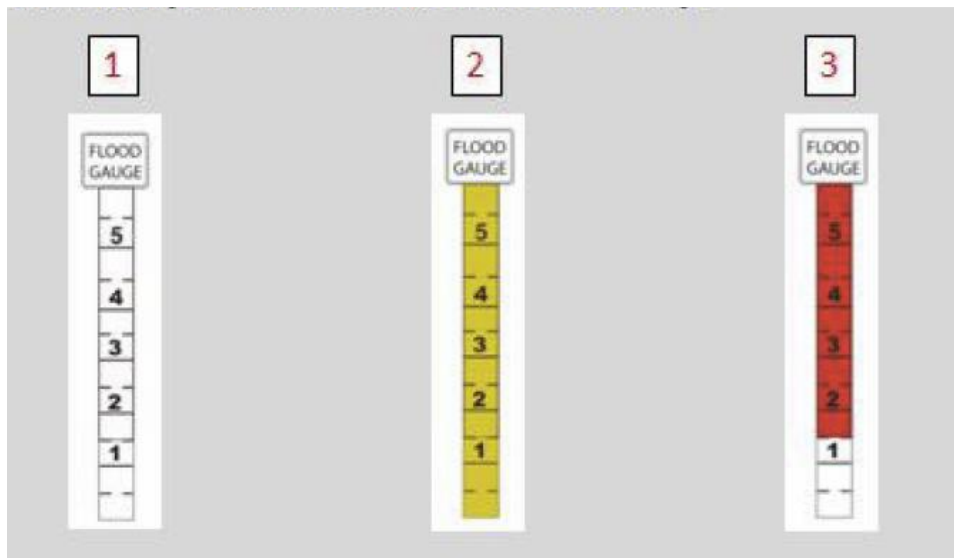


Figure 3.7: Common colors of flood gauges (adapted from Balke et al. 2011).

Active signs had a significant effect on participant responses across all sign messages, types, and beacon colors. At high water levels, most participants (approximately 82%) opted not to cross the stream even when the active signs were powered off. This is an important result because it indicates that even without the added cue from a lit sign, the participants used other cues from the roadway and the flood gauge to make their decision about whether to continue on the road. When active signs were powered on, however, the number of participants who would not cross rose significantly, to 96%. At low water levels, an average of 25% of participants opted not to continue on the road when an active sign was off. When signs were on, this percentage rose significantly, to 61%. Figure 3.8 shows a bar chart that summarizes these findings.

The specific text of a sign and gauge did not significantly alter drivers' likelihood to make a crossing at either low or high water levels. At high water levels, there were no significant differences among participant responses between four different active sign examples, with the exception of beliefs about the likelihood of getting a traffic citation for continuing past an active sign that was powered on (Balke et al. 2011). More participants believed that continuing past a lit *DO NOT ENTER* sign could result in a traffic ticket. Similarly, the presence of a limit line did not significantly alter participants' perceptions of the roadway or their decisions about whether to proceed (Balke et al. 2011). Of various advance warning signs presented during a driver comprehension survey, *ROAD MAY FLOOD* was preferred by a significant majority of participants (Balke et al. 2011). Though it did not make a significant difference in decision making, participants preferred the text *HIGH WATER/ROAD CLOSED WHEN FLASHING* on active warning signs at crossing sites. This is consistent with the focus group discussions, in which participants stated a preference for specific warnings about possible flooding on the roadway. Table 3.3 presents the preferences for various sign texts.

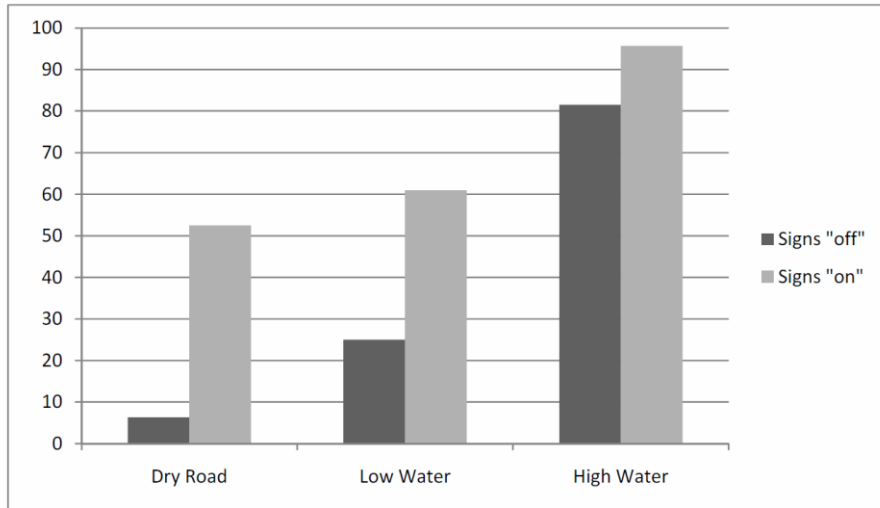


Figure 3.8: Percentage of survey participants who would turn around at an LWC depending on road condition and whether an active warning sign is lit. Lit signs are significantly more effective at deterring drivers (adapted from Balke et al. 2011).

Table 3.3: Preferred Text of LWC Warning Signs (modified from Balke et al. 2011)

Sign	Percent Preferred
<i>ROAD FLOODED WHEN FLASHING</i>	9.0
<i>HIGH WATER/DO NOT ENTER WHEN FLASHING</i>	30.5
<i>HIGH WATER/ROAD CLOSED WHEN FLASHING</i>	35.0
<i>DO NOT ENTER (LED)</i>	25.5
Sample Size	200

3.6 LWC ECONOMICS

The design and installation of LWCs should always be accompanied by an economic evaluation. Although the cost associated with LWC construction is usually much smaller than of culvert or bridge construction, LWC construction in some cases, such as in unstable or frequently flooded sites, can be costly. Similarly, a LWC will be overtopped by higher flows several times during its lifetime. Thus, a thorough economic analysis should be performed that encompasses construction cost, maintenance cost, and damage to property and lives associated with the flooding conditions. The LWC selection process should minimize the total annual cost of the installation over the life of the structure. Selecting a material that does not degrade or corrode easily may be associated more with high initial cost, but the total annual cost may be less over its longer service life due to low maintenance requirements.

3.6.1 LWC Cost

LWCs are economical alternatives to culverts and bridges used to convey water through a roadway, by allowing minor traffic interruptions at the time of roadway flooding. The major costs associated with the construction of LWC are cost of material, embankment, protection structures, and installation of signage. The costs of materials used in LWC construction vary with the region where it is constructed.

The cost of construction of a low-water bridge may lie between \$40,000 and \$ 50,000, whereas a vented ford can be constructed for as low as \$15,000 to \$20,000 (Lohnes et al. 2001).

The estimated cost of low-water crossings per current practices and experiences of IDOT’s Bureau of Bridges and Structures are given in Table 3.4.

Table 3.4: Cost of Different Types of LWCs

LWC Type	Description	Cost/ft ²	Total Cost
Unvented ford	Asphalt pavement	\$10	\$10/ft ²
Vented ford	24 in. concrete pipe culverts or precast concrete box culvert	\$55	\$ 65/ft ²
	Asphalt pavement	\$10	
Low-water bridge	Deck beam bridge	\$120	\$120–160/ft ²
	Slab bridge	\$160	

The economic analysis of LWC material selection requires site-specific considerations. Structural strength, channel slope, permissible exit velocities, and other factors govern the materials used. Riprap is needed in areas associated with high erosion. Cement slab, rocks, etc. may be used to construct hardened unvented LWCs. In areas with acidic drainage, corrosion of metals may limit the use of corrugated metal pipes.

In the case of vented fords, the shape also plays an important part in cost evaluation along with the material. Circular pipes are generally found everywhere, are reasonably priced, can withstand high

structural loads, and are hydraulically efficient. However, pipes with low VAR are not recommended. Arches and ellipses are more expensive than circular pipes, and they require additional attention to their foundations. It is recommended to compare the costs for pipes of equal hydraulic capacity to help guide the material selection.

Box culverts provide flexibility in rise-to-span ratios by using multiple cells. Precast box culverts take less time to install compared to cast-in-place installations, but they may be associated with additional handling cost.

3.6.2 Service Life

The service life of LWCs should be considered when selecting the appropriate type of LWC. If the LWC is located where replacement would be impractical, a LWC with a longer service life is desired, and the LWC and materials should be selected accordingly. In areas where changes are expected in traffic patterns or if the roads will be rebuilt in a relatively short time, a LWC with a shorter service life can be an option.

3.6.3 Risk Analysis

Traditionally, LWCs have been designed based on the importance and use of the roadway being served, with relatively little attention given to other economic and site factors. Risk analysis is necessary for LWC installations on major roads or those located in areas with high potential flood damage. The objective of the risk analysis is to find the best LWC capacity and design based on a comparison of costs and benefits.

LWC construction represents a flood plain encroachment, with the associated flood risks and initial construction costs. HEC-17, *Highways in the River Environment—Floodplains, Extreme Events, Risk, and Resilience* (2nd edition) should be consulted when performing risk analysis in the LWC design so that the design is associated with the least total expected cost (LTEC) (Kilgore et al. 2016).

During the risk analysis, alternative designs should be considered. Engineering, legislative, and policy constraints may limit the range of alternatives (Schall et al. 2012). Some of the constraints are the following:

- Prescribed minimum design flood criteria
- Limitations imposed by roadway geometrics such as maximum or minimum grade lines, site distance, and vertical curvature
- Flood plain ordinances or other legislative mandates limiting backwater or encroachment on the floodplain
- Channel stability considerations that would limit culvert velocity or the amount of constriction

CHAPTER 4: LWC DESIGN GUIDELINES

4.1 GENERAL DESIGN GUIDELINES

LWCs are generally appropriate at stable sites where the proposed design matches with channel geometry, bed material, and flow characteristics and conforms to safety and access requirements. Construction of LWCs should be done such that the stream faces as minimum disturbance as possible.

Some of the important design considerations based on previous studies (Lohnes et al. 2001; Motayed et al. 1982) are listed in Table 4.1.

Table 4.1: Design Considerations for LWCs

Considerations	Criteria
Channel cross section	Should not be altered
Overtopping flow depth	Less than or equal to 6 in.
Vertical curve at dip (approach grades)	Less than 10%
Stream bank height	Less than 12 ft
Orientation of structure	Straight, avoid skew
Approach distance	750 ft minimum sight distance for warning signs
Height of crossing above streambed	Less than 4 ft
Erosion from flows overtopping crossing	Elevation difference between crossing and streambed kept to minimum. LWC surface material extended in both directions away from structure. Downstream slope 4:1 or milder
Core material protection	Provide cutoff walls and sidewalls
Stream bank protection	Establish vegetation

Figure 4.1 is a flowchart showing LWC selection, design and construction steps, and important factors to consider in the process. The first step is data collection and deciding whether a LWC is a suitable option for the site. Section 3.3 of this report, on the suitability of LWCs should be consulted, and the traffic, safety requirements of the crossing, and judgment of the engineer should be taken into consideration. A suitable LWC can then be selected for the site considering stream type, flow, economic consideration, etc. Section 3.4 of this report provides general guidelines on this process.

The next step consists of LWC design and construction, which are covered in this chapter. Design examples for LWCs can be found in Appendix H. Permitting requirements for LWC construction are discussed in Appendix F. LWC inventory and inspection criteria are provided in Appendix I.

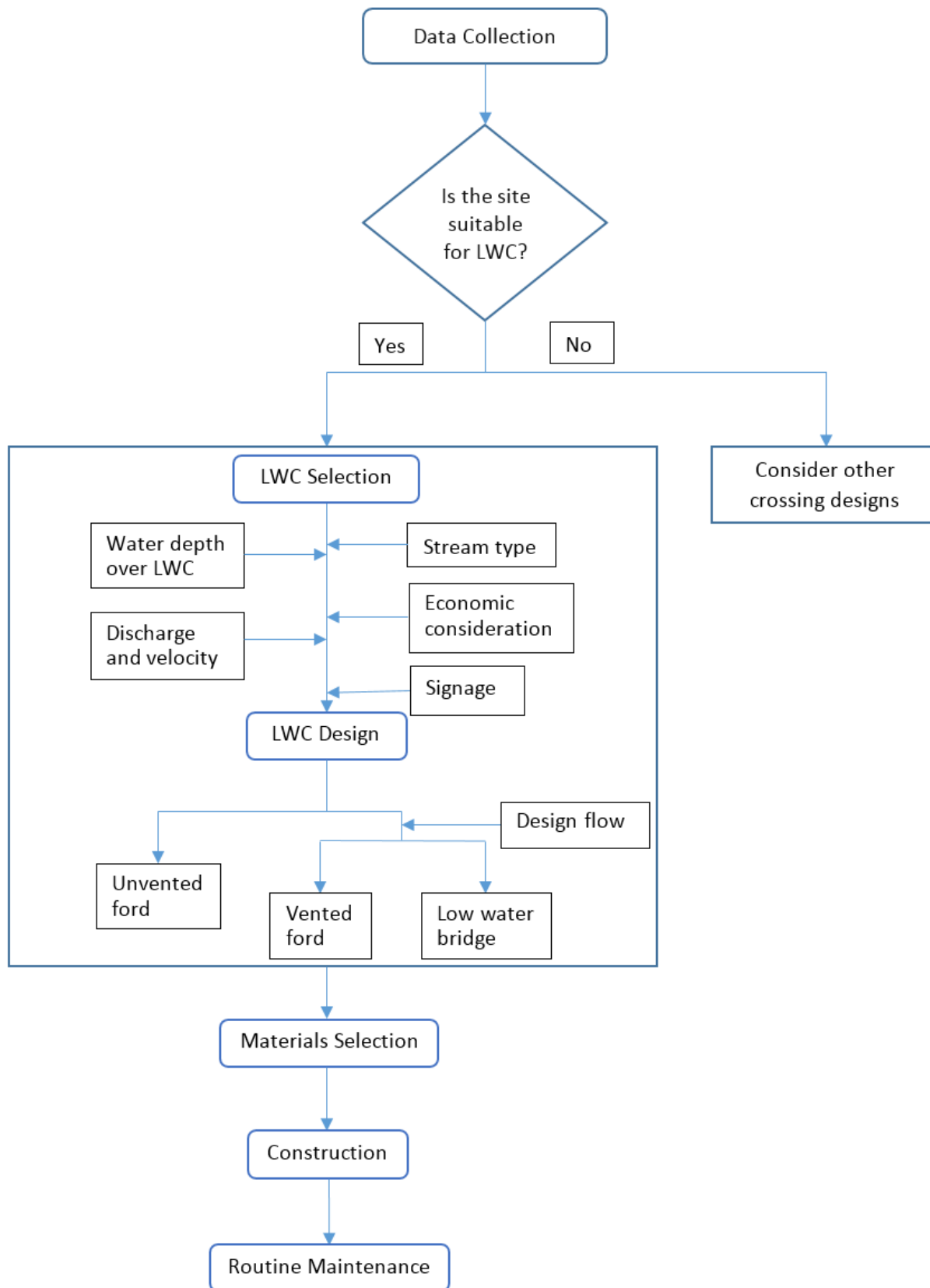


Figure 4.1: Flowchart for LWC selection, design, and construction.

4.2 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 design of the fords:

- 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.

4.2.1 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 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 (mi²)

i and a_1 = coefficients

On solving, we get

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

The equation can be simplified as

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

$$\text{where, } b = 10^i$$

Intercepts (i) and coefficients (a_1) for different regions and the corresponding flow-duration-area curves are provided in Figures 4.2 through 4.5 and Tables 4.2 through 4.4.

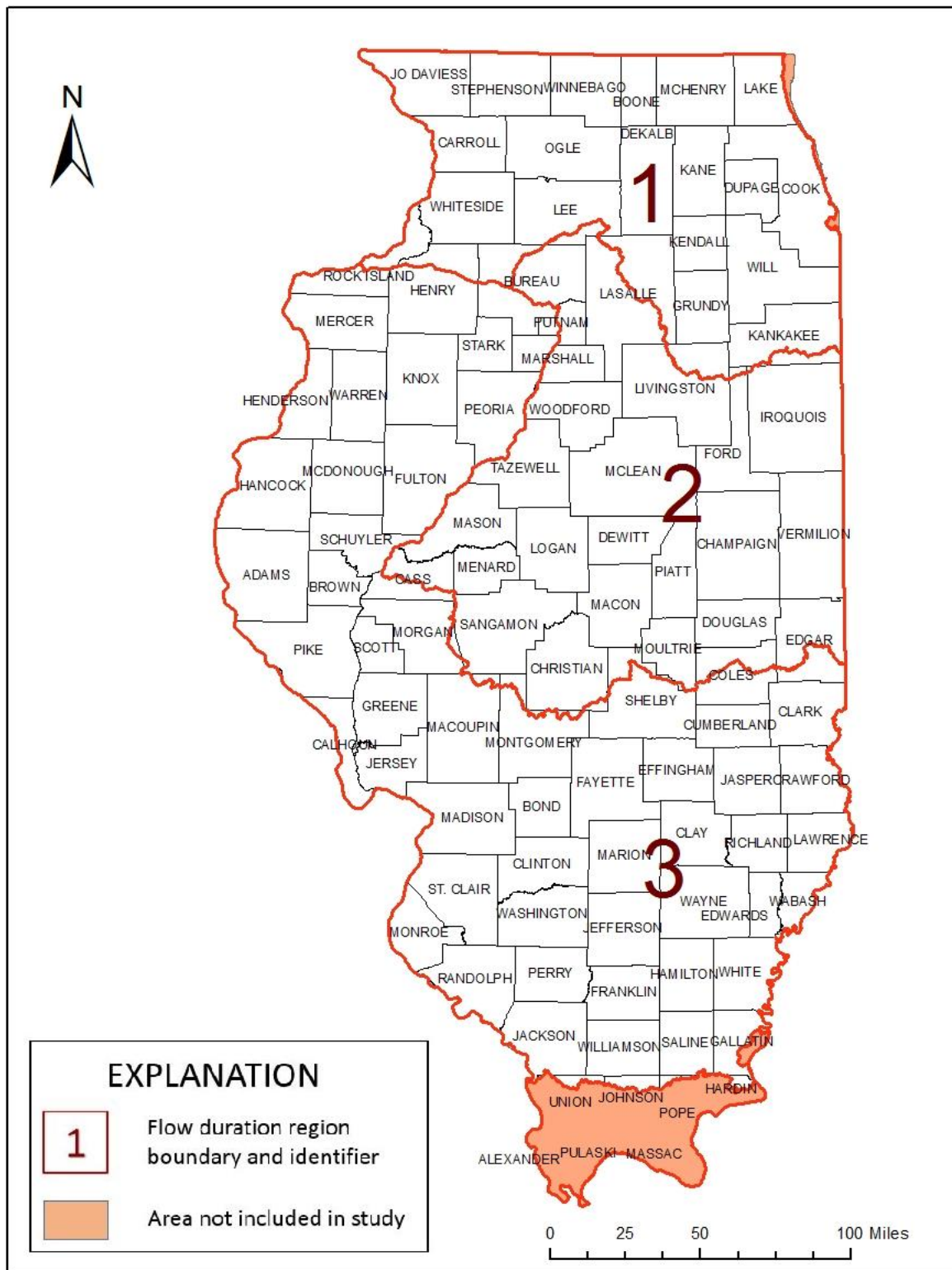


Figure 4.2: Regions in Illinois used to obtain FDC (reproduced after Over et. al 2014).

Region 1

Table 4.2: 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

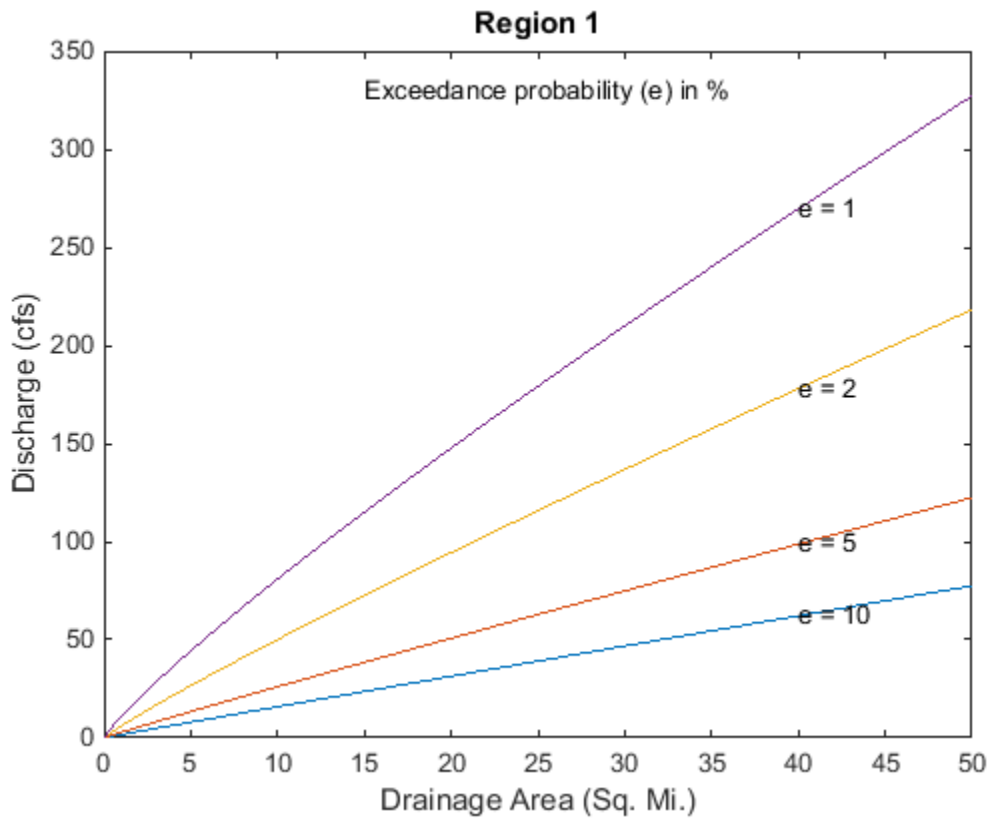


Figure 4.3: FDC for different exceedance probabilities for Region 1 in Illinois.

Region 2

Table 4.3: 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

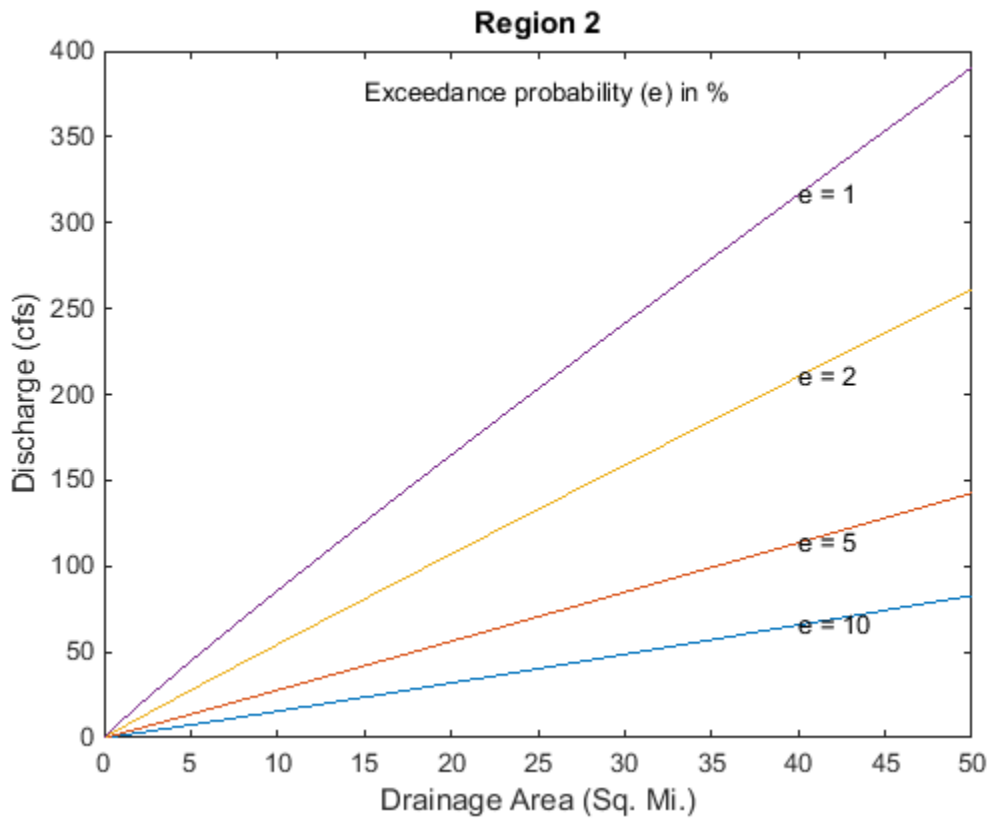


Figure 4.4: FDC for different exceedance probabilities for Region 2 in Illinois.

Region 3

Table 4.4: 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

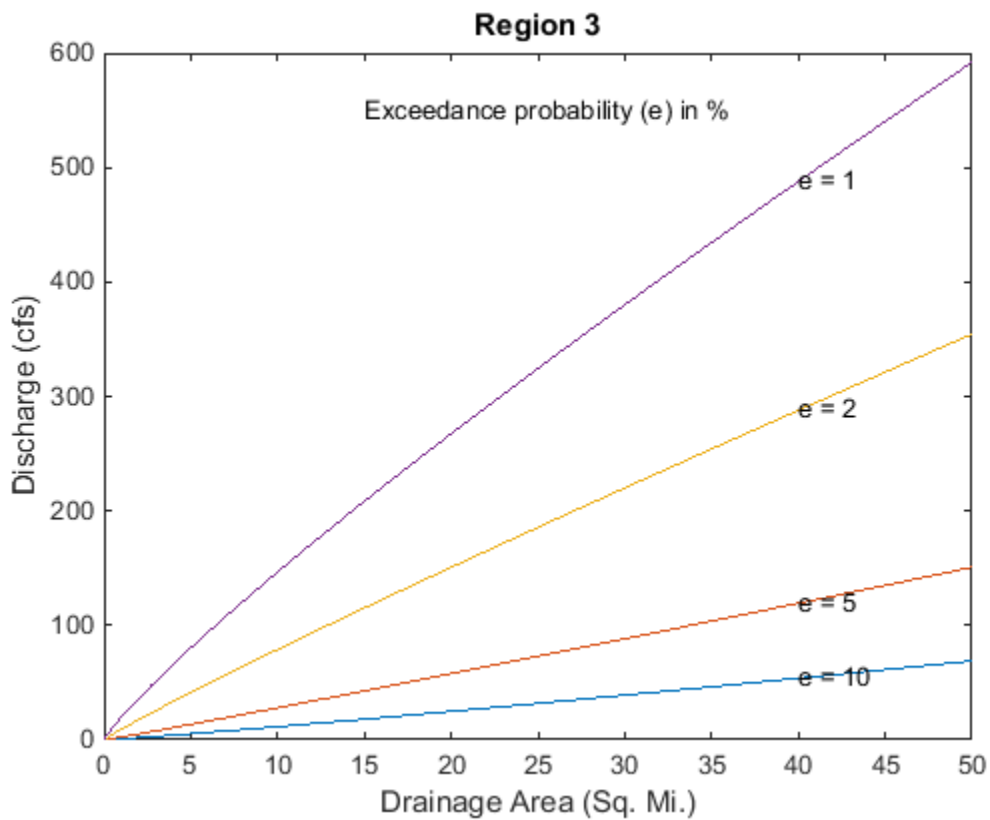


Figure 4.5: FDC for different exceedance probabilities for Region 3 in Illinois.

4.2.2 Flood-Frequency Approach

The flood-frequency approach is a relatively simple method of hydrologic design. Crossing components are usually designed for 10- or 25-year flow, whereas the pipe in a vented ford is designed to pass a 0.5- or 1-year event. HEC-HMS can be employed as an analytical tool to derive a flow hydrograph for the study area. For gauged streams, the design discharge can also be computed by flood-frequency analysis of the yearly maximum discharge values over a long range.

It is recommended that Illinois StreamStats be used for ungauged catchments in Illinois (USGS 2016). It provides information on peak discharges (Q2, Q5, Q10, Q25, Q50, Q100, and Q500), drainage area, stream slope, and average soil permeability among many other statistics. StreamStats is a web-based GIS application created by USGS in cooperation with Environmental Systems Research Institute (ESRI). It estimates streamflow statistics for ungauged sites using regional regression equations (Ries et al. 2008).

More information on the method used by StreamStats can be found in the USGS report, *Estimating Flood-Peak Discharge Magnitudes and Frequencies for Rural Streams in Illinois* (Soong et al. 2004). The study divides Illinois into seven hydrologic regions (Figure 4.6) and uses the following equations:

$$Q_T = a(TDA)^b(MCS)^c(PermAvg)^dRF(N) \quad \text{[for hydrologic regions 1, 3 and 5]}$$

$$Q_T = a(TDA)^b(MCS)^c(\%Water + 5)^dRF(N) \quad \text{[for hydrologic regions 2, 6 and 7]}$$

$$Q_T = a(TDA)^b(MCS)^c(BL)^d \quad \text{[for hydrologic region 4]}$$

where

Q_T = estimated flood quantile, in ft³ / sec (cfs), for the designated recurrence interval T years

a, b, c, d = coefficients and exponents of the equations for the variables TDA, MSC, PermAvg, BL, and (%Water + 5), respectively

TDA = total drainage area, in mi²

MCS = main channel slope, in ft/mi

PermAvg = averaged permeability of the watershed, in inches per hour

BL = basin length, in miles

(%Water + 5) = calculated percentage of open water and herbaceous wetland plus a constant 5% (to avoid zero values)

RF(N) = regional factor for hydrologic region N

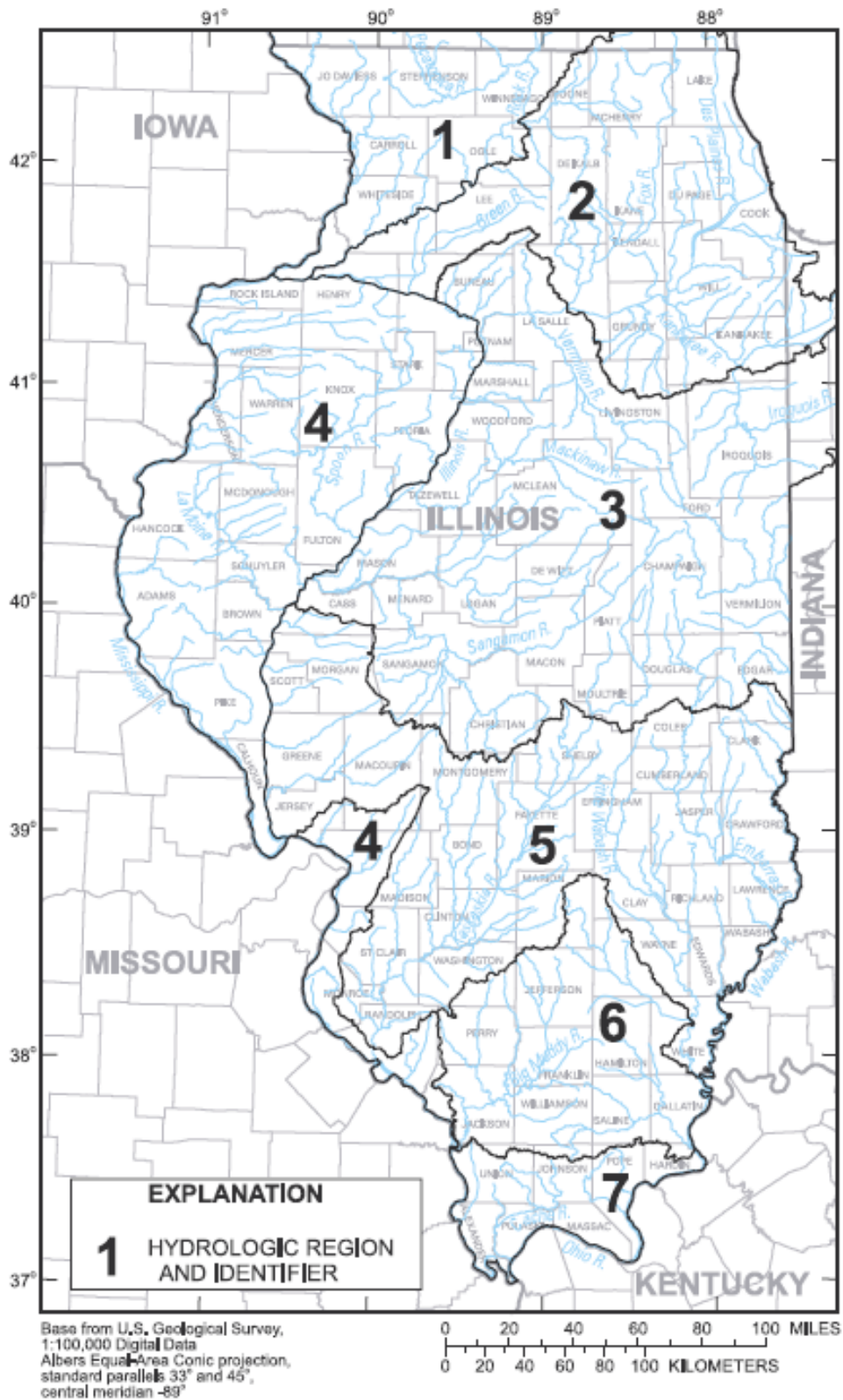


Figure 4.6: Hydrologic regions for flood-frequency analysis (adapted from Soong et. al 2004).

4.2.3 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 year, 1.01 year, 1.5 years, etc.

More information on PDS regional equations can be found on the USGS report, Estimating Flood-Peak Discharge Magnitudes and Frequencies for Rural Streams in Illinois (Soong et al. 2004). The same seven hydrologic regions (Figure 4.6) are used for the PDS regional equations as well. The regional equations are as follows:

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

$$Q_T = a(TDA)^b (BL)^c (PermAvg)^d \quad [\text{Region 2}]$$

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

$$Q_T = a(TDA)^b (MCS)^c (BL)^d \quad [\text{Region 4}]$$

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

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).

4.3 HYDRAULICS

Hydraulic analysis tools used for LWC design depend on the type of LWC to be designed. Manning's equation is often used to design fords on channel bottoms while raised fords are treated as broad-crested weirs in order to determine flow depth, flow velocity, etc. for a given design flow (Lohnes et al. 2001). Vented fords are typically analyzed as culvert structures with weir flow over the road when the water overtops the structure (Schall et al. 2012). The hydraulic analysis and design of low-water bridges is done exactly the same way for normal bridges, with special consideration given to overtopping flows (Zevenbergen et al. 2012). Because scour at and near LWCs can cause structural failure and safety concerns, as well as degrade water quality and habitat, LWC designs should incorporate consideration and design guidelines for scour (Arneson et al. 2012), stream stability (Lagasse et al. 2012), and appropriate scour countermeasures (Lagasse et al. 2009).

Various hydraulic models are available which can be used for modeling bridges and LWCs. 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 common. 3D models are used to analyze complex flow fields.

1D models are appropriate for in-channel flows and when floodplain flows are minor. They are also frequently applied to small streams and for extreme flood conditions. 1D models generally provide accurate results for narrow to moderate floodplain widths. They can also be used for wide floodplains when the degree of bridge constriction is small and the floodplain vegetation is not highly variable. In general, where lateral velocities are small, 1D provide reasonable results. Leahy (2014) used HEC-RAS to model LWCs at a U.S. Army installation in Indiana.

2D models should be used when flow patterns are complex and 1D model assumptions are significantly violated. However, data requirements for 2- models are much higher. It is recommended to use HEC-RAS for the hydraulic analysis of low-water crossings and look for other alternatives when HEC-RAS analysis is not adequate.

4.4 DESIGN OF UNVENTED FORDS

Unvented fords are simple to design and are very effective in certain conditions such as ephemeral streams. Disturbance to the channel hydrology can be minimized. Fords on channel bottoms are good for sites with a stable streambed, or where minimum strengthening of bed is required. Raised unvented fords can be used where the depth of normal flow exceeds 6 in. or the approach roads are higher than the stream bed. General considerations for unvented fords are provided in Table 4.5.

Table 4.5: General Design Considerations for Unvented Fords (from Lohnes et al. 2001)

Considerations	Criteria
Precast concrete panels	Should include a proper filter to prevent piping and consequent undermining of the crossing
Erosion protection	End walls and/or gabion protection may be desirable; wide, sloped shoulders downstream may be helpful
Markers	Provide markers to help drivers spot the limits of the roadway when flooded

4.4.1 Ford on the Channel Bottom

For fords constructed on a channel bottom, there is no hydraulic control (i.e., no energy loss); therefore, the discharge and velocity can be estimated using Manning's equation.

$$v = \frac{1.486}{n} R^{\frac{2}{3}} S^{\frac{1}{2}}$$

where v is velocity in ft/s, R is hydraulic radius in ft, n is the Manning's coefficient (dimensionless), and S is slope in ft/ft.

From the continuity equation, $Q = A * v$,

where Q is the flow in ft³/s, A is the area of flow in ft² and v is velocity in ft/s.

Assuming a rectangular channel with the channel width w and depth of flow h associated with design discharge (Q_d , cfs),

$$\text{Hydraulic radius (R)} = \frac{\text{Area of flow (A)}}{\text{Wetted Perimeter (P)}} = \frac{wh}{w+2h}$$

Combining the above equations results in

$$Q_d = \frac{1.486}{n} \frac{(wh)^{\frac{5}{3}}}{(w+2h)^{\frac{2}{3}}} S^{\frac{1}{2}}$$

For a given channel of width (w), slope (S) and design discharge (Q_d), the depth of flow (h) can be determined by solving the above equation.

For wide channels with $\frac{w}{h} \geq 10$, the above equation can be simplified as

$$Q_d = \frac{1.486}{n} w h^{\frac{5}{3}} S^{\frac{1}{2}}$$

Thus, the flow depth h can be found using

$$h = \left(\frac{nQ_d}{1.486wS^{\frac{1}{2}}} \right)^{\frac{3}{5}}$$

The computed value of h should be less than the allowable maximum flow depth of 6 in. for the ford on the channel bottom to be a viable option at the LWC site.

4.4.2 Raised Unvented Ford

The depth of flow (h) associated with design discharge (Q_d) in the case of a raised unvented ford can be determined using the broad-crested weir formula, which has the general form

$$Q_d = CLH^{\frac{3}{2}}$$

where Q_d is in cfs, C is the weir coefficient (dimensionless), L is the length of LWC normal to flow in feet, and H is the upstream head of water above the level of ford, as shown in Figure 4.7.

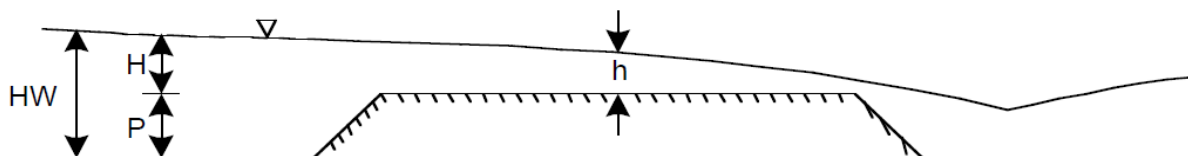


Figure 4.7: Crossing profile of a raised unvented ford (from Lohnes et al. 2001).

To find the depth of flow, the empirical equation developed by Rossmiller et al. (1983) and used by Lohnes et al. (2001) to design low-water crossings in Iowa can be employed:

$$H = 0.389 Q_d^{0.599} L^{-0.493}$$

and

$$h = 0.6H$$

where h is the depth of water above the level of ford at the center of the roadway.

Combining above two equations results in

$$h = 0.233 Q_d^{0.599} L^{-0.493}$$

According to Rossmiller et al. (1983), the height of the raised ford above the streambed (P) is a flexible design parameter that has very little impact on the discharge-depth relation. However, it should not be so high that the movement of fish and other aquatic organism is affected. Hence, the recommended range is between 2 and 4 ft (Lohnes et al. 2001).

4.5 DESIGN OF VENTED FORD

In vented fords, vehicles do not go into the water when the structure has normal flow; hence, the tires are dry and free of soils from the stream. There is less problem with erosion of approach roads compared to unvented fords. The design of vented fords is simple as well, and smaller vented fords can be built as inexpensively as unvented fords. General design considerations for vented fords are given in Table 4.6.

Table 4.6: General Design Considerations for Vented Fords (from Lohnes et al. 2001)

Considerations	Criteria
Depth of cover above pipes	Minimum 1 ft recommended
Exit velocity of pipes (vents)	Limit exit velocity of the flow not to exceed 10 ft/s
Pipes	Pipes should be anchored in the ground; both ends should be beveled or mitered to reduce debris accumulation Minimum size 1 ft diameter
Guard rails	Guard rails are not recommended so as to avoid catching debris and floating materials during a flood
High streamflow	Road surface is raised above streambed to accommodate the flow
Streambed erosion protection	Riprap should be placed upstream and downstream to reduce the scour in erodible channel

Vented fords are designed in a manner similar to a culvert. In the case of a vented ford, a portion of the design flow passes above the ford (Q_{top}), while the remaining part of the flow is designed to pass through the vent or pipes (Q_v). Thus, the total design flow (Q_d) is as follows and as shown in Figure 4.8:

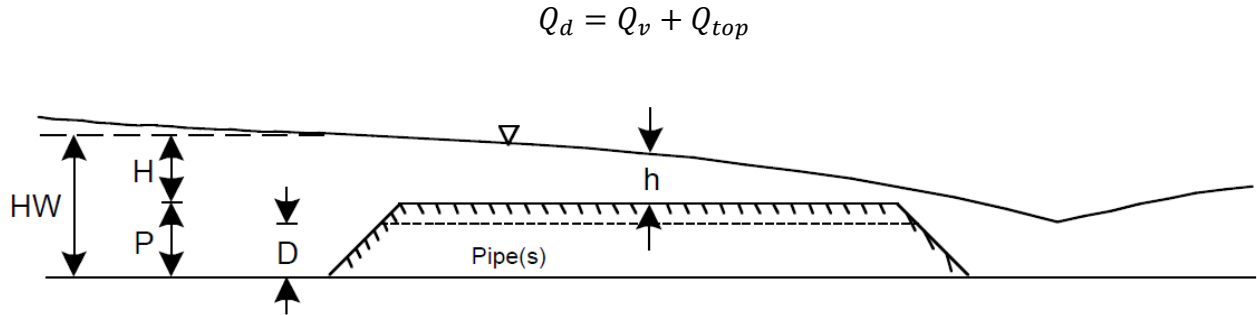


Figure 4.8: Crossing profile of a vented ford (from Lohnes et al. 2001).

The flow over the top of the vented ford is limited to 6 in. in depth, similar to the unvented ford. This flow can be estimated using the weir equation used for the raised unvented ford:

$$H = 0.389 Q_{top}^{0.599} L^{-0.493}$$

Rearranging above equation,

$$Q_{top} = 4.83 L^{0.823} H^{1.67}$$

Using the maximum allowable depth of water (h) of 0.5 ft and considering $H = h/0.6$, we calculate H as 0.833 ft, and we can obtain the flow overtopping the ford by substituting it in above equation as

$$Q_{top} = 3.562 L^{0.823}$$

Now the capacity of pipes in vented ford is given by

$$Q_v = Q_d - Q_{top}$$

After determining the discharge through the vent, the size of the pipe may be found by following the methods used for culverts, among which culvert hydraulics and flow equations (Jones et al. 2006; Normann et al. 1985) and HEC-5 charts are recommended (Herr and Bossy 1965; Normann et al. 1985). In addition, the FEMA-approved computer program CulvertMaster (Bentley 2016) can be used for solving culvert hydraulics problems.

4.5.1 Culvert Flow Equation

Because a larger-sized pipe is required for a culvert that is operating under inlet control as compared to the outlet control, sizing of pipe in a vented ford is based on the inlet control conditions. The flow equation for submerged entrance under inlet control is

$$\frac{HW}{D} = c \left[\frac{Q_v}{AD^{0.5}} \right]^2 + Y + f_s S$$

where HW is headwater depth (ft), D is culvert size (ft), Q_v is the design discharge (cfs), A is full cross-sectional area of the culvert barrel (ft²), Y and c are inlet regression constants, S is culvert barrel slope, and f_s is slope correction factor. Lohnes et al. (2001) analyzed the sensitivity of the slope (S) to the design size (D) and found that D is not sensitive for S less than 0.02.

Rearranging the above equation, we get

$$Q_v = \frac{AD^{0.5}}{c^{0.5}} \left(\frac{HW}{D} - Y - f_s S \right)^{0.5}$$

A flow equation for a vented ford with corrugated metal pipes (CMP) with 0.5 ft overtopping flow depth, 1 ft cover above the pipe, and mitered inlet was used to develop a design curve (Figure 4.9). The headwater depth is equal to the sum of the diameter of pipe (D), overtopping flow depth at the entrance (0.83 ft) and pipe cover (1ft). The area of a circular pipe is $A = \frac{\pi D^2}{4}$, and corresponding values of constants Y , c , and f_s are 0.75, 0.0463, and 0.7, respectively, for CMP with a mitered entrance, which gives the equation:

$$Q_v = 3.65D^{2.5} \left(\frac{1.83}{D} + 0.25 - 0.7S_0 \right)^{0.5}$$

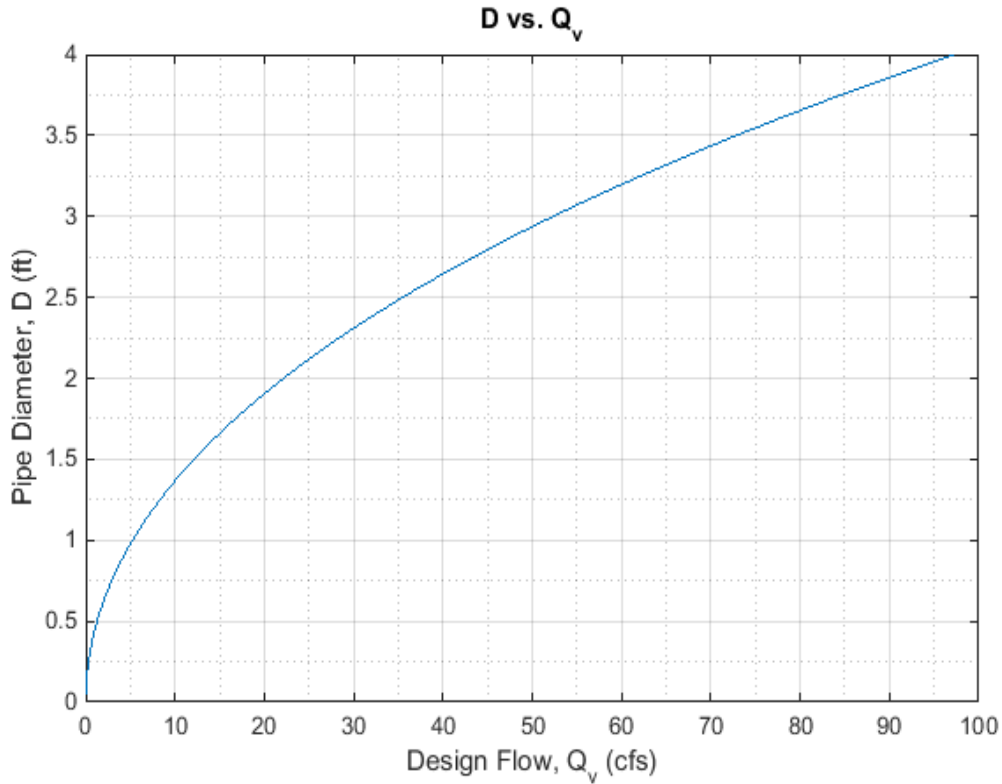


Figure 4.9: Design curve for vented fords using CMP with mitered entrance, under inlet control ($s \leq 0.02$).

At first, a single pipe is considered and its size is computed, with the help of the design curve, to pass the design flow (Q_v). If the size is larger than the height of LWC or the required pipe size is not available, multiple identical pipes should be used. The discharge through each pipe is equal to the total design discharge divided by the number of pipes.

Pipe exit flow velocity should be below 10 ft/s for scour control and channel protection.

Pipe exit velocity is given by $V_e = \frac{Q_v}{\frac{\pi D^2}{4}}$

Similarly, the design curve for a circular concrete pipe (Figure 4.10) with groove end projecting as an inlet configuration is computed using the constants Y , c , and f_s as 0.69, 0.0317, and -0.5 , respectively, which gives the equation:

$$Q_v = 4.41D^{2.5} \left(\frac{1.83}{D} + 0.31 + 0.5S_0 \right)^{0.5}$$

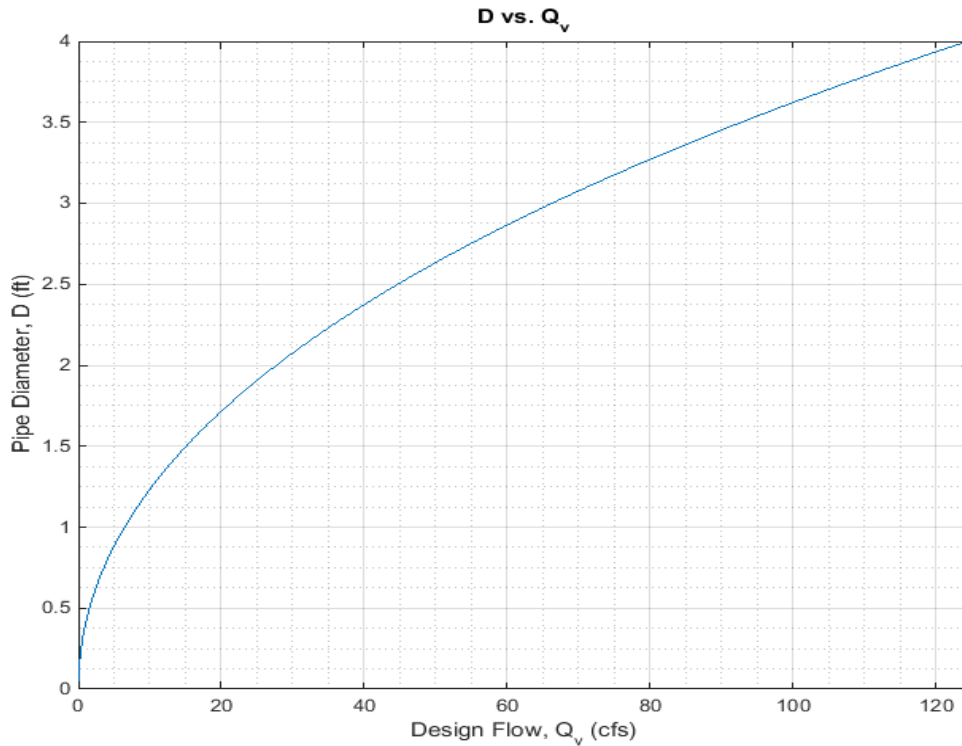


Figure 4.10: Design curve for vented fords using circular concrete pipes with groove end projecting, under inlet control ($s \leq 0.02$).

4.5.2 HEC-5 Charts

The process of finding the size of pipes in vented fords using HEC-5 charts is an iterative trial-and-error process. Several charts are provided in Herr and Bossy (1965) and Normann et al. (1985) for different inlet submergence, flow control, barrel shape and material, inlet treatment, etc. HEC-5 charts to determine pipe diameter for the given design flow for circular concrete pipes, corrugated metal pipes (CMP), and concrete box culverts are given in Figures 4.11 through 4.13.

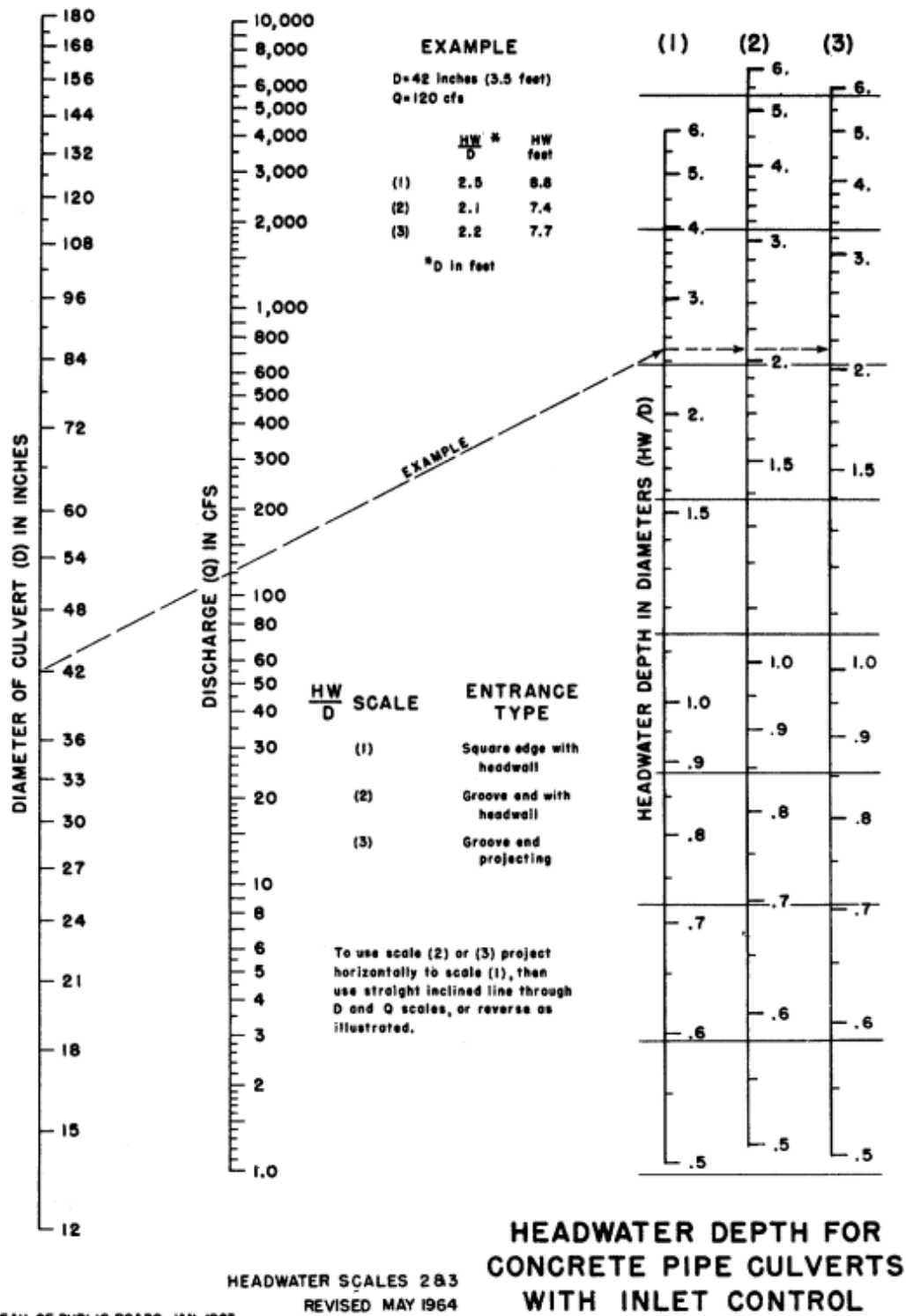
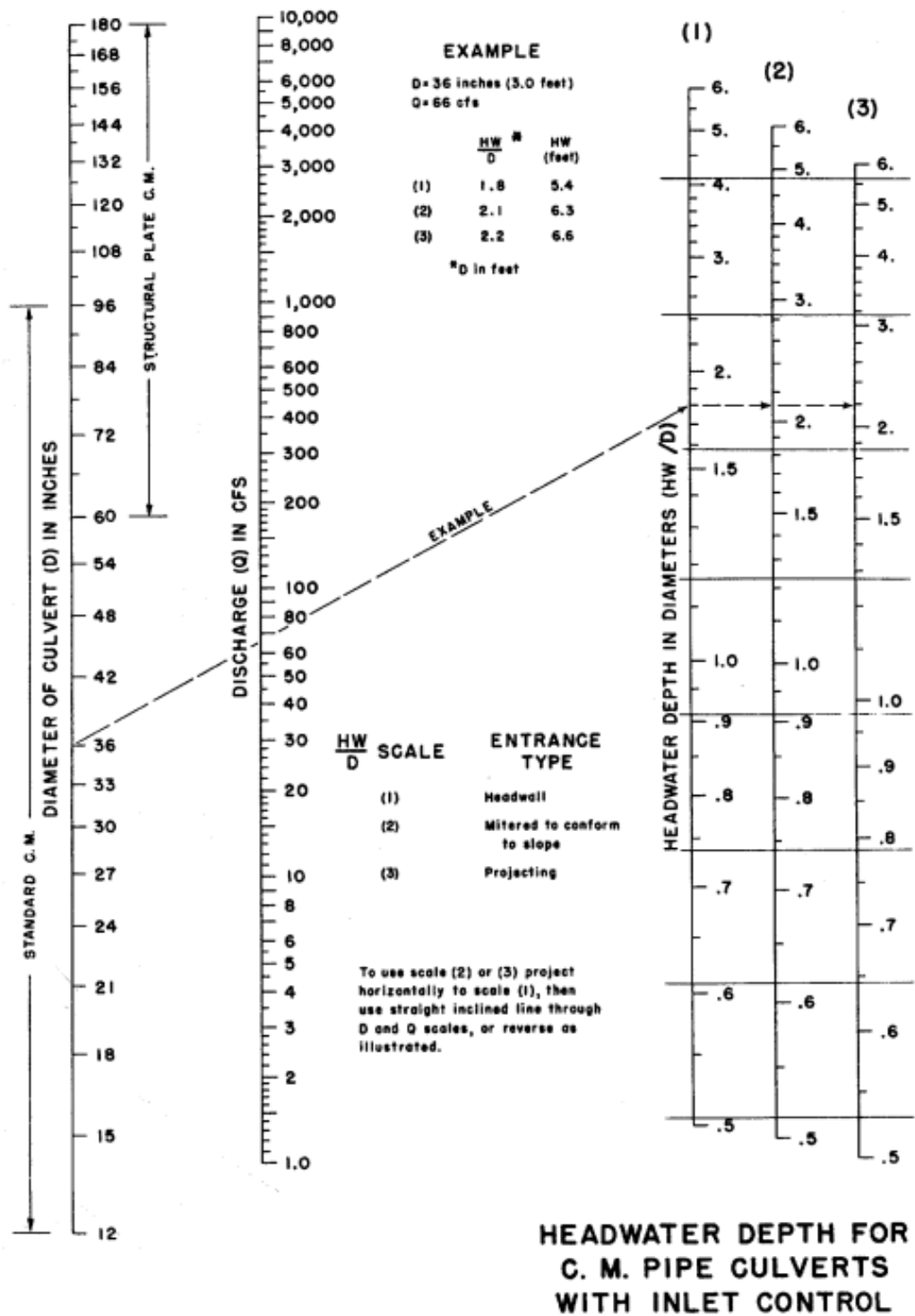
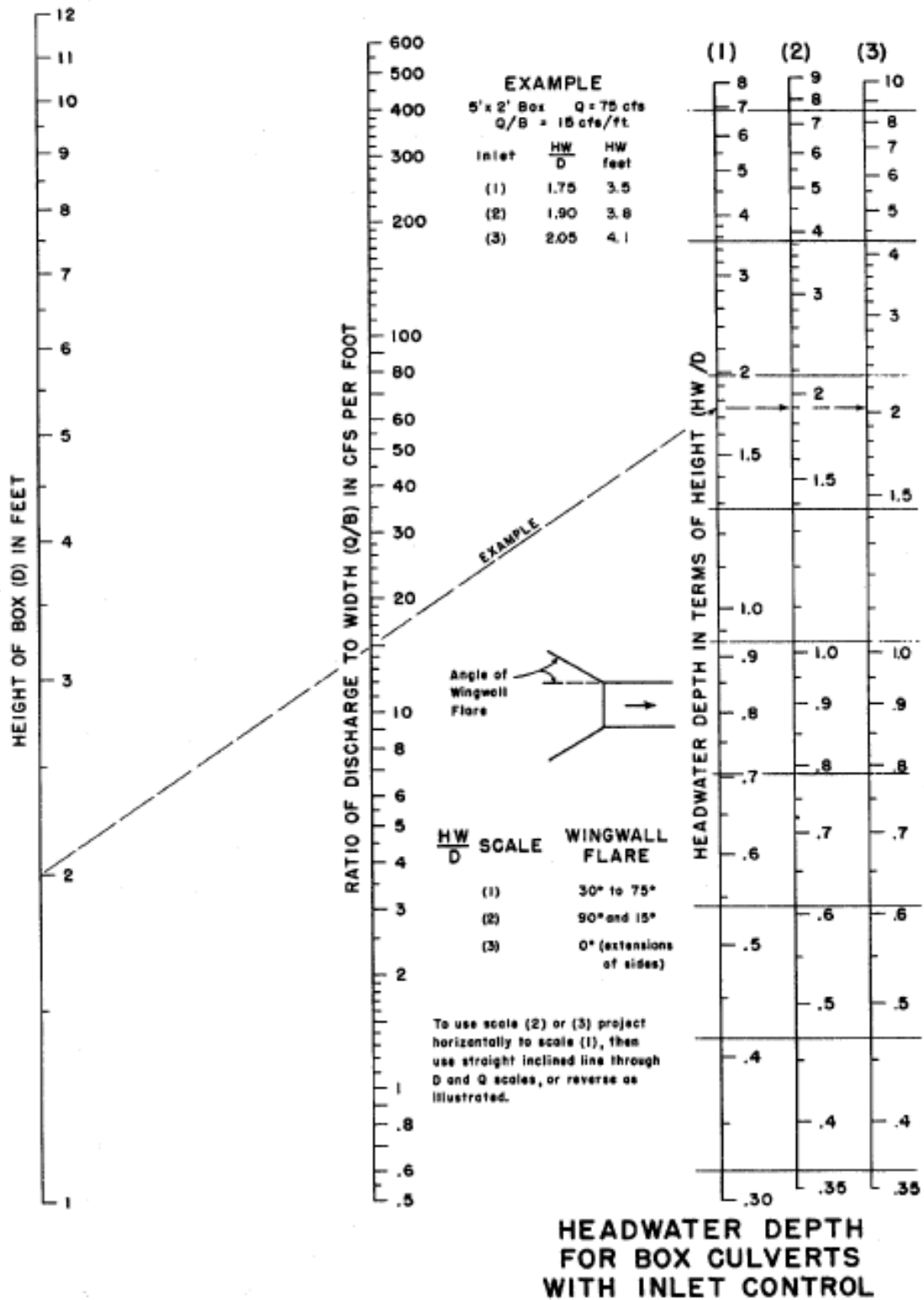


Figure 4.11: Headwater depth for concrete pipe culverts with inlet control (adapted from Normann et al. 1985).



BUREAU OF PUBLIC ROADS JAN. 1963

Figure 4.12: Headwater depth for CMP culverts with inlet control (adapted from Normann et al. 1985).



BUREAU OF PUBLIC ROADS JAN. 1963

Figure 4.13: Headwater depth for box culverts with inlet control (adapted from Normann et al. 1985).

4.6 DESIGN OF LOW-WATER BRIDGES

Low-water bridges can be used in areas with higher streamflow and traffic volume, or areas where AOP is of high importance. Because low-water bridges provide minimum channel disturbance, they can also be used in areas where channel modification is undesirable. General considerations for design of low-water bridges are given in Table 4.7.

Table 4.7: General Design Considerations for Low-Water Bridges (from Lohnes et al. 2001)

Considerations	Criteria
Surface	Decks must be heavy to withstand drag, uplift, and lateral forces due to overflow and upstream water; upstream and downstream edges should be rounded
Height of piers	To reduce the risk of overturning, the height of piers should be limited to approximately 10 ft

Two bridge designs, which are commonly used in low-volume roads, are provided: the slab bridge design and the precast concrete channel beam bridge design. The best design for any LWC site depends on the site factors as well as budget available. In cases where these designs do not seem to be the best option, other bridge design options can be followed. The designs should meet all the AASHTO requirements as given in the *Standard Specifications for Highway Bridges* (AASHTO 2002) and IDOT's *Standard Specifications for Road and Bridge Construction* (IDOT 2012a).

4.6.1 Slab Bridge

Slab bridges are short-span bridges that are frequently constructed in areas where shallow structure depths are required. They can function very well as low-water bridges. With slab bridges, the deck slab also serves as the main load-carrying component of the bridge. These bridges are designed for simple one-way bending, which is possible due to their span-to-width ratio (Figure 4.14).

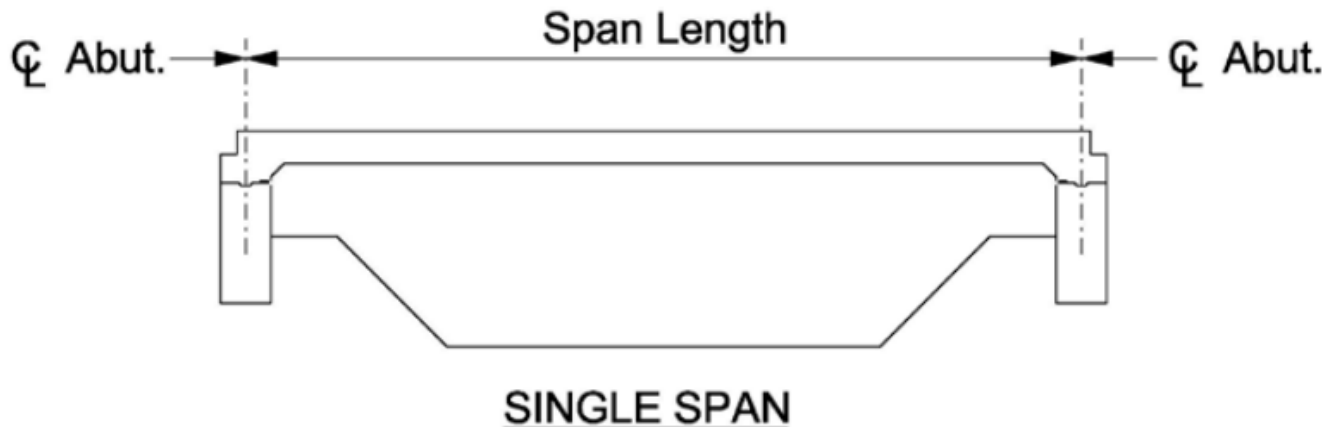


Figure 4.14: Single-span slab bridge (adapted from Wisconsin DOT 2015).

The slab bridge should be designed following either the LRFD or LFD design process. The design procedures are given in the *Illinois Bridge Manual* (IDOT 2012b). The main reinforcement in both of these processes is designed for strength, fatigue, cracking control, and limits of reinforcement. The slab thickness should be based on the design requirements.

The *Illinois Bridge Manual* limits the maximum span of the slab bridges to 40 ft, which is appropriate for LWCs. For longer length of spans, continuous slab design must be used (IDOT 2012b).

4.6.2 Precast Concrete Channel Beam Bridge

Precast concrete channel beams are a suitable option for short-span bridges, and they can be used as a LWC. These beams are commonly available for spans from 16 ft up to 40 ft in length. The beams in channel beam bridges are positioned and tied together, which also acts as the superstructure of the bridge. In these bridges, the stems of the channel resist flexural as well as shear forces, and the flange acts as bridge deck (Durham et al. 2003). Figure 4.15 shows a section of a channel beam bridge.

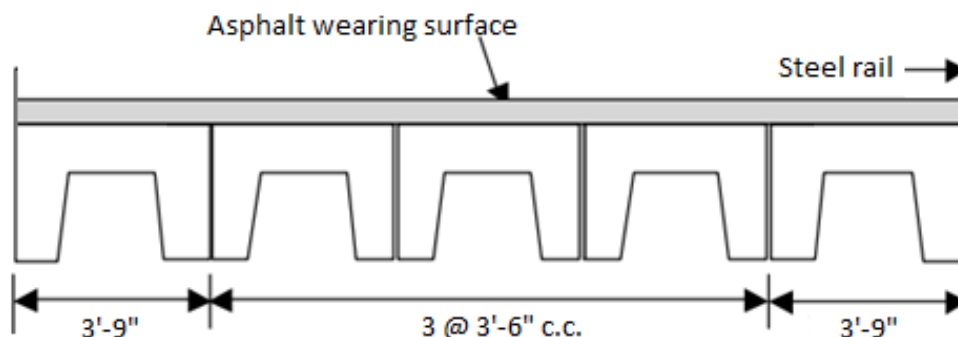


Figure 4.15: Cross section of a concrete channel beam bridge (modified from Durham et al. 2003).

The asphalt wearing surface is an optional element for LWCs, and it is often not installed on low-volume roads. Typically, side-mounted steel rails are installed at the ends. Precast concrete channel beam bridges are beneficial as LWCs due to reduced construction time, long life, and low maintenance cost.

IDOT currently has no standards for channel beam bridges. Concrete channel beam bridges should be designed following the AASHTO Standard Specifications.

4.7 MATERIALS SELECTION AND CONSTRUCTION

This section discusses the selection of materials used in the construction for the crossing surface, bank protection, and approach roads.

4.7.1 Unvented Fords

The selection of materials should be done carefully in the case of unvented fords because the crossing surface is constantly overtopped by water in the stream. Commonly used materials include compacted earth, Portland cement concrete, interlocking blocks, gravels, riprap, and gabions. The crossing surface should be stable and should not wash away due to the flow over it. Depending on channel stability, appropriate protection should be provided to the streambed using gravel, riprap, or geotextile materials. Sidewalls and bank erosion protection may be required, which can be achieved by the use of riprap and gabions.

4.7.1.1 Compacted Earth

Compacted earth is the cheapest alternative, and it can be used when the streambed is very stable. The soil used should be borrowed from the surrounding area.

4.7.1.2 Gravels

Gravel consists of loose rock fragments and is a very useful material in LWC construction in rural areas with very little traffic. The average size of the gravel to use depends on the flow in the stream and the flow velocity. It is preferred to use gravel with a length-to-width ratio of less than 3 (Lohnes et al. 2001).

4.7.1.3 Interlocking Blocks

Interlocking blocks are best for unvented fords that must withstand heavier loads. They are relatively inexpensive, require less installation time, and provide good aesthetics (Howard et al. 2011). Extra riprap may be added to prevent scouring, and gravel may be added on top to provide a smooth driving surface.

4.7.1.4 Portland Cement Concrete

Portland cement concrete is an alternative material for the crossing surface of a LWC. Portland cement concrete can be precast or cast-in-place. The latter is more difficult to construct and place due to the flowing stream. Portland cement concrete provides a smooth and durable riding surface at crossings, and it requires minimal maintenance. Additional care should be taken to prevent cracking and scour. It may also be reinforced with steel bars to provide additional strength required for movement of larger vehicles and to prevent settlement. Routine maintenance, such as removal of sediments and debris, is required after flooding events.

Lohnes et al. (2001) have suggested the following design parameters for precast panels to be used in unvented fords:

Thickness	8 in.
Size	6 ft x 16 ft (varies based on stream size)
Reinforcement	Number 5 bars at 12 in. centers
Panels	Tied together with 5/8 in. steel cable
Side gaps	Filled with 3 to 5 in. crushed stones

4.7.1.5 Riprap and Gabions

Riprap and gabions are used to protect the banks from being eroded by the high water. Riprap is stone pieces used to protect shoreline from scouring. Concrete rubble may also be used for riprap. A bank slope of 1V:2H is recommended in design of riprap for bank protection. Typical riprap installation for bank protection is as given in Figure 4.16. Riprap should be designed following the guidelines given in HEC-11, *Design of Riprap Revetment* (Brown and Clyde 1989).

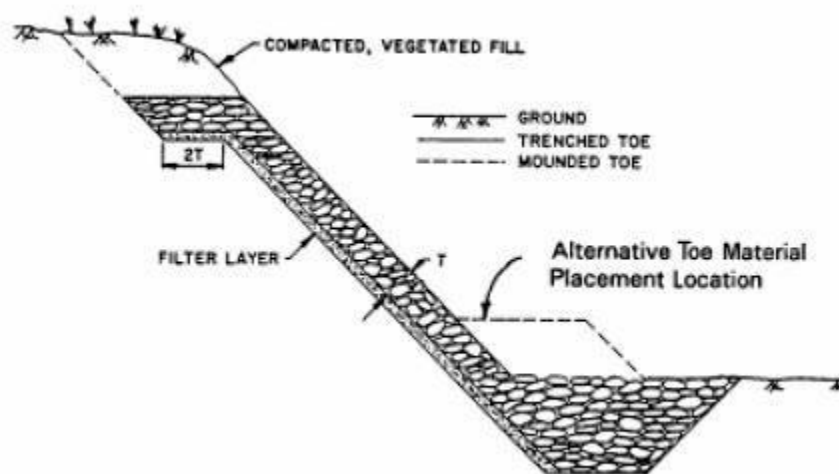


Figure 4.16: Typical riprap installation for bank protection.

Gabions are steel wire baskets filled with rocks and boulders that are generally placed to stabilize river shorelines. Due to their heavier mass, they cannot be displaced by the water, and the water is contained in the stream channel. They are generally filled with rocks 4 to 8 in. in size.

4.7.2 Vented Fords

Vented fords consist of pipes, crossing surface, approach roads, sidewalls and cutoff walls. The streambed in the construction site should be stabilized before the LWC is installed. IDOT's standard specifications should be followed during the installation of pipes or vents, which depends on the material selected.

The crossing surface in vented fords can be constructed similar to unvented fords, using gravel or reinforced concrete. Surfaces should be constructed such that water does not pond after being overtopped.

Sidewalls and cutoff walls are constructed to provide protection to the sides of the structure. The sidewalls may be constructed of concrete, riprap, or geotextiles. Similarly, the approach roads can be improved using the crossing materials to reduce erosion. Bank erosion control can be achieved by the use of gabions or riprap when necessary.

4.7.3 Low-Water Bridges

A low-water bridge construction consists of foundation (footing or pile), substructure (piers and abutments), superstructure (beams and bridge deck), and approach roadway (Lohnes et al. 2001; Motayed et al. 1982). Construction of these bridges should conform to AASHTO specifications (AASHTO 2002).

With low-water bridges, the deck of the bridge acts as the crossing surface. Based on site conditions, bank scour protection measures such as gabions or riprap may be installed similar to the unvented fords.

While a guardrail provides additional safety to the traffic, it also collects debris when the structure is overtopped, which is undesirable in case of LWCs.

4.8 BEST MANAGEMENT PRACTICES

The North Carolina Forest Service defines best management practice (BMP) as a practice, or combination of practices, that is determined to be an effective and practicable (including technological, economic, and institutional considerations) means of preventing or reducing the amount of pollution generated by nonpoint sources to a level compatible with water quality goals (NCFS 2006). BMPs exist for a wide variety of management activities, including those both in and near the channel, in case of LWCs.

To maintain the water quality in LWCs, it is necessary to incorporate standard erosion and sediment control practices (BMPs) into the projects as needed. The implementation and effectiveness of BMPs should be monitored and additional maintenance needs should be identified. Some of the BMPs that apply to the construction of LWCs are discussed below (Clarkin et al. 2006).

4.8.1 Erosion Control Plan

An erosion control plan should be created before starting the project. The practices to be implemented for controlling erosion and preventing sediment from reaching the drainage must be included.

4.8.2 Location of Stream Crossing

LWCs should be located perpendicular to the channel on a straight stretch, whenever possible. This will reduce the effects of stream current on the structure as well as the effect of flow in stream banks.

4.8.3 Timing of Construction Activities

Construction activities should be done during the dry season or when precipitation and runoff are unlikely, whenever possible. Construction should be stopped when soils are too wet for equipment to operate without damaging the soil resource, because there are more chances for water quality degradation.

4.8.4 Construction of Stable Embankments

Approaches and road surfaces with adequate strength should be constructed to support the treadway, shoulders, subgrade, and traffic loads. When fill is required, the embankment must be stabilized with retaining walls, confinement systems, plantings, etc.

4.8.5 Control of Road Drainage

The effects of increased runoff and sediment transport caused by LWCs can be reduced by providing dips that shunt water off the road, culverts that carry water from a road ditch and disperse it on the other side away from the channel, paved approaches, and armored ditches. In some areas, it may be necessary to slow the flow by using sediment basins, check dams, etc.

4.8.6 Servicing and Refueling Construction Equipment

Service and refueling areas should be kept far from wet areas, surface water, and drainages. The soil contamination potential can be minimized by using berms around the sites and using impermeable liners.

4.8.7 Controlling In-Channel Excavation

Heavy equipment should cross or work in and near streams only with specific protection requirements. Excavation in these areas should adhere to all of the following minimum water quality protection requirements:

- Do not excavate outside of caissons, cribs, cofferdams, or sheet piling, unless previously authorized.
- Do not disturb natural streambeds adjacent to the structure.
- Keep disturbance of banks to a minimum, and stabilize any banks that are disturbed.

4.8.8 Diversion of Flows Around Construction Sites

The stream flow around construction sites should be diverted and returned to the natural stream course as soon as possible after construction or before the wet season.

4.8.9 Specifying Riprap Composition

Riprap should be sized and installed to resist erosive water velocities. Care should be taken not to include any material that might add to the sediment load, such as weakly structured rock, organic material, or soil.

4.8.10 Structure Maintenance

Structures and approaches may suffer deterioration from either large runoff events or normal use. Basic maintenance should be provided to protect the structure and prevent damage to resources. At a minimum, annual inspection is necessary to ensure structure and channel compatibility, function, and stability.

4.8.11 Water Quality

Although implementing effective BMPs gives a high degree of water quality protection, there are locations where protection can be verified through a testing program. Water quality parameters and test methods should be specified by an established water quality monitoring plan.

CHAPTER 5: LWC CASE STUDIES

5.1 STUDY SITES

As a part of the research, several case studies were conducted on existing LWCs in Illinois. Five sites were included in the case study, which includes two unvented and three vented fords. The sites were chosen to represent different regions of Illinois.

The LWC research team performed the initial site assessment of 12 LWC location sites in Edgar, Coles, and Christian counties in the summer of 2015. At the time of the visits (June 8 and 10), most of the crossings were overtopped by the flow from the rainfall event (>1.7 in.) that occurred on June 7 and the early morning of June 8. A summary of the site conditions is presented in Appendix B. While most of the structures seemed capable of passing the higher flows safely, some structures were flooded to an extent that it was difficult to identify whether they were unvented or vented structures. From these sites, two vented LWC sites in Edgar County were used in the case study.

Another site visit, along with members of the project's Technical Review Panel (TRP) members, was made on July 10, 2015, to a Logan County LWC site. This site has a vented ford and has been included in the study. The site is different from other sites because the crossing is skewed at a 35-degree angle to the streamflow direction.

In March 2016, two more LWC sites (unvented) were added to the case study, one each in Franklin County and Ogle County. The unvented ford in Franklin County serves an agricultural area, whereas the one in Ogle County is inside a state park and is used mostly by visitors for recreational purposes such as fishing.

5.1.1 Site Selection Criteria for Survey

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

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 5.1.

Table 5.1: LWC Sites Selected for Case Study

County	ID	LWC STR	Latitude	Longitude	Structure Type	Stream
Edgar	Edgar #1	023-4720	39.5084	-87.9236	Above-Grade Vented	North Fork
	Edgar #3	023-5324	39.5136	-87.7297	At-Grade Vented	Fork Big Creek
Franklin	Franklin	NA	38.0171	-88.7879	At-Grade Unvented	Tributary to Akin Creek
Logan	Logan	054-LWC-001	40.0673	-89.5458	At-Grade Vented	Tributary to Salt Creek
Ogle	Ogle	NA	41.9924	-89.4707	At-Grade Unvented	Pine Creek

5.1.2 Topography and Survey

Topography of the watersheds associated with the LWCs was developed from light detection and ranging (LiDAR) data 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 LiDAR data, and elevation information was extracted.

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. Surveys included structure measurement, channel topography up to top of bank, and other relevant data. Four cross sections per site, two cross sections upstream and downstream of the structure, were obtained and then processed.

5.2 HEC-RAS IN LWC STUDY

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 current 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).

Additionally, the US Army Corps of Engineers has also released HEC-GeoRAS, which is an ArcGIS extension designed to process the geospatial data that can be used by 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).

The modeling of LWC in HEC-RAS is similar to culvert analysis. Based on the type of LWC, single culvert, multiple culvert, or bridge analysis is performed. Usually, modeling is done for steady flow conditions with the appropriate design flood of return period 1, 10, and 25 years. In the case of a vented ford, weir flow takes place when the LWC is overtopped.

5.2.1 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 5.2 lists the datasets required for modeling a LWC in HEC-RAS and the source from which they were obtained.

Table 5.2: 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

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 surveys.

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 5.3.

Manning’s n for the main channel and flood plains for natural streams is given in Appendix D (Brunner 2010b; Chow 1959).

Table 5.3: 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

Flow rates for different return periods to use in the HEC-RAS modeling was obtained from USGS StreamStats application for Illinois.

5.2.2 HEC-RAS Modeling

Numerical modeling of LWCs can be divided into three categories: HEC-RAS preprocessing, HEC-RAS processing, and HEC-RAS postprocessing, as shown in Figure 5.1.

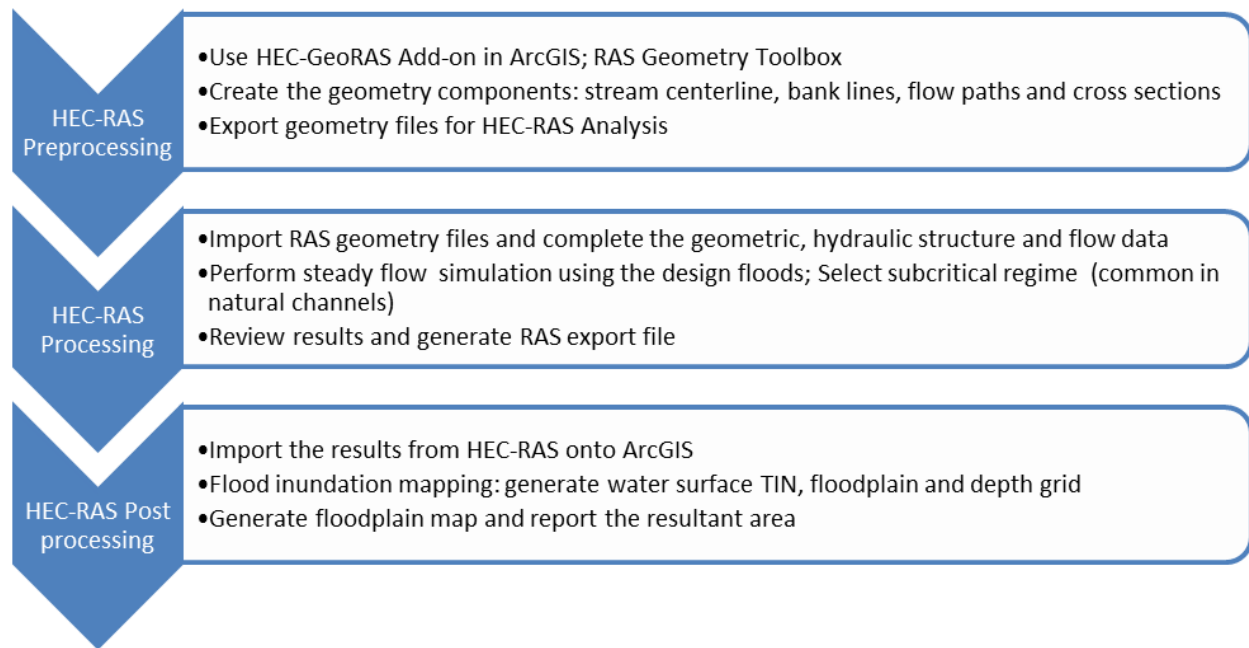


Figure 5.1: Steps in HEC-RAS modeling.

HEC-RAS preprocessing is completed in ArcGIS using HEC-GeoRAS add-on. Creation of geometry files is the first step in the analysis. The RAS geometry toolbox is 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 is completed, the geodatabases and geometry files are exported into HEC-RAS format using the Export RAS Data option.

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

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

5.2.3 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% exceedance flow
- 1-year return flow
- 10-year return flow
- 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 be passed through the pipes in case of vented LWCs and the overtopping depth in case of unvented LWCs is expected to be below 6 in. In some cases, 1-year return flow was used to make additional observations.

10-year and 25-year return flows were used for the flood inundation 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 crossing is overtopped by the flows. The flow to use for analysis depends upon the desired life of the crossing. In this analysis, we have used 25-year flow to perform the flood inundation analysis.

5.2.4 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 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.

5.3 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 5.2, 5.3). 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.

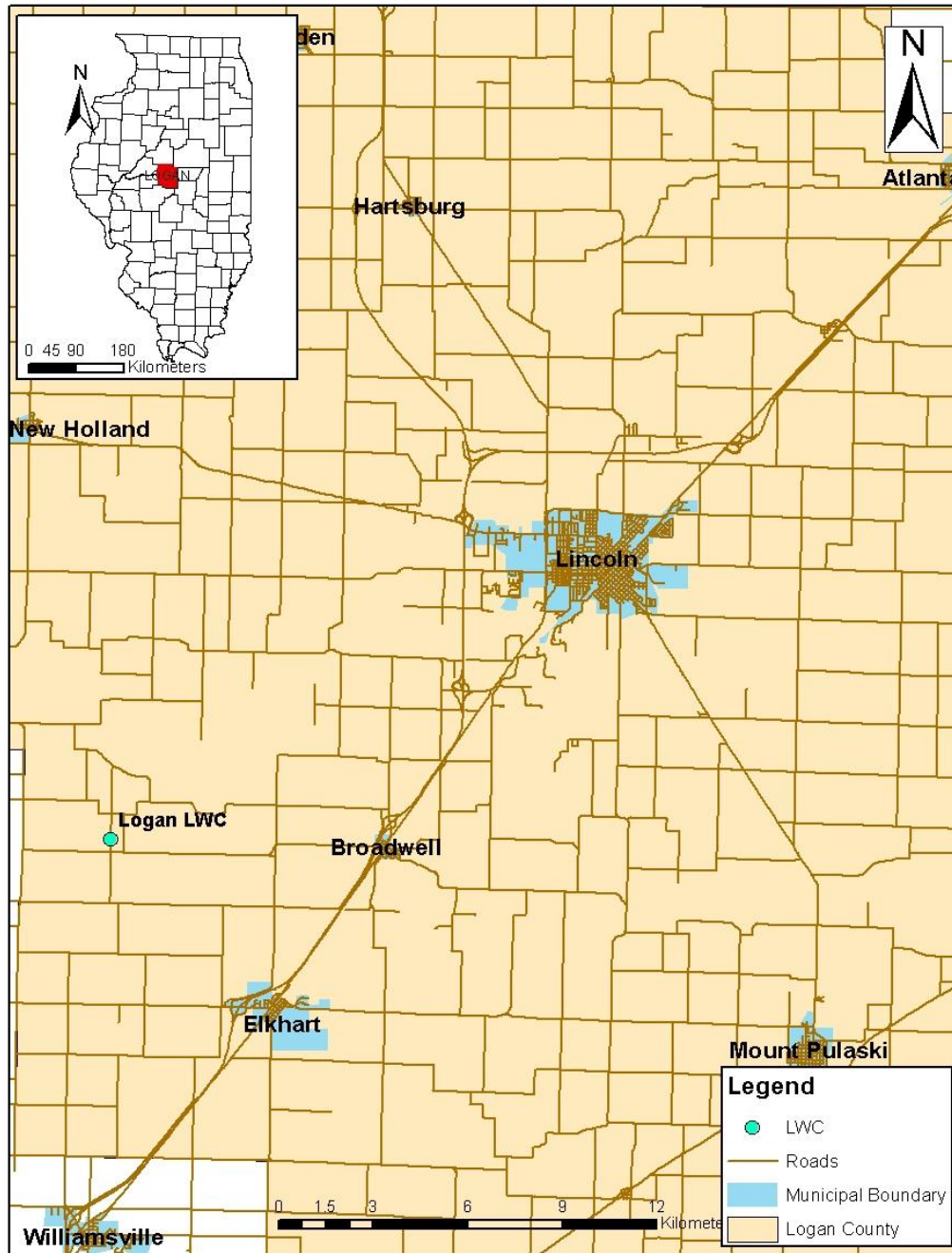


Figure 5.2: Location of LWC in study (vented) in Logan County, Illinois.



Figure 5.3: LWC (vented) in Logan County, Illinois.

5.3.1 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 diameter. The crossing is 27 ft wide and has a skew angle of 35 degrees (Figure 5.3).

Cost: The structure was built in 1988 at a 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 mi.

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.

5.3.2 Watershed

The vented LWC is placed across the stream, which is a tributary to the Salt Creek (Figure 5.4). The LWC area has a main channel slope of 20.33 ft/mi and the watershed upstream of the LWC has drainage area of 3.13 mi² per Illinois StreamStats. It lies within the Salt Creek of Sangamon River Watershed.

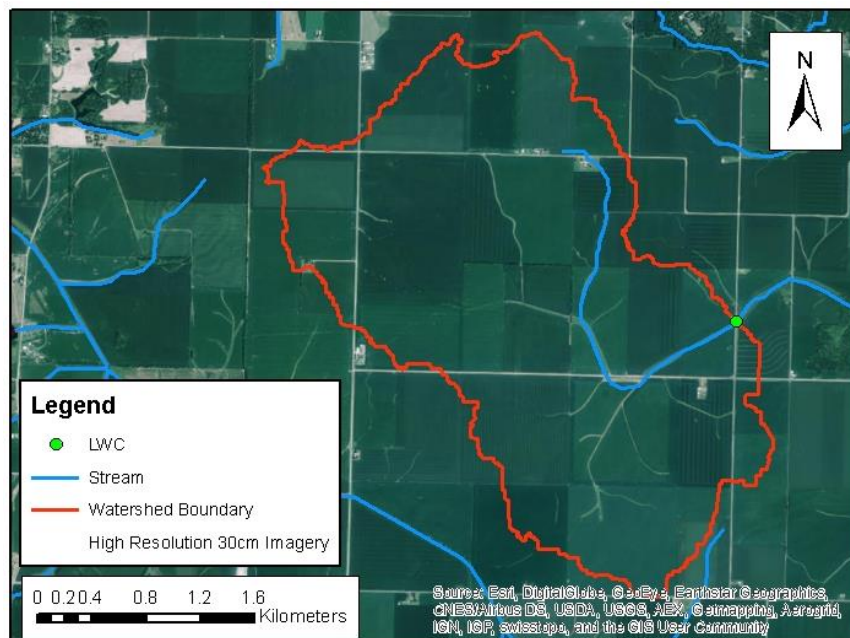


Figure 5.4: Stream network in the Logan LWC watershed.

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 predominately-agricultural area, with a little bit of developed area (Figures 5.4 and 5.5; Table 5.13).

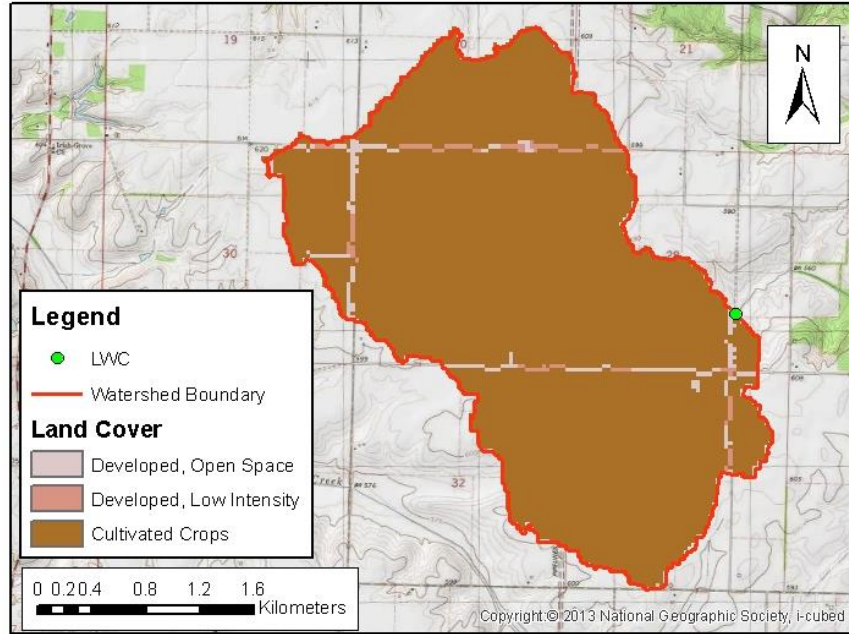


Figure 5.5: NLCD land cover in the watershed associated with the Logan LWC.

Table 5.13: 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

5.3.3 Hydrology

The design flows used in the modeling of Logan LWC are 1% exceedance (E1), 1-year flood (P1) and 25-year flood (P25) which have magnitudes of 28.71 ft³/s, 186.01 ft³/s and 970 ft³/s respectively. More details on how the flow values were obtained can be found in Hydrology section in Appendix G.

5.3.4 Soil

The majority of the watershed has soils in the B or C/D hydrologic group (Figure 5.6, Table 5.14). 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).

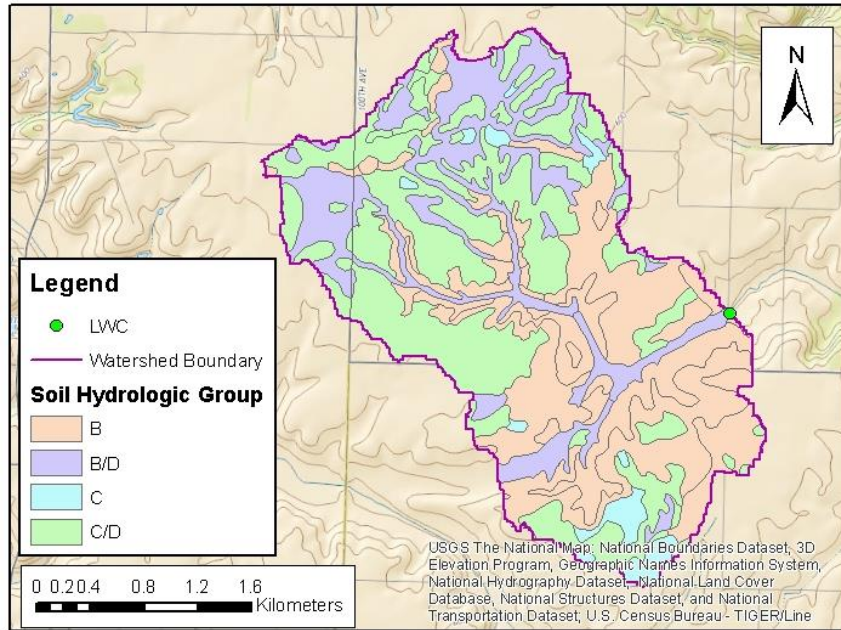


Figure 5.6: Hydrologic soil groups for watershed associated with Logan LWC.

Table 5.14: 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

5.3.5 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 5.7 and 5.8. 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 ft³/s passes through the structure. This flow will be exceeded 4 days in a year, during which the LWC might be impassable. The 1-year flow of 186.01 ft³/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.

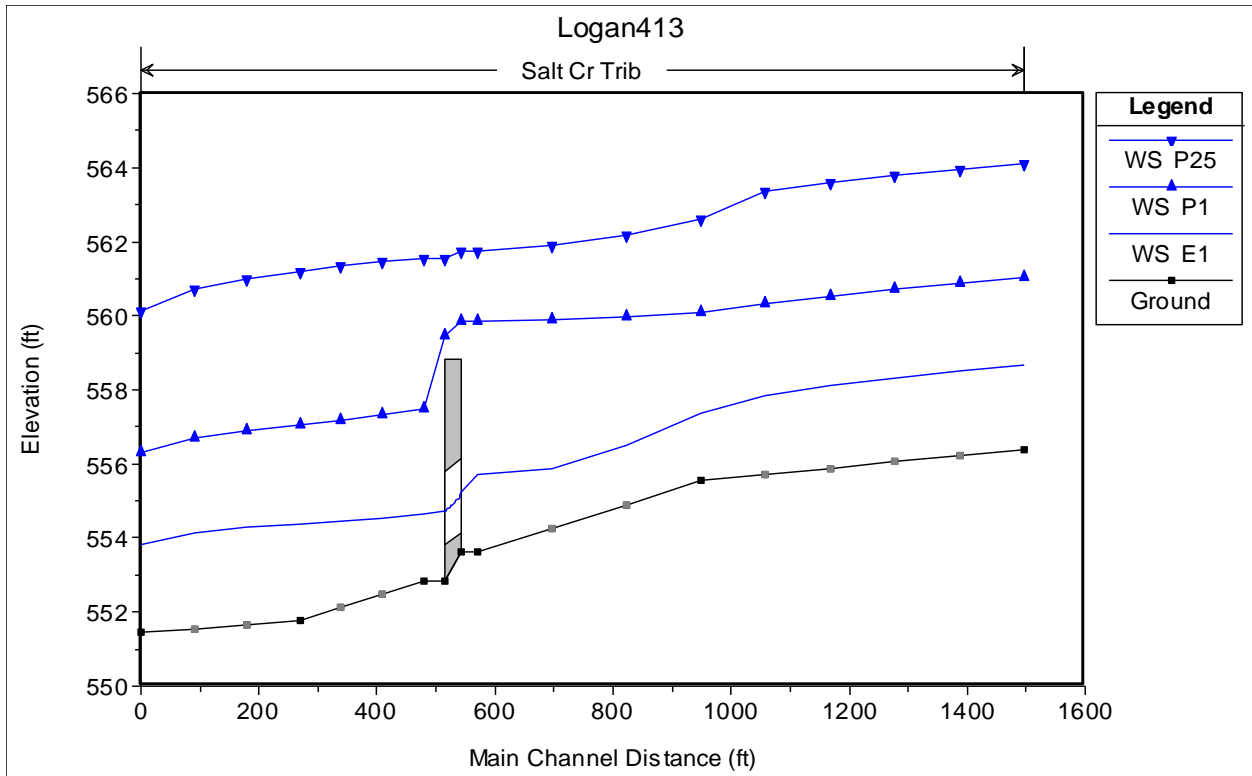


Figure 5.7: Water surface elevation for design flows in Logan LWC.

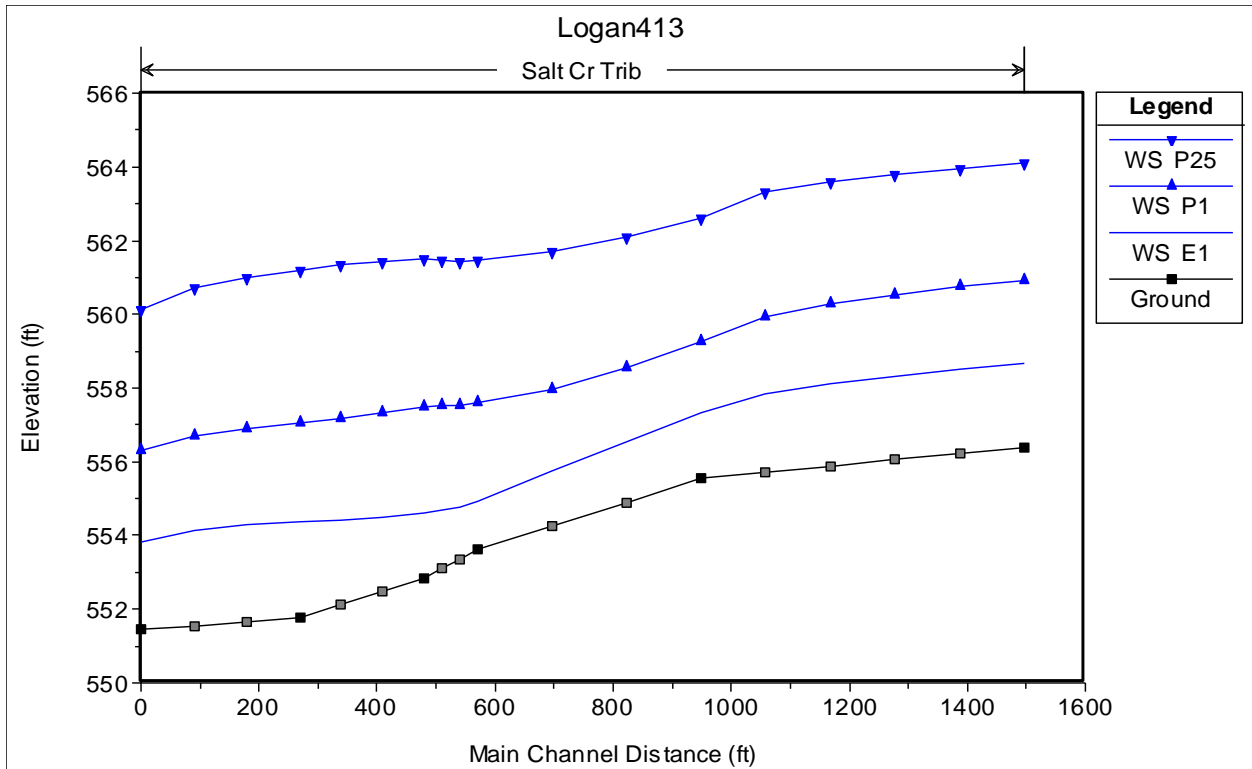


Figure 5.8: Water surface elevation for design flows in LWC free scenario.

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 ft³/s, the floodplain extent was minimally affected by the presence of the LWC (Table 5.15). 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 in. or greater totaled 4.64 acres whereas it is 4.79 acres in LWC free condition. This can be justified by the fact that after the LWC is installed, there is more obstruction to the flow, causing the water to back up more in depth, which results 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 5.9. A portion of the approach roadway, and the surrounding agricultural area is affected by this flood, which was found to be acceptable.

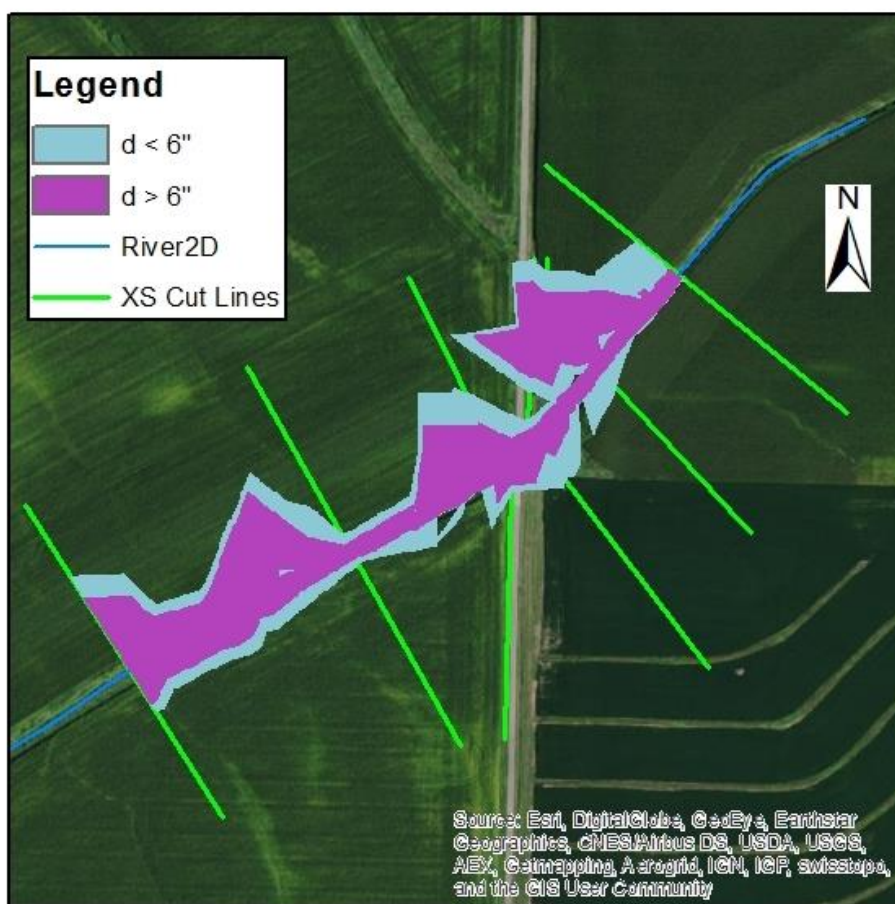


Figure 5.9: Flood inundation map for 25-year flood in Logan LWC.

Table 5.15: 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 5.16 are average bed shear stress for the cross sections upstream and downstream of the LWC at the 1% exceedance flow of 28.71 ft³/s.

Table 5.16: 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/ ft² in the LWC free scenario to 0.08 lb/ ft² in the present scenario. This means that the LWC is restricting the sediment transport in the downstream direction, which may lead to sediment deposition.

5.3.6 Summary and Recommendations

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 mi away when the crossing is impassable. Maintenance will be required across the structure after high flows.

5.4 OTHER CASE STUDIES

Case studies from other sites (Edgar, Franklin and Ogle counties) are included in Appendix G, along with the site hydrology information for these sites.

CHAPTER 6: SUMMARY

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. 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). The current study, conducted by researchers at University of Illinois at Urbana-Champaign in collaboration with the U.S. Army Corps of Engineers–Construction Engineering Research Laboratory, includes findings from the previous studies incorporated with case studies, a LWC survey, and other information specific to Illinois.

Selection, design, and construction of LWCs depend on consideration of various factors such as stream type, hydrology, channel conditions, road use, economics, and aquatic organism passage. 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 economic way that has minimum effects on aquatic organism passage in the stream.

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.

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