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Hydrologic and hydraulic design for low water stream crossings

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LIST OF SYMBOLS AND ABBREVIATIONS

A	Watershed drainage area
A_s	Projected cross-sectional area of low water bridge
A_1	Cross-sectional area of channel upstream
A_2	Cross-sectional area of channel at LWSC
A_3	Cross-sectional area of culvert
ADT	Average daily traffic
B	Length of low water bridge
B_c	Limiting width for channel constriction without altering upstream flow
B_o	Width of undisturbed channel for low water bridge design
BIA	Bureau of Indian Affairs
BLM	Bureau of Land Management
BOR	Bureau of Reclamation
C_o	Depth of cover over culvert
D	Diameter of culvert
DOT	Department of Transportation
E_c	Minimum specific energy
E_1	Upstream specific energy
E_2	Specific energy over LWSC
e	Exceedence time for flow, as a percent of time
FEMA	Federal Emergency Management Agency
FHWA	Federal Highway Administration
Fr	Froude number
g	Acceleration due to gravity
H	Upstream flow depth and velocity head above LWSC height
h_1	Overflow depth before culvert inlet
h'	Backwater flow depth
HEC	Hydrologic Engineering Center
HW_o	Culvert outlet invert up to headwater surface
L	Width of LWSC parallel to flow
L_o	Length of LWSC perpendicular to flow
LWSC	Low water stream crossing
N	Total number of streamflow data observations
n	Manning's roughness
NACE	National Association of County Engineers
NRCS	Natural Resources Conservation Service
P	Wetted perimeter used to find hydraulic radius
p	Probability
Q	Stream discharge
Q_e	Design stream discharge
Q_{top}	Stream discharge overtopping a LWSC
Q_v	Design flow for culverts in vented fords
Q_{vi}	Design flow for culverts under inlet control
Q_{vo}	Design flow for culverts under outlet control

q	Flow per unit width of stream channel
R	Hydraulic radius
r	Rank of flood magnitude from data set
S	Stream channel slope
SCS	Soil Conservation Service
T	Flood return period in years
TW	Tailwater depth after LWSC
USDA	United States Department of Agriculture
USGS	United States Geological Survey
V	Stream velocity
w	Channel width
y_c	Depth for critical flow conditions
y_1	Upstream headwater depth
y_2	Depth of streamflow overtopping a LWSC
Δz	Height of raised streambed, i.e. height of LWSC
Δz_{max}	Maximum height of raised streambed needed for critical flow conditions

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1. INTRODUCTION

1.1. Background

In many areas of the United States there are low volume, or reduced maintenance, roads with old, unsafe bridges. There is a growing need to replace these outdated structures. Problems arise when governmental agencies are faced with limited funding to replace the deficient structures. Low water stream crossings (LWSCs) can provide safe, low cost alternatives to bridges on low volume and reduced maintenance roads.

A LWSC is a structure that provides a reasonable roadway crossing over a waterway. It is designed to be periodically overtopped with high streamflow and therefore closed to traffic during those flood events. A suggested criterion for crossing access is road closure one to three days at a time, totaling not more than 15 days a year (Coghlan and Davis, 1979). These structures are relatively inexpensive and are particularly suitable for low volume roads, streams with occasionally dry beds, or streams having shallow depths during normal conditions.

The use of LWSCs can have special benefits in agricultural regions. Farmers using modern equipment may have problems with bridges that were not designed for farm machinery with widths of 18- to 20-ft, and in some cases 28-ft, with axle loads approaching 80,000-lbs (Rossmiller et al., 1984). In these situations, LWSCs are appropriate as long as other site conditions are favorable. Field access, park infrastructure, primitive roads, or other places where low traffic levels could be expected provide suitable sites for the use of LWSCs as well.

Careful planning and design are important in LWSC project development. As part of the planning process, specific aspects of LWSC design should be investigated. These include: hydrologic, hydraulic, structural, geotechnical, and transportation design considerations. In this thesis, the research focus is on hydrologic and hydraulic design components of LWSC projects.

1.2. Objective and Approach

The objective of this study is to develop a systematic approach for hydrologic and hydraulic design that will aid in the planning of LWSC projects. Hydrologic and hydraulic design guidelines and procedures will be provided, as the necessary tools, for uses by LWSC planners and designers to conduct LWSC analyses and design computations.

Several steps were taken to achieve the objective. First, extensive reviews of previous LWSC studies was completed to provide background information on LWSCs and to help establish design guidelines. Next, a survey was conducted to obtain up to date information on the LWSC design process. The survey feedback provided additional information on design considerations that are currently used in the United States. Then, a thorough investigation of available design methodologies and techniques were explored and improved. Finally, hydrologic and hydraulic design procedures and guidelines, based on previous and present studies, were developed for LWSC design.

2. LITERATURE REVIEW

2.1. Introduction

A compilation of existing information and recent developments regarding the design of LWSCs was developed in this literature review. General information is provided to give background on LWSCs, and specific details for hydrologic and hydraulic design are also presented. Many different research articles have been written on the topic, but some of these resources are outdated. Several methods were used to attain more recent information regarding LWSCs in effort to update previous studies. The approaches used to obtain information for the literature review are described in the following section.

2.2. Review Methodology

A variety of resources and databases were utilized in this literature review. The review of existing data was completed through literature searches, interviews, and field trips. Publications and other useful material for literature review were obtained through library investigations, extensive searches on the Internet, and from contacts with different agencies. The literature search was conducted using various databases available at Iowa State University, including Water Resources Abstracts, Transportation Research Information Services, Applied Science and Technology Abstracts, Environmental Abstracts, and Dissertation Abstracts. The University's on-line library catalog program was also searched. Examples of key words used include: low water stream crossing, low volume roads, low cost water crossing, low water or submersible bridges, and stream crossing.

Information from interviews with engineers from selected states and from field trips also aided in the collection of existing LWSC data. The technical materials were reviewed and information was summarized. The result was a literature review that enabled a compilation of existing LWSC data to be made.

2.3. LWSC Types

There are three common types of LWSCs: unvented fords, vented fords (with pipes), and low water bridges. A LWSC is designed to accommodate low stream flows and allow safe vehicle crossing most of the time, but periodically they are subjected to high flows that overtop the roadway during flooding (Carstens and Woo, 1981). On occasion, these roadway crossings have to be closed until flood flows recede. In the following sections, unvented fords, vented fords, and low water bridges are described in greater detail.

2.3.1. Unvented Ford

The concept of using unvented fords goes back in history. Early settlers of this nation located trails so that they would be able to cross streams at locations where the streambed was hard and the water depth during relatively dry periods allowed for the passage of vehicles (Ring, 1987). The same ideas are often used for modern design and construction of LWSCs.

Unvented fords are waterway crossing structures without pipes. Examples are shown in Figures 1 and 2. These crossings may be constructed of crushed stone, riprap, cast in place concrete, precast concrete slabs, or other appropriate material. They are suitable for crossing streams that are dry most of the year or where normal stream flow depth is less than 6-in, with low velocity (Coghlan and Davis, 1979; Lohnes et al., 2001). They are commonly used on intermittent streams, or perennial streams with low flows (Warhol and Pyles, 1989). Unvented fords are placed to conform to the streambed elevation or may be raised above the streambed. Often times low stream flows for unvented fords flush to the streambed or the design geometry of raised, unvented fords do not allow for fish passage. Therefore, only streams for which safe fish passage is not a consideration should be candidates for unvented ford crossings (Warhol and Pyles, 1989).

According to Motayed et al. (1983), an unvented ford consists of an unsurfaced crossing formed by leveling the streambed for the width of the roadway. Various improvements can be made by adding end walls or providing more stable road surface by the use of asphalt or concrete surfaces, etc., as was done for the LWSC in Figure 2. Markers are usually provided to delineate the edge of the roadway, and the grades of the roadway approaches are shaped to provide a smooth transition for crossing traffic. When capital costs and maintenance costs are taken into consideration, unvented fords are the least costly of the three types of LWSCs.

Figure 1. Unvented ford in Iowa constructed with aggregate

Figure 2. Paved unvented ford in Iowa

2.3.2. Vented Ford

Vented fords are LWSCs with built-in drainage pipe(s) that accommodate low flows without allowing roadway overtopping. Examples are shown in Figures 3 and 4. They are different from traditional culverts. The design of vented fords allows water to periodically exceed pipe flow capacity during high flows, resulting in stream flow over the roadway and occasional closing of the crossing. As with unvented fords, the roadway approaches are designed to provide acceptable grades by shaping the roadway or adjusting the elevation of the crossing. The pipe(s) or culverts placed in the structure may be embedded in earth fill, aggregate, riprap, or portland cement concrete. Vented fords should be considered where the normal depth of stream flow is calculated to exceed 6-in over a raised unvented ford (Coghlan and Davis, 1979; Lohnes et al., 2001).

Careful planning is necessary when vented fords are considered. The construction of vented fords across a stream usually results in narrowing of the natural channel at the crossing site, creating flow disturbances with potential for severe erosion. This has led to designs that include sloped culvert entrances, sloped embankments, and splash aprons or cut-off walls (Motayed et al., 1983). Unlike unvented fords, culverts in these structures can provide a passageway for aquatic life. By proper sizing and careful placement of the pipe, a vented ford may be designed to provide for fish passage (Warhol and Pyles, 1989). The use of vented fords rather than traditional culvert crossings can save money as well. With a vented ford, smaller size culvert can be used and the amount of fill material is reduced. The reduction of materials needed for construction lowers the cost of the structure (Wilent, 2002).

Figure 3. Vented ford with corrugated metal pipes

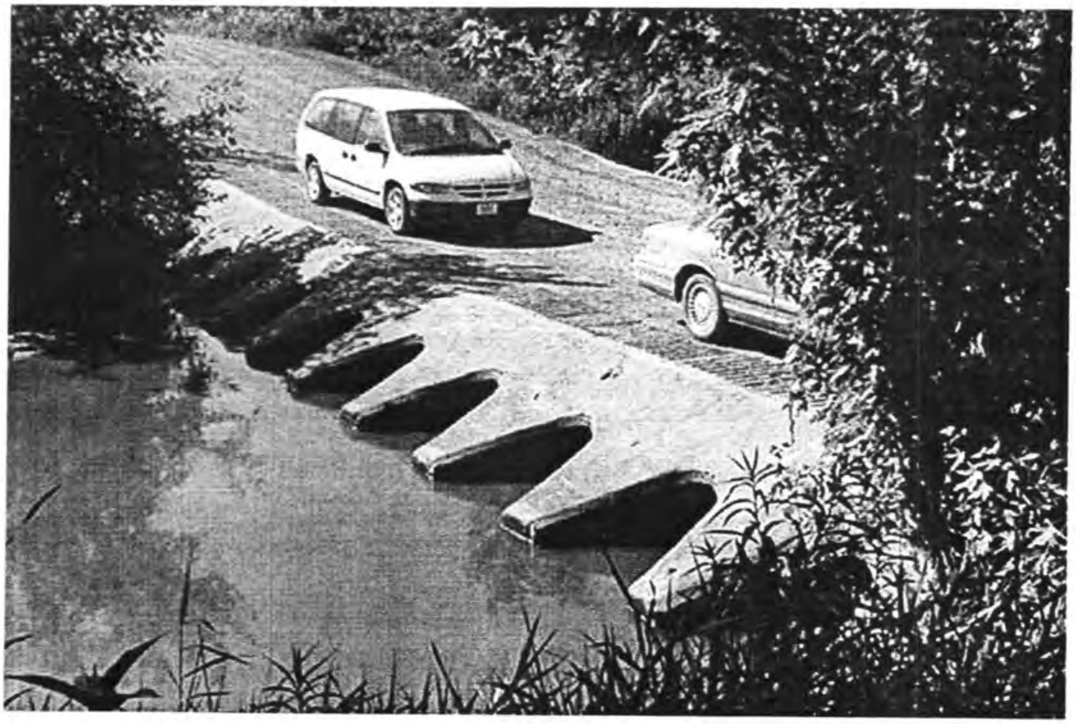

Figure 4. Vented ford reinforced with concrete

2.3.3. Low Water Bridge

Low water bridges are flat-slab bridge decks, with no guard rails, that span waterways providing vehicle crossings at lower cost than standard bridges. They have a smooth cross-section designed so that high water will flow over the slab during flooding events without damaging the structure. Examples are shown in Figures 5 and 6. Similar to high level bridges, low water bridges consist of the following components: a foundation to transmit the load from and above the structure to the natural soil below, a substructure to support the roadway slab and provide an adequate opening for passage of normal flow, and a superstructure consisting of the roadway slab, approaches, etc. (Motayed et al., 1982a).

Low water bridges are especially suitable for drainage basins with high debris potentials that could obstruct vented fords or in environmentally sensitive areas where alteration of streambed is not acceptable (Motayed et al., 1982a). This would include those streams where safe fish passage is important. This type of LWSC is recommended where typical stream flows exceed levels suitable for the use of fords. These structures are also appropriate when a roadway is relatively important and the average daily traffic (ADT) level is high.

The primary concerns in the design of this type of LWSC include erosion of the foundation soil and pavement, and lateral uplift forces of the water passing over the structure (Motayed et al., 1983). There are also safety concerns that should be addressed since these structures do not have guard rails. When looking at total cost, low water bridges are the most expensive of the three types of LWSCs to construct, but they are still considerably cheaper than conventional bridges that are only impassible during the most severe flood conditions.

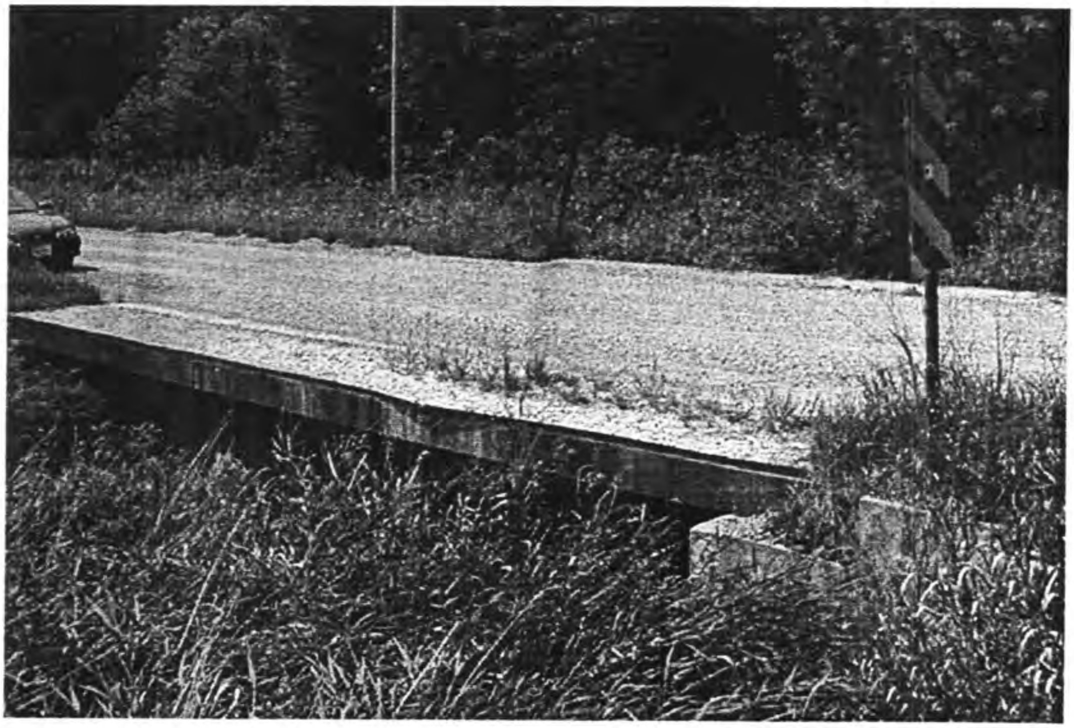

Figure 5. Low water bridge in Iowa

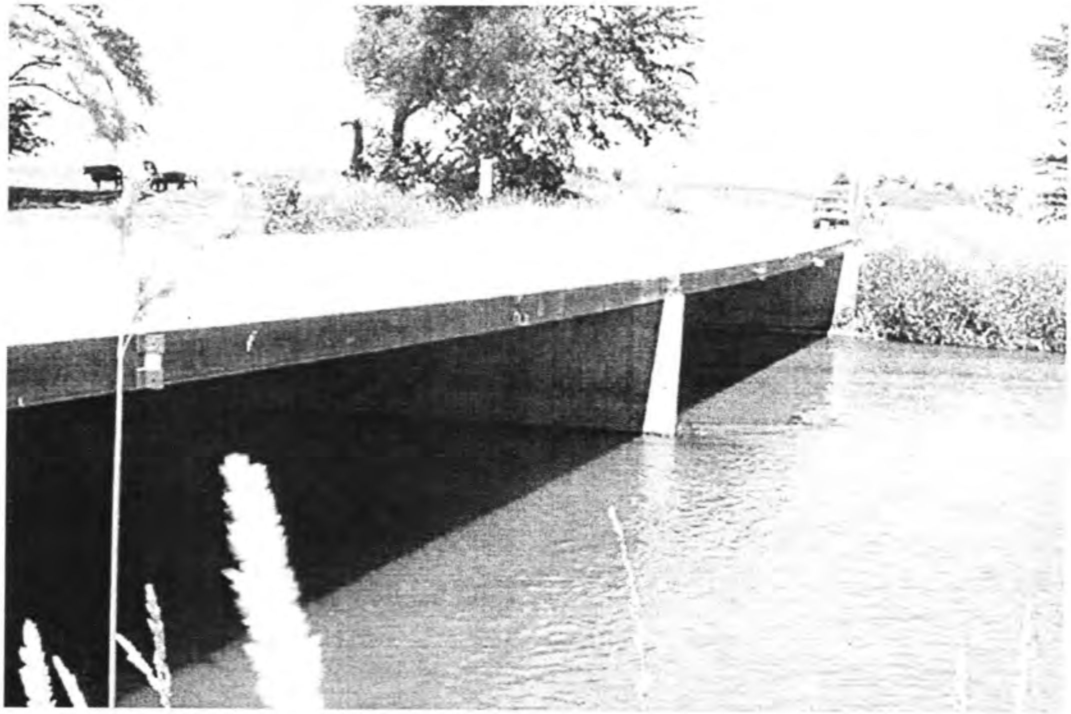

Figure 6. Low water bridge with two spans

2.4. LWSC Design

2.4.1. Design Process

The design of LWSCs is a process that has not been completely formalized for the entire nation. Different methods are used throughout the United States and the design procedure varies depending on location of the project. Motayed et al. (1982a) suggested that well documented information on LWSC selection, design, cost, construction, and performance was often scarce and fragmented so it could not be readily used in design, and that common practice was primarily based on individual experience, judgment, and intuition. Since 1982, additional research has been done on LWSCs and there is more useful information available. There are different design and construction methods that can be found and the challenge is choosing techniques appropriate to use. Thus, it is important that effort be made to compile existing knowledge and data to develop a systematic design approach that can be utilized for future LWSC projects.

LWSC construction projects involving work in or near streams may have special rules or procedures that need to be followed. The requirements for stream crossings vary from state to state and often a permit is required by local natural resources agencies (USDA Forest Service, 2002b). In Oregon, there is a mandatory written plan for installing stream crossing structures. Other states may have similar expectations, so it's necessary to investigate all requirements when planning LWSCs.

General steps involved in the design of unvented or vented fords were developed by Rossmiller et al. (1984) and are presented in Figure 7. As shown in this flowchart, the first step requires analysis of the site and all of the factors associated with the decision to build a LWSC. The next step involves the decision for whether an unvented or vented ford would be more appropriate for the site in question. Following that is hydrologic, hydraulic, structural, geotechnical, and transportation analysis for the LWSC selection.

In the hydrologic and hydraulic analyses, overtopping of the LWSC structure is an important consideration. The overtopping frequency and duration is a function of unique local conditions. Overtopping discharge can be calculated once an acceptable percent of time for overtopping/road closing is decided. Duration of overtopping must be based on the existing physical, social, economic, and political factors for the site.

Crossing elevations and grades are a function of channel and stream bank physical features, and are related to the overtopping discharge depth. Vertical curves are checked for a given traffic speed and headwater depth over the crossing is verified to assure safety. For a vented ford, number and size of pipes can be adjusted accordingly if flow characteristics over the initial design of a structure are not satisfactory.

Selection of material for the crossing relates to overtopping velocity, tractive force, and availability of equipment and supplies in the area. Structure material should be able to hold up under a range of stream flows that may overtop the crossing. The final design considerations for LWSCs involve erosion protection. Stream bed and channel bank protection may be necessary.

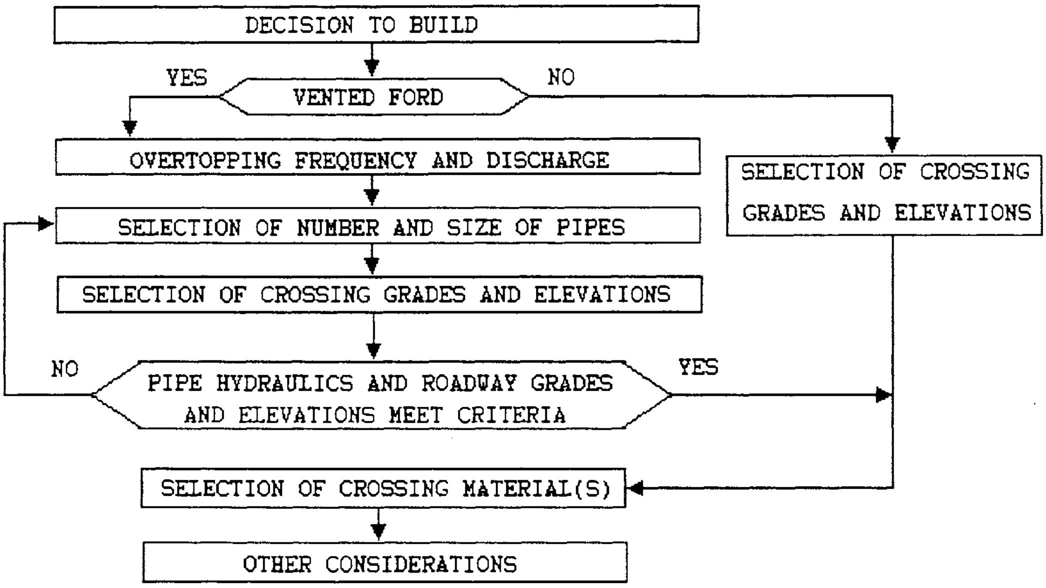

Figure 7. General design steps for a low water stream crossing (Rossmiller et al., 1984)

As demonstrated by Rossmiller et al. (1984), LWSC design components include: hydrologic, hydraulic, structural, geotechnical, and transportation design. Motayed et al. (1982a) suggests that in order to have a successful and maintenance free ford, the experience gained from past performance of fords dictates that a ford should include the following:

- Unerodible paved roadway over which vehicles can smoothly run.
- Two end cutoff walls, one on each edge of roadway, of sufficient depth to provide support to the pavement and counter any subsoil flow
- Rock filled gabion or other endwall on the downstream side to check scouring of the streambed.
- Markers that enable drivers to identify the width of the roadway when flooded.

Analysis of LWSC design components is an important part of the planning process. Data requirements, considerations, design components, design factors, and parameters and specifications are elaborated and discussed in the following sections.

2.4.2. Data Requirements and General Considerations

Motayed et al. (1982b) lists data requirements and considerations that are essential for design of LWSCs. In designing small structures such as LWSCs, a complete economic analysis may not be necessary since the cost savings as a result of the investigation may not justify the effort needed for analysis. In designing a larger structure, substantial savings can be obtained by careful risk-based economic analysis of various designs. Data required for risk analysis include construction costs, site geometry and land use, hydrologic and hydraulic data, traffic data, and flood loss data, which can be found in Table 1. General considerations for the design of LWSCs can be found in Table 2. These considerations are based on the behavior of LWSCs during floods, their performance, and the opinions and common practice of experienced engineers.

Table 1. Summary of data requirements for LWSC design (Motayed et al., 1982b)

Data Needed	Source	Where used in Analysis
1. Construction costs Unit prices of materials, for all structural components	C & M Unit	Capital cost
2. Site geometry and land use Contour map Stream crossing sections Crops (kind, area, location) Buildings (value, location)	 USGS, county, township Field survey USDA, field survey Field survey	 Hydraulic analysis Backwater computation Backwater damage est. Backwater damage est.
3. Hydrologic and hydraulic data Gaging data (stage/discharge) Watershed parameters Flood frequency and magnitude	 USGS, SCS, drainage manual USGS, SCS maps USGS, state highway	 Stage-discharge Hydrograph Annual risk costs
4. Traffic data Design ADT Traffic mix Vehicle running cost Average occupancy Value of time Length of normal route Length of shortest detour route Average speed of traffic	Traffic/planning units	Traffic detour, cost estimation
5. Flood loss data Agricultural products Buildings Bridges and components	Local agencies, USDA, US Army Corps, FEMA, FHWA	Risk analysis

Table 2. General considerations and criteria in LWSC design (Motayed et al., 1983)

Considerations	Criteria
A. Hydrologic & Hydraulic	
1. Frequency of overtopping	Less than 10 times per year
2. Duration of overflow and repair time	Less than 3 days, each occurrence
3. Overtopping depth	Less than 12-in (ADT<100) for 2-yr flow
B. Geomorphic and Land Use	
1. Drainage area and shape	Long and narrow (>3 to 4 times width in length)
2. Stream and basin slope	Steep
3. Channel and overbank	Low valley storage upstream, in a stable stream reach
C. Structural	
General	
1. Vertical curve at dip	Mild and gradual
2. Orientation of structure	Straight; skew should be avoided when possible
3. Approach length	Long, to provide sufficient distance for warning signs
4. Height of pavement above streambed	Less than 4-ft
Fords	
1. Normal daily flow depth	Less than 4- to 6-in
2. Pavement material	May vary from riverbed gravel to concrete
3. Erosion protection	End walls and gabion protection may be desirable, Wide, sloped shoulders in downstream may be helpful
Vented Fords	
1. Pavement and fill materials	Should be dense packed; heavy to withstand erosion and wash out. May be encased in concrete
2. Vents	Pipes of various materials can be used. Should be anchored in ground; both ends beveled to allow easy passage of debris. More than one vent should be used; but fewer lines of larger pipes is desirable
3. Erosion protection	Cut-off walls and splash aprons may be needed. Rip rap protection of slope may be considered
Low Water Bridges	
1. Pavement	Light and loose pavements such as bituminous or gravel pavements are not desirable
2. Bridge deck	Must be heavy to withstand drag, uplift, and lateral forces due to overflow and upstream water. Must be secured to the sub-structure. Upstream and downstream edges should be rounded. Rounded edges with one way camber
3. Erosion Protection	Cut-off walls and impervious aprons may be desirable
D. Signs and Markers	
1. Signs	Must have adequate warning signs
2. Road markers	Guard rails are not recommended (avoid collecting debris) Road markers may be desirable

2.5. Hydrologic Design

The hydrologic analysis is a critical element for LWSC planning. Hydrologic design factors include design flow frequency and magnitude. In this phase of design, flood frequencies and durations that interfere with vehicle crossing are determined, and magnitudes of streamflow are estimated. They can be obtained by analyzing streamflow gage data if available at the LWSC site. If data are not available, hydrologic design can be accomplished with regression analysis of available flow data from nearby streams in the same region, i.e. empirical equations using physical properties of the watersheds or drainage areas including size, slope, runoff/infiltration capacity, etc. The magnitude of design streamflow, with a design flood frequency, is utilized in other components of LWSC design.

Initially in hydrologic design, a decision should be made for the frequency and duration of flooding that will be allowed to cause road closure. This is also referred to as design exceedence time. As described by Ring (1987), the selection of an exceedence time percent is based on site conditions and road use. The need to have the road open depends on the type and volume of traffic and the characteristics of the users. As an example, a field access road could be closed more frequently than a road that serves access to a home, school bus, or mail route. Each site is unique and the decision on acceptable overtopping flow frequency and duration must be based on the existing physical, social, economic, and political factors for that site (Rossmiller et al., 1984).

Once the design for overtopping and road closing frequency or duration is determined, the magnitude of design discharge can be calculated (Ring, 1987). Two different methods have been developed and used in hydrologic design and analysis for LWSCs. One technique involves flow-duration curves, requiring daily stream flow data. The other is a conventional flood frequency method using annual maximum flow data.

The analysis of relationships between flow magnitude-duration and exceedence frequency (so called flow-duration curves) requires the use of daily streamflow data. Since data used for these curves is from daily flow measurement, duration curves can show the number of days in a year when levels of stream discharge are equaled or exceeded. This information is very useful for LWSC design.

The second hydrologic design method for LWSCs is the conventional flood frequency analysis using annual maximum flow data. It estimates the magnitude and frequency of instantaneous flood discharges. In traditional flood frequency analysis, Hydraulic Engineering Circular No. 17 (HEC-17) and the United States Geological Survey (USGS) method may be used (Motayed et al., 1982a). HEC-17 provides design guidelines for encroachments on flood plains using risk analysis. It is an economic accounting of the risks and potential harm associated with design plans under investigation.

The following information is an example of conventional flood frequency analysis used in LWSC planning. This is a case discussed by Pienaar and Visser (1995) involving a 2-yr flood as the design flow, i.e. the return period is two years and the annual exceedence probability is 50 percent. Engineers in South Africa design for the 1-in-2-yr flood, but others believe this is excessive, particularly for large catchment areas, relatively dry areas, or low-order roads where unvented causeways (fords) may be acceptable. Flood analysis is carried out using past records. Historical river data obtained using gauging stations on rivers are used to determine the flood with a 2-yr recurrence interval to be utilized in the design of LWSCs. A design level, which provides an indication of the level of service to be expected from the structure, is chosen after evaluating traffic volume, importance of the route, and availability of alternate routes. In the last step, the design flood is calculated using the 1-in-2-yr flood and a safety factor based on design level for the route. The design flood information is then used to develop a reasonable LWSC design.

2.6. Hydraulic Design

Hydraulic design is an essential component of LWSC design, considering impact of streamflow on the crossing structure, accommodation of flow capacity by the structure, and the effect a structure may have on natural conditions. In LWSC planning and design, it is essential to modify hydraulic design so that adverse affects from stream velocities and other flow characteristics on the crossing structure and foundation are lessened. In addition, if changes to natural stream flow are minimized, damage to the streambed, stream banks, and aquatic environment is less likely to be a problem.

A reduction of hydraulic stress from flows overtopping the crossing is possible by keeping the difference between the upstream and downstream water surface to a minimum, and allowing the water through the crossing at the same rate or near the same rate as the stream flow until the crossing is overtopped (Rossmiller, 1984). Another consideration is uniformity of stream flow passing a LWSC. To help prevent stream channeling at a LWSC site, the crossing grade of the structure should be nearly flat (USDA Forest Service, 2002a).

Hydraulic design parameters for LWSCs include streamflow stage or depth, allowable overtopping flow depth, size or dimension of structure, numbers of pipes or openings to accommodate design flow capacity of the structure, and pipe exit velocity. Flow depths are determined from the design discharge, which is established with hydrologic design and geometry (width, length, and slope) of the streambed and LWSC structure.

Vehicle and driver safety must be considered when determining overtopping flow depth on LWSCs. It has been suggested by previous investigators (Motayed et al., 1983; Rossmiller et al., 1984), that the maximum allowable overtopping flow depth over LWSCs be 6-in. According to Pienaar and Visser (1995), the maximum acceptable flow depth on LWSCs is 4-in for supercritical and 6-in for subcritical flow.

2.6.1. Unvented Ford

When a ford is constructed at stream bed elevation with minimal disturbance to the channel cross-section, there is little effect on the flow of the stream. Therefore, less stream protection may be acceptable. A stage- or depth-discharge relationship for an unraised streambed can be obtained analytically using Manning's equation (Motayed et al., 1982 b; Rossmiller et al., 1984).

The flow over a raised ford is comparable to the flow over a broad crested weir (Motayed et al., 1982b). When considering streambeds raised by LWSCs, flow depths may be computed using broad crest weir equations or empirical equations developed by experiments (Rossmiller et al., 1984). Erosion protection is more important both upstream and downstream of a raised unvented ford due to weir flow and increased stream bed erosion potential.

Exit velocity at the downstream side of the roadway embankment should be computed so that erosion protection measures can be selected and designed, if needed. The exit velocity can be calculated based on downstream flow depth in accordance with elevation of tail water (Motayed et al., 1982b).

2.6.2. Vented Ford

The hydraulic design of a vented ford is similar to that of a culvert. Available design tools include culvert hydraulics and flow equations (Normann et al., 1985), Hydraulic Engineering Circular No. 5 (HEC-5) charts (Herr and Bossy 1965; Normann et al., 1985), a computer modeling program called Culvert Master (Haestad Methods 1999), and culvert design procedures developed by Gupta (2001). FHWA's publication Hydraulic Engineering Circular No. 5, *Hydraulic Charts for the Selection of Highway Culverts*, contains useful design charts. These charts can be used to determine the flow capacity of culverts of various types and sizes under inlet and outlet conditions (Motayed et al., 1982a). The number and size of pipes and headwater depth can be determined from a trial and error process using this document (Rossmiller et al., 1984). Culvert design not only assures desired flow capacity to be met, but also considers flow velocity. Care should be exercised in selecting the culvert so that the size is large enough to limit exit velocity of the flow not to exceed about 10 ft/s to prevent scouring (Motayed et al., 1982b).

In order to establish the number and size of pipes needed for design of vented fords, the following information is needed: location of site, watershed area, design overtopping duration, channel cross-section and roughness coefficient (Manning's n) of existing channel at site, and slope of the channel at site. When determining the number and size of pipes, several other items must be considered (Rossmiller et al., 1984). These include:

- The total width of pipes, including the spaces between them, must be less than the width of the existing channel.
- The headwater elevation for the selected overtopping frequency and estimated discharge must be at, or slightly below, the low point in the roadway
- The pipes can operate under either inlet control or outlet control.
- Pipe lengths may be short, but differences in friction losses due to pipe material still could be significant.
- A large difference between the low point in the roadway and the downstream water surface increases the erosion potential on the downstream foreslope.
- A large difference between the low point in the roadway and the stream bed increases the volume of material needed in the crossing and, thus, its cost.
- The minimum depth of cover over the pipes in a vented ford is one foot.

Culvert style and structure configuration are additional details to consider. Variation of culvert type may result in differences for hydraulic design because each style may affect flow of water differently. Variations in structure configuration are also important to recognize. Pipes in a vented ford may protrude or be flush with the foreslopes of the cross-section. The decisions are left up to the designer, but it should be noted that some arrangements work better than others. For example, vents and embankment should be sloped since it is believed that the sloped entrance and embankment catch less debris and have a natural self-cleaning tendency during high water (Motayed et al., 1982b). Rossmiller et al. (1984) also suggests that a 2:1 foreslope with smoothly trimmed pipes may be self-cleaning on the upstream side, creating a more hydraulically efficient design. For smaller stream crossings where safe fish passage is important (stream channel less than 20-ft wide), bottomless pipe arches or buried pipe arches are possible alternatives to bridges. A buried pipe arch is simply a pipe arch where the invert is covered by 1.5- to 3-ft of native streambed material (Eriksson, 1983).

After a vented ford is designed, built, and put into operation, streamflow characteristics at the structure can change depending on the level of flow. Three types of flow conditions occur at a vented ford (Motayed et al., 1982b):

- During low flow, the crossing experiences open channel flow under atmospheric pressure.
- As flow level rises, low flow changes into a pressure flow when the upstream headwater affects the flow through openings, increasing velocity and discharge; and
- When the roadway is overtopped, the structures experience weir flow and pressure flow.

There are many possibilities for the design of a vented ford. Hydraulically, the culverts should be able to handle design flows and the entire structure should tolerate high flows that overtop the roadway. It is best if stream velocity is not increased after culvert addition and if erosion protection is used when turbulence threatens stream bed stability. A properly constructed vented ford is often able to withstand peak flows that damage or destroy other crossings and costs less to design and build than a conventional bridge over the same stream (Wilent, 2002).

2.6.3. Low Water Bridge

Hydraulic design for low water bridges may be simple if streamflow is undisturbed by addition of a structure. In this case Manning's equation can be used for analysis. If streamflow disturbance is expected, the process becomes more complicated. Bridge hydraulics are needed for analyzing and computing flow stage or flood level, flow velocity, and flow depth in hydraulic design of low water bridges. In a publication by Gupta (2002), the HEC-2 model is described as a tool for computing head losses through bridge structures. The special bridge method, available in the HEC-2 model, can be used to estimate losses through the structure as a result of pressure flow, weir flow, or a combination of the two. Once typical flows and flood levels are determined for a given site, low water bridge size and placement height can be decided based on results from hydraulic analyses. These bridges should be designed to allow regular stream flows to pass under the structure with little disturbance, but let higher flows overtop the structure during flooding.

3. SURVEY

Previous LWSC survey results were analyzed and a new survey was conducted throughout the United States to obtain updated information on LWSCs. A survey on LWSCs done by Shen (1983) was examined as an alternative for survey format and distribution. After considering several options, the new survey questionnaire, presented in Appendix A, was carefully developed and then posted on the Internet. By putting the survey on a webpage, increased participation was expected because the survey was convenient, uncomplicated, and easy to return. This procedure was also expected to reduce the time required for the data collection process. The website address was distributed via e-mails to all state departments of transportation (DOT), selected county engineers, and several agencies including: United States Department of Agriculture (USDA) Forest Service, Natural Resources Conservation Service (NRCS), Bureau of Land Management (BLM), Bureau of Indian Affairs (BIA), Bureau of Reclamation (BOR), and the National Association of County Engineers (NACE).

Approximately 22 detailed responses to the online survey questionnaire were received and analyzed. This was a lower number of responses than expected, so effort was made to increase the amount of feedback. In attempt to get more responses, a simplified survey questionnaire was developed. This revised version was distributed in the same manner as the first survey, resulting in approximately 26 additional responses. The end result was a total of 48 respondents proving feedback to the questions on the web based survey.

3.1 Survey Results

The LWSC survey contains a wide variety of questions. In conducting this survey, the goal was to get updated information on many aspects of the LWSC planning and design process from around the United States. In this analysis of the survey feedback, a focus is put on the information that is important to this thesis research. The following information summarizes responses to questions that are associated with the hydrologic and hydraulic aspects of LWSC design.

Question 3:

"Based on your experience with LWSCs, please indicate factors considered and specify constraints in choosing and designing LWSC structures."

The summary of responses for hydrologic and hydraulic factors considered is in Table 3. In this table is a list of important factors, the percent of respondents that considered the factors to be important, a range of constraints suggested for each factor, and average values for the constraints. Other factors and constraints that were suggested include: consideration of disturbance to the natural channel shape and modification to flow, acknowledgment of debris potential, and selection of a stream section with straight alignment.

Table 3. Summary of factors and constraints for Question 3

Factors	% of Respondents Considered	Range of Constraints	Mean
Overtopping frequency (times/yr)	50	1 to 50	17
Overtopping depth (in)	36	6 to 18	12
Streamflow discharge (cfs)	31	0 to 1000	
Drainage area (acre)	27	3 to 500	
Streambank height (ft)	23	2 to 10	4
Streambed slope (max)	21	2:1 to 10:1	

Question 6:

"If your state/county/agency has built or is going to build LWSCs,

b. Did/would you use inlet and outlet protection and erosion control? What type?"

On this question 31 of 48 (65%) respondents replied and confirmed that they would use inlet and outlet protection and erosion control when necessary. Riprap was most commonly suggested. Concrete aprons, concrete cutoff walls, and geotextile were also mentioned as effective methods.

f. "What data would you use for hydrologic analysis, daily or annual peak flows?"

The results for this question indicate that use of annual peak flow data for hydrologic analysis is more common, where:

- 77% use annual peak flow
- 23% use daily flow

g. "What methods of hydrologic analysis does your agency employ?"

Survey responses indicate that hydraulic analysis have been carried out using the Soil Conservation Service (SCS) method, USGS regression equations, HEC-HMS, TR-55, or rational methods.

4. HYDROLOGIC DESIGN

4.1. Flood Frequency Analysis

Instantaneous annual peak flow data have been used in the conventional flood frequency analysis method. In this method, flow frequency and magnitude are determined by analyzing data for a series of independent annual maximum floods on a given stream. A return period, or recurrence interval is established in the process. As discussed by Bedient and Huber (2002), an annual maximum event has a return period of T years if its magnitude is equaled or exceeded once, on the average, every T years. The exceedence probability is equal to $1/T$. Thus, a 30-yr flood has a 3% probability of being equaled or exceeded in any given year.

When historical streamflow data are available for the site under investigation, flood frequency relationships can be developed by means of various methods, which can be used to determine flow frequencies of given magnitudes. The empirical method is one of the relatively simple methods that can be used (Gupta, 2002). With the empirical method, magnitudes of annual maximum flows are ranked in descending order, where the largest streamflow is assigned a rank (r) of 1 and the lowest level of flow is given a rank equal to the total number of data observations (N) available for the analysis. After the data are ranked, the probability (p) for each flow can be determined using a formula proposed by Weibull (Gupta, 2002), i.e. $p = r/(N+1)$. The estimated return period for each flood level can then be calculated by taking the reciprocal of the probability.

At a site where a stream does not have historical data available, regional regression equations may be used to estimate flood levels and frequency. Statistical regression and frequency analysis techniques are combined to develop equations for predicting peak flows at ungaged sites based on observed streamflow records at gaged sites in the same hydrologic region. In order for the regression analysis to be effective, the gaged stream location must be situated in an area similar to the site under investigation where there is reasonably similar hydrology, land use, topography, and climate. The USGS (2002) has developed and published regional regression equations for every state in the U.S. based on annual peak flow stream data collected. These equations can be used for LWSC design if a decision is made to use annual peak flows for hydrologic design where no stream data are available.

When considering the use of instantaneous peak flow data for the hydrological design of LWSCs, the disadvantage is that the duration of flooding cannot be determined with this type of analysis. When LWSCs are being planned it is important to determine how many days a year would be acceptable for road closure due to overtopping. Since the design flood frequency in the traditional flood analysis method uses instantaneous flow data, i.e. annual maximum flows, and gives the yearly return period of a design flood or bigger one or the annual exceedence probability, the period of occurrence on a daily basis cannot be determined. Therefore, this method does not meet the need of LWSC design if acceptable road closing duration is considered as a design parameter.

4.2. Flow-duration Curves

Daily flow data are used to develop flow-duration curves (Rossmiller et al., 1984). Flow-duration curves indicate the percent of time, within a certain period, in which given rates of stream flow (design flow) were equaled or exceeded. The percent of time stream flow may exceed design flow must be determined before flow-duration curves can be used effectively. The decision to use an exceedence time percent equal to 10 percent would mean that water should flow over the road, causing closure, an average of 37 days per year. The resulting design discharge would be $Q_{10\%}$. The selection of a design discharge of $Q_{2\%}$ would mean that the acceptable closing percent of time per year is 2 percent, indicating that the road would be closed an average of 7 days per year. The flow-duration curve charts are useful because they provide information for various flow rates, how often they occur, and how long (days in a year) to expect them.

When historical daily data from gaged streams are available, average daily stream discharges can be ranked in ascending order of magnitude. The percent of time in a streamflow record, during which flow is equaled or exceeded, is calculated for each magnitude of flow. The data can then be used to generate flow-duration-frequency curves which can be utilized to determine the LWSC design flow for a given acceptable time of road closure in a year.

If stream discharge and duration information are not available at stream crossings and no recorded data exist, an empirical approach is required. Streamflow records are typically available from the USGS for streams with gauging stations. This information can be used to make streamflow estimations for streams without gauges. In some states the data have been statistically analyzed on a regional basis and regression equations developed (Ring, 1987).

Flow duration information at ungauged sites in Iowa can be found in Rossmiller et al. (1984). Flow-duration-area equations for ungauged streams were developed from flow data at gauged sites by dividing Iowa into three different hydrological regions based on geomorphology and hydrology, shown in Figure 8 (Rossmiller et al., 1984). The following equation can be used to analyze ungauged sites in Iowa:

$$Q_e = aA^b \tag{1}$$

where Q = discharge (cfs), A = drainage area (mi^2), e = exceedence time percent, i.e. duration of road closure per year due to overtopping, and a, b = regression coefficients found in Table 4. If no regional equations have been developed in the area under investigation, an alternative is to use adjacent flow-duration curves from a nearby stream with similar conditions (Ring, 1987).

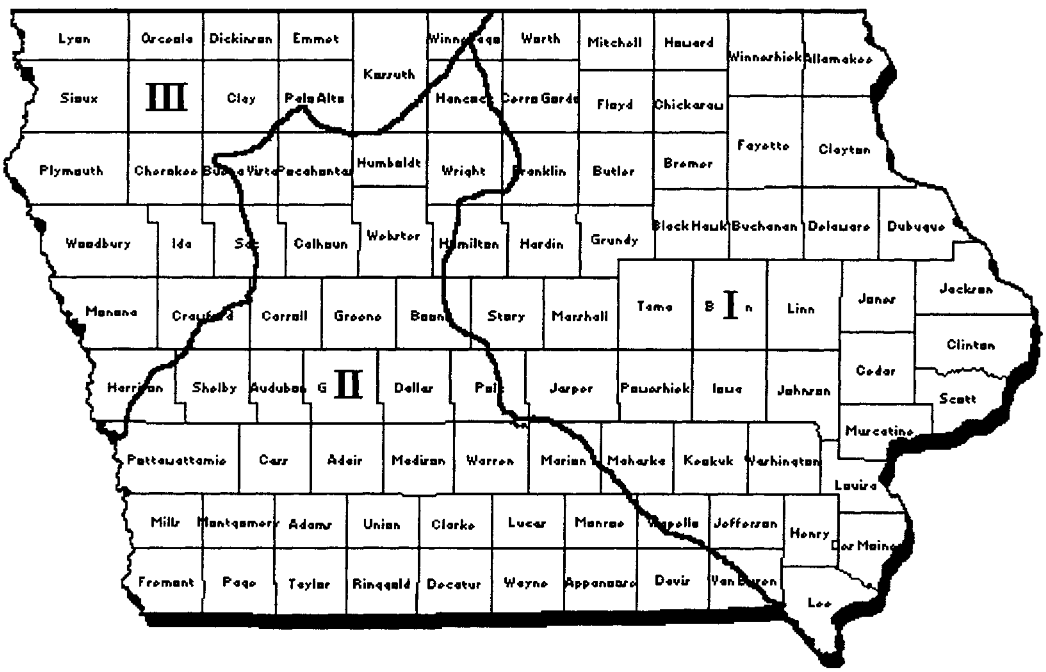

Figure 8. Hydrologic regions of Iowa for discharge estimation (Rossmiller et al., 1984)

Table 4. Regional regression coefficients for equation (1)

Exceedence Time e, %	Region a	I b	Region a	II b	Region a	III b
50	0.17	1.05	0.06	1.09	0.02	1.24
25	0.52	1.01	0.24	1.06	0.04	1.25
10	1.37	0.98	0.91	1.00	0.15	1.19
5	2.58	0.96	2.26	0.95	0.33	1.15
2	6.78	0.90	6.78	0.90	1.23	1.06
1	13.5	0.85	13.5	0.85	3.56	0.96

As mentioned earlier, Coghlan and Davis (1979) suggested that total duration of road closure should not exceed 15 days in a year, which is approximately a 4 percent exceedence time. It should be recognized that duration of road closure is highly dependent on site specific factors so the duration could vary from project to project. In some situations, less than 15 days of road closure in a year may be desirable while more than 15 days could be suitable at another location. Therefore, lower range exceedence probabilities, less than 10 percent, are presented for use in LWSC planning and design.

In this thesis research, tools presented by Rossmiller et al. (1984), have been elaborated to make them more convenient to use for LWSC design in Iowa. Table 4 was expanded to provide values for regression coefficients *a* and *b* for exceedence times less than 10 percent. These new values can be found in Table 5 where the values in bold are from Rossmiller et al. (1984), and the italicized numbers represent new values added.

The new values listed in Table 5 were generated by plotting and analyzing the coefficient values calculated by Rossmiller et al. (1984). Trend lines were added to fit plotted data. Equations for the trend lines could then be used to estimate new coefficient values at various exceedence probabilities. Figure 9 shows the plotted data for regression coefficient *a*, with trend lines and respective equations. Regression coefficient *b* has a linear trend, so it was not necessary to plot.

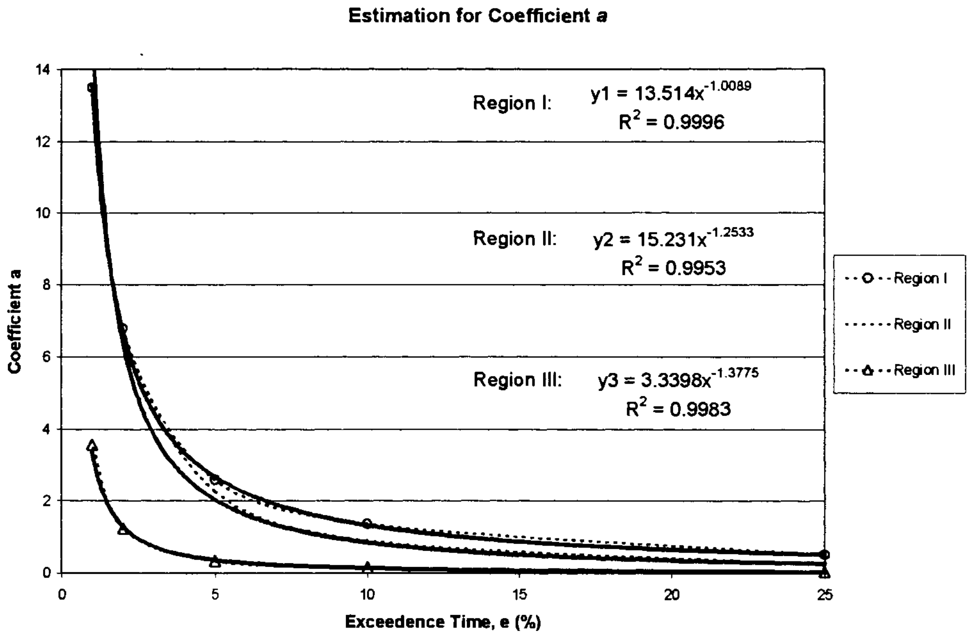

Figure 9. Fitted curves and equations for determination of regression coefficient *a*

Table 5. New regional regression coefficients for estimating duration of flows in Iowa

Exceedence Time e, %	Region I a	b	Region II a	b	Region III a	b
50	**0.17**	**1.05**	**0.06**	**1.09**	**0.015**	**1.24**
25	**0.52**	**1.01**	**0.24**	**1.06**	**0.04**	**1.25**
10	**1.37**	**0.98**	**0.91**	**1.00**	**0.15**	**1.19**
9	*1.47*	*0.98*	*0.99*	*0.99*	*0.17*	*1.18*
8	*1.66*	*0.97*	*1.12*	*0.98*	*0.19*	*1.17*
7	*1.90*	*0.97*	*1.33*	*0.97*	*0.23*	*1.17*
6	*2.22*	*0.96*	*1.61*	*0.96*	*0.28*	*1.16*
5	**2.58**	**0.96**	**2.26**	**0.95**	**0.33**	**1.15**
4	*3.34*	*0.94*	*3.10*	*0.93*	*0.50*	*1.12*
3	*4.46*	*0.92*	*4.25*	*0.92*	*0.74*	*1.09*
2	**6.78**	**0.90**	**6.78**	**0.90**	**1.23**	**1.06**
1	**13.50**	**0.85**	**13.5**	**0.85**	**3.56**	**0.96**

Equation (1) was used with data from Table 5 to generate useful charts for LWSC hydrologic design in Iowa. With these charts, an exceedence time percent can be chosen for a site with known drainage area to determine the design discharge. Figures 10, 11, and 12 are charts that were generated for each of the three hydrologic regions in Iowa. Similar charts can be made for other regions with use of regression equations and available daily streamflow data.

In summary, the flow-duration method has advantages over the traditional flood frequency analysis method for hydrologic design of LWSCs. The design flow frequency or exceedence time percent developed from daily flow data, indicate not only how often a design flood or larger one would occur, but also how long the flood would last.

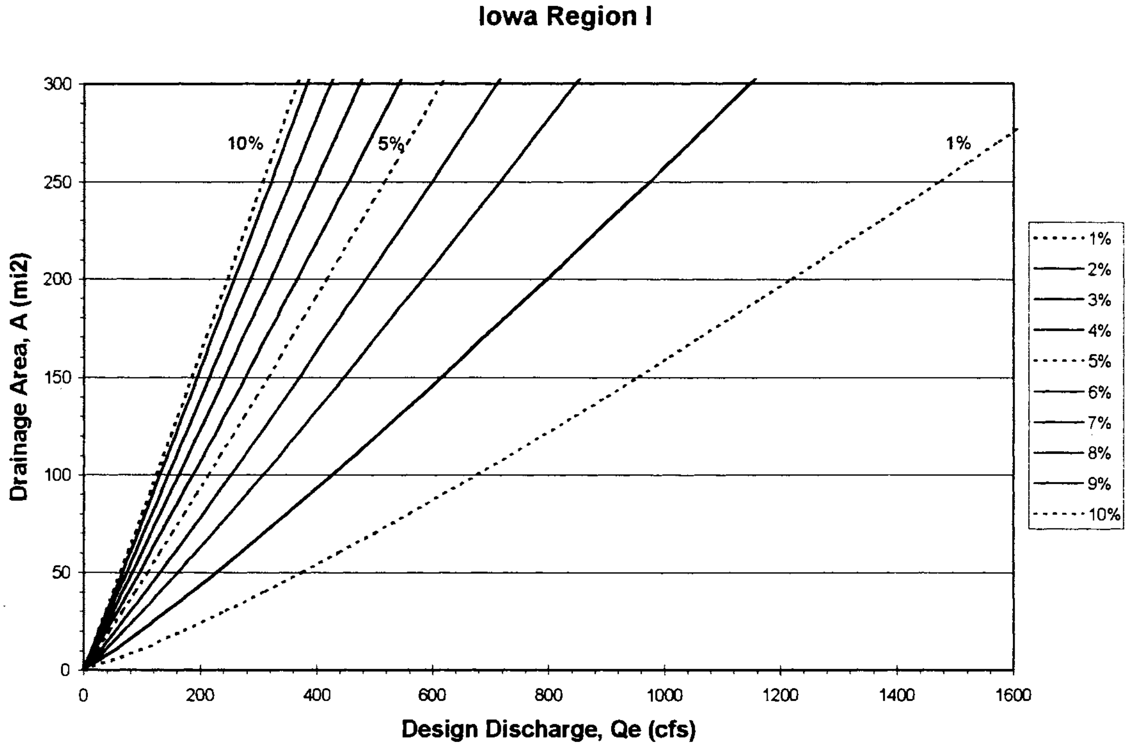

Figure 10. Design discharge estimation for Region I in Iowa

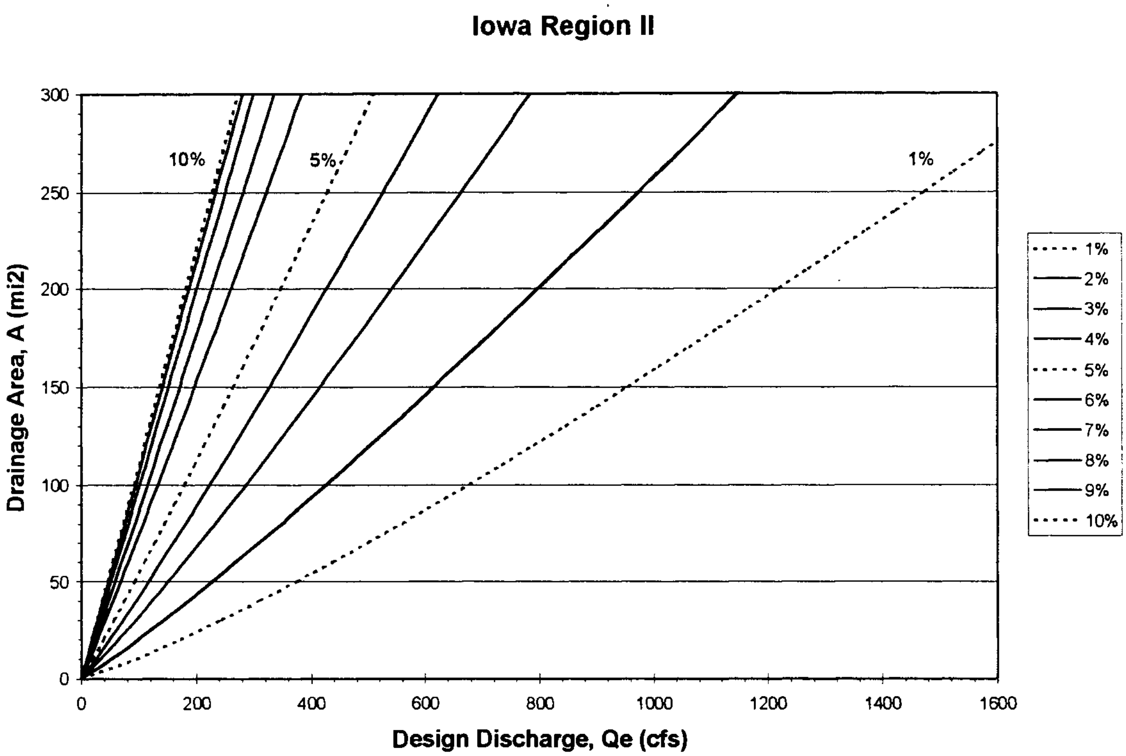

Figure 11. Design discharge estimation for Region II in Iowa

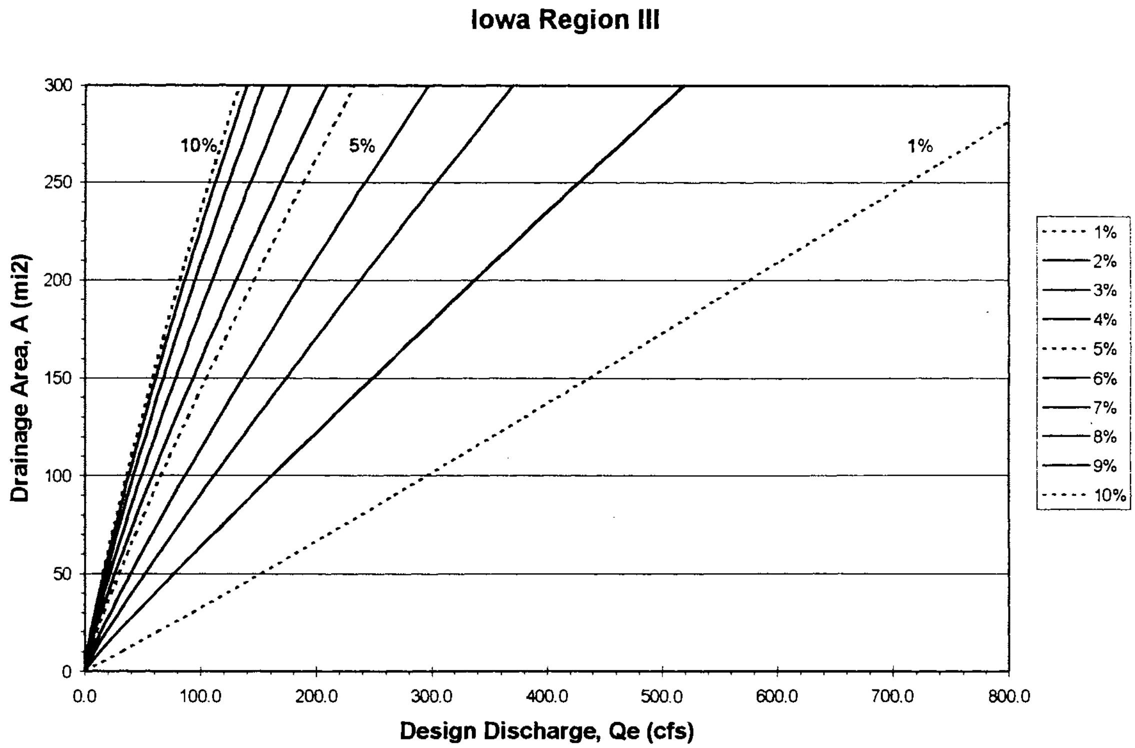

Figure 12. Design discharge estimation for Region III in Iowa

5. HYDRAULIC DESIGN

5.1. Unvented Fords

5.1.1. Flow Regimes and Overtopping Flow Depth

In research by Chaudhry (1993), it was determined that for a step rise, Δz, on the channel bottom, water surface elevation over the step drops if the upstream flow is subcritical, while the overtopping flow depth increases if the flow upstream of the step rise is supercritical. An equation was derived with assumptions that the pressure distribution is hydrostatic and there are no losses, which can be explained by the following (Chaudhry, 1993):

$$\frac{dz}{dx} = \left(F_r^{\,2} - 1\right)\frac{dy}{dx} \qquad (2)$$

where F_r is the Froude number, *dz/dx* represents the raised LWSC height, and *dy/dx* indicates the change in flow depth. The Froude number can be determined using

$$F_r = \frac{V}{\sqrt{y_1 g}} \qquad (3)$$

where V is the stream velocity and y_1 is the existing headwater depth.

Unvented fords can be elevated above the streambed and it can be assumed that *dz/dx* is always positive when a crossing is raised. This implies that $(F_r^{\,2}\text{-}1)$ and *dy/dx* are either positive or both negative values. Thus, we can see that:

- $F_r < 1$ (subcritical), $dy/dx < 0$, indicating that depth decreases
- $F_r > 1$ (supercritical), $dy/dx > 0$, indicating that depth increases

This concept is very important for the hydraulic design of LWSCs, particularly when choosing between an unvented ford on the streambed and a raised unvented ford. The existing conditions should be evaluated to determine if the flow of the stream is subcritical or supercritical for the design flow Q_e. In most natural rivers the streamflow is subcritical. However, there are unique circumstances in which supercritical flow can exist. Since flow regime is an important parameter in LWSC design, especially for unvented fords, the state of existing flow should always be analyzed.

According to Pienaar and Visser (1995), the maximum acceptable flow depth on LWSCs for safe vehicle passage is 4-in for supercritical flow and 6-in for subcritical flow. If design flow depth is less than or equal to 6-in, an unvented ford on the streambed is acceptable under subcritical flow conditions. If the subcritical flow depth for the design discharge is more than 6-in, the LWSC height may be raised. This will cause a decrease in the overtopping depth such that 6-in or less may be achieved. Methods that can be used to determine if raising an unvented ford will result in overflow depth less than or equal to 6-in will be discussed later in this report.

If the design flow depth is less than or equal to 4-in under supercritical flow conditions, an unvented ford on the streambed is acceptable. Under supercritical flow conditions it is not recommended to raise an unvented ford because the result is increased water surface elevation over the LWSC compared to the upstream water surface.

The specific energy is measured with respect to the channel bottom and is defined as the sum of the depth, y, and velocity head, $V^2/2g$ (Gupta, 2002). According to Modi and Seth (1991), there is a limit up to which the specific energy for a given discharge can be reduced by increasing the height of the raised structure, Δz. This means there is a limiting or maximum Δz at which the specific energy at a raised unvented ford is equal to the minimum specific energy, E_c, for the upstream discharge. The minimum specific energy occurs when the critical depth, y_c, of flow is attained. If Δz is increased beyond the maximum value then the upstream water level and flow rate are influenced, choking the stream. When a stream is choked, the upstream water level is lifted and the flow discharge is reduced. Assuming there is no loss in total head when stream flow passes a structure, the following equation can be used:

$$E_1 = E_c + \Delta z_{\max} \tag{4}$$

where E_1 is the specific energy upstream, E_c is the minimum specific energy for a given discharge, and Δz_{max} is the maximum height of a raised LWSC that is needed to achieve critical flow conditions.

Analyses using hydraulic principles were performed to determine how overtopping flow depth varies when the height of a raised unvented ford is changed. Table 6 shows results for various stream channel properties and levels of stream flow. As shown in the table, when $\Delta z < \Delta z_{max}$, overtopping flow depth is greater than critical flow depth, thus allowing subcritical flow over the crossing. When $\Delta z = \Delta z_{max}$, critical flow depth crosses the structure. Finally, if $\Delta z > \Delta z_{max}$, the overtopping flow depth is less than critical flow depth causing supercritical flow over the structure.

Table 6. Variation of overtopping depth, y_2, with structure height, Δz

Constants	Δz	y_2
Q=150 cfs	2.00	4.04
n=0.04	2.50	3.40
w=8 ft	**3.02**	**2.22**
S=0.002	3.20	2.12
Δz_{max}=3.02 ft	3.80	1.72
yc=2.22 ft	4.50	1.24
Q=150 cfs	0.40	1.21
n=0.04	0.50	1.07
w=40 ft	**0.62**	**0.76**
S=0.002	0.70	0.70
Δz_{max}=0.62 ft	0.80	0.64
yc=0.76 ft	1.00	0.50
Q=1500 cfs	4.00	9.42
n=0.04	5.00	8.06
w=20 ft	**6.03**	**5.58**
S=0.002	6.50	5.26
Δz_{max}=6.03 ft	7.00	4.92
yc=5.58 ft	7.50	4.60
Q=1500 cfs	1.5	5.56
n=0.04	2	4.82
w=40 ft	**2.49**	**3.52**
S=0.002	2.7	3.36
Δz_{max}=2.49 ft	3.5	2.84
yc=3.52 ft	4	2.5
Q=150 cfs	0.30	2.92
n=0.04	0.40	2.72
w=8 ft	**0.53**	**2.22**
S=0.01	0.60	2.18
Δz_{max}=0.53 ft	0.70	2.10
yc=2.22 ft	1.00	1.90
Q=150 cfs	0.05	1.54
n=0.04	0.10	1.42
w=20 ft	**0.15**	**1.20**
S=0.01	0.20	1.16
Δz_{max}=0.15 ft	0.30	1.10
yc=1.20 ft	1.00	0.64
Q=1500 cfs	0.50	6.62
n=0.04	0.60	6.34
w=20 ft	**0.75**	**5.58**
S=0.01	0.80	5.54
Δz_{max}=0.75 ft	1.00	5.40
yc=5.58 ft	2.00	4.74
Q=1500 cfs	0.10	4.12
n=0.04	0.15	3.94
w=40 ft	**0.22**	**3.52**
S=0.01	0.30	3.46
Δz_{max}=0.22 ft	0.40	3.40
yc=3.52 ft	1.00	3.00

The location of critical flow occurs at the highest point of a raised streambed, when it is a rounded bump step rise (Chaudhry, 1993). Since the geometry of a ford has a flat top, the location of critical flow on the structure can be variable. For a given design flow, critical flow, y_c, occurs approximately at the center of the structure when $\Delta z = \Delta z_{max}$. When $\Delta z < \Delta z_{max}$, the location may move downstream from the center of the LWSC, while when $\Delta z > \Delta z_{max}$, it moves upstream from the center of the LWSC. Therefore, when Δz_{max} is used for LWSC design and 6-in depth as a design constraint, we can assume that the 6-in design overtopping depth will occur near the center of the crossing. If streamflow is less than design flow after the structure is built, the design overtopping depth location may move upstream from the center of the crossing and the depth over the crossing would be less than the design overtopping depth.

The concept of using a maximum height, Δz_{max}, for a raised LWSC to achieve critical flow over the crossing can be a very useful tool for the hydraulic design of these structures. It will be used as a part of the design methodology to be described in the next sections.

5.1.2. Hydraulic Principles

If an unvented ford is to be placed conforming to the streambed with minimum disturbance to channel cross section, Manning's equation can be used for analysis. This equation is written as:

$$Q = \frac{1.486}{n} A R^{2/3} S^{1/2} \tag{5}$$

where Q = discharge (in English units), A = cross-sectional area of channel, R = hydraulic radius, S = channel slope, and n = Manning's roughness factor.

The roughness coefficient (n) is a function of channel material, degree of irregularity in channel cross-section surface, variation in cross-section along the channel's length, effect of obstructions, height of vegetation, and degree of channel meandering (Rossmiller et al., 1984). Manning's equation can be used in this form, assuming a rectangular cross section:

$$Q_e = \frac{1.486(y_1 w)^{5/3}}{n(w + 2y_1)^{2/3}} S^{0.5} \tag{6}$$

where Q_e is the design discharge from hydrologic analysis, n is the roughness coefficient, S is the channel slope, w is the channel width, and y_1 is the depth of flow associated with Q_e. For very wide and relatively shallow channels, for example

$$\frac{w}{y_1} \geq 10$$

Manning's equation can be simplified and rearranged to get a new equation. Equation (7) provides a quick method for estimating the headwater depth, y_1.

$$y_1 = \left[\frac{nQ_e}{1.486w\left(S^{0.5}\right)}\right] \tag{7}$$

The computed y_1 value can then be compared with the allowable maximum flow depth to determine if an unvented ford would be an acceptable option. If the stream under analysis doesn't have a wide channel, i.e. when

$$\frac{w}{y_1} < 10$$

Manning's general equation must be used to solve for y_1. With substitution, Manning's equation can be written as the following:

$$w(y_1^{-2.5}) + 2(y_1^{-1.5}) = \frac{1.486^{1.5} w^{2.5} S^{0.75}}{n^{1.5} Q_e^{1.5}} \tag{8}$$

Using Equation (8), the headwater depth, y_1, can be determined through trial and error or by use of a mathematical equation solver. This depth can then be compared to the maximum allowable overflow depth.

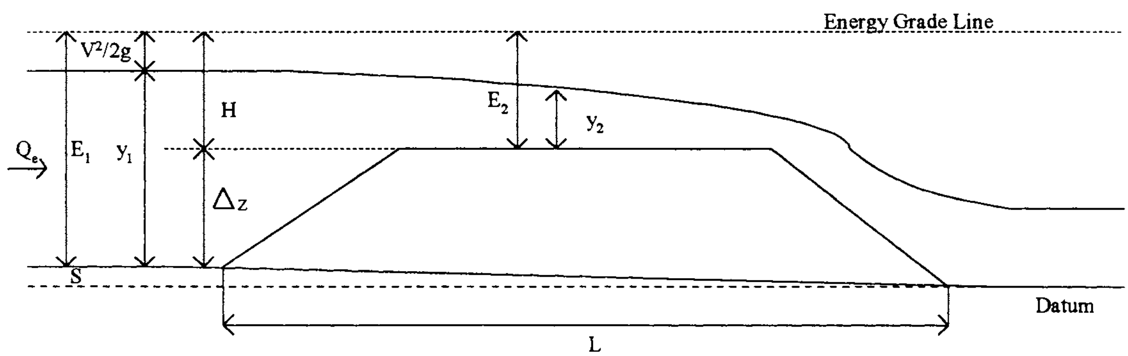

Figure 13. Diagram for a raised unvented ford

If a raised unvented ford is going to be designed, as shown in Figure13, the overtopping flow depth must be determined differently. In order to find y_2, the overtopping depth, the energy equation must be used. The general energy equation from Gupta (2001), is

$$y_1 + \frac{v_1^2}{2g} + z_1 = y_2 + \frac{v_2^2}{2g} + z_2 + h_f \tag{9}$$

Assuming no losses, $h_f = 0$, the general energy equation can be modified with substitutions to get the following equation:

$$y_1 + SL + \left(\frac{Q_1^2}{2gw^2 y_1^2}\right) = y_2 + \left(\frac{Q_2^2}{2gw^2 y_2^2}\right) + \Delta z \tag{10}$$

where y_1 is the upstream flow depth or headwater depth, which is calculated using Equations (7) or (8), y_2 is the overtopping flow depth, Q_1 and Q_2 are the upstream and downstream flows respectively, which are assumed to be equal, w is the width of the stream, and Δz is the height of the raised LWSC. In this equation we can assume that the slope is small so the SL term of the equation can be neglected. Rearranging this equation we get

$$\frac{1}{y_2^2} = \frac{2gw^2}{Q_e^2}\left[y_1 + \left(\frac{Q_e^2}{2gw^2 y_1^2}\right) - \Delta z - y_2\right] \tag{11}$$

and

$$1 = \left[\frac{2gw^2}{Q_e^2}\left(y_1 + \left(\frac{Q_e^2}{2gw^2 y_1^2}\right) - \Delta z\right) y_2^2\right] - \left[\frac{2gw^2 y_2^3}{Q_e^2}\right] \tag{12}$$

Equation (12) is solved for y_2, the overtopping flow depth, based on design flow, Q_e, headwater depth, y_1, raised LWSC height, Δz, and the width of the stream, using a mathematical equation solver or trial and error methods.

All of the modified Manning's equations listed above were used to analyze natural stream conditions including a wide range of flow discharges, stream slopes, and channel widths, while assuming Manning's n to be 0.04 for natural channels. The results from using the varying parameters were used to find a relationship between the change in structure height, Δz, and the change in overtopping depth as a result of raising the streambed for a LWSC.

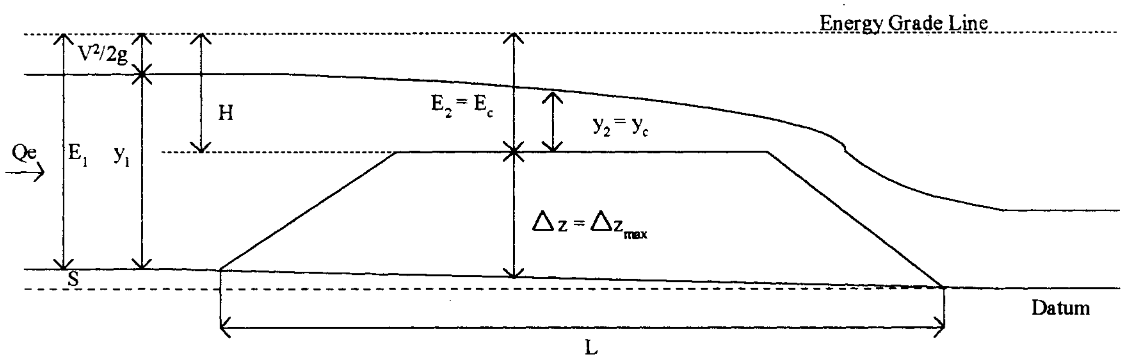

Figure 14. Diagram for a raised unvented ford with a height of Δz_{max}

The first analysis was for the conditions when a structure height is equal to Δz_{max}, thus creating a state of critical flow over the crossing where $y_2 = y_c$ and $E_2 = E_c$, as shown in Figure 14. Under critical flow several relationships have been derived, assuming a rectangular channel (Chaudhry, 1993; Modi and Seth, 1991). One important equation is

$$y_c = \left(\frac{q^2}{g}\right)^{\frac{1}{3}} \tag{13}$$

where y_c is the critical flow depth and $q = Q_e/w$. Using this as a design parameter helps to simplify the hydraulic design process. For example, a design flow from the hydrologic analysis can be used to find a critical flow depth, y_c, using Equation (13). If the critical flow value exceeds the maximum allowable overtopping depth, then an unvented ford should not be considered for the given situation. If the critical flow depth is less than or equal to the maximum allowable flow depth, and a raised unvented ford is desired, then Δz_{max} should be determined. Equations (4) and (10) can be used to find Δz_{max} as shown in the following:

$$\Delta z_{\max} = E_1 - E_c \tag{14}$$

or

$$\Delta z_{\max} = \left(y_1 + \frac{Q^2}{2gA_1^{\ 2}} \right) - \left(y_c + \frac{Q^2}{2gA_2^{\ 2}} \right) \tag{15}$$

where $A_1 = y_1 w$, and $A_2 = y_c w$, and Q is assumed to be constant.

As described by Chaudhry (1993), $y_2/E_2 = 2/3$ when Δz is maximum and critical conditions exist. When an assumption is made that total head remains constant, $E_2 = H$ as shown in Figure 14. Substitutions can be made so that under these assumptions:

$$\frac{y_c}{H} = \frac{2}{3} \tag{16}$$

An analysis was performed to develop a relationship between the ratio y_2/H and Δz when the raised structure height is less than the maximum value, Δz_{max}. In this investigation, data for natural stream conditions under subcritical flow were used. These data include: mild to steep channel slopes (0.002- to 0.2-ft/ft), low to high values of stream discharge (1.5- to 3000-cfs), and narrow to wide streams (5– to 50-ft).

Assuming that the subcritical design flows, slopes, and widths of the streams where known, the overtopping depths for different values of Δz were calculated using Equation (12). This analysis showed that as the height of the crossing, Δz, is increased from 0-ft to the maximum Δz for the design flow, the ratio of y_2/H varied from 1.0 to 0.667. These findings agree with the concept that under subcritical flow conditions overtopping flow depth gradually decreases as the height of the step on the streambed increases. Results for the ratio y_2/H, based on hydraulic principles, are presented in Table 7, column 3.

5.1.3. Examination of Laboratory Results

The overtopping flow depth on a raised unvented ford, y_2 as shown in Figure 13, can be computed with an empirical equation for a broad crested weir (Rossmiller et al., 1984). Laboratory flume experiments conducted by Barrett (1984) resulted in a modified broad crested weir equation that can be used for LWSC analysis. The modified equation is as follows:

$$H = 0.389\ Q_e^{0.599} L_o^{-0.493} \tag{17}$$

The laboratory investigation also provided results that led Barrett (1984) to make an assumption that the overtopping flow depth could be approximated using:

$$y_2 = 0.6H \tag{18}$$

Combining Equations (17) and (18),

$$y_2 = 0.233\ Q_e^{0.599} L_o^{-0.493} \tag{19}$$

where L_o is the length of LWSC perpendicular to flow, and Q_e is design discharge from hydrological analysis. Equation (19) can be used to calculate the depth of water over the raised ford for a given design discharge and length of LWSC, which is equal to the width of the stream. Once y_2 is calculated, it can be compared with the acceptable overtopping flow depth. The laboratory experiments demonstrated that LWSC height, Δz, does not significantly affect the discharge-depth relation. Therefore, it was assumed that Δz is a flexible design parameter.

Based on the results from the laboratory experiments, Barrett (1984) and Rossmiller et al. (1984), have concluded that the change in Δz does not have significant impact on discharge-depth relations and that the ratio y_2/H is a constant (0.6). To examine these findings, an analysis was conducted using the concepts discussed in section 5.1.1. flow regimes and overtopping flow depth and section 5.1.2. hydraulic principles.

The analysis was performed with natural stream data under subcritical flow conditions, similar to what were used for the energy equation analysis. With a given design flow, slope, and stream width, H could be calculated using Equation (17), developed by Barrett (1984). The overtopping depth, y_2, was calculated with the same stream properties using methods discussed in section 5.1.2. Once H and y_2 were calculated for a variety of stream conditions, comparisons could be made. Results for the ratio y_2/H, based on equations developed by Barrett (1984), are presented in Table 7, column 4. These results show that under natural conditions the ratio y_2/H had a variety of values ranging from less than 0.6 to values greater than 2.0, depending on the magnitude of stream flow and the height of the raised crossing, Δz. The findings are different than results from the analyses using hydraulic principles.

The discrepancy between the laboratory findings and the analysis of natural streams using the hydraulic principles indicates that the lab experiments had limitations and the findings may only be applicable to lab conditions, not natural streams. The headwater remains constant in a natural river. In conducting a study using a laboratory flume, with limited length, it is a challenge to recreate natural stream conditions. In this case, it would have been difficult to control the headwater depth. Due to the short length of a flume, backwater elevation in response to a raise in the step height on the channel bottom was difficult to keep constant. The changing headwater depths as a result of changing ford heights would make the laboratory findings irrelevant for natural streams.

The equations developed by Barrett (1984) where analyzed for critical conditions. In the investigation the LWSC height was Δz_{max} and the overtopping flow was equal to the critical flow depth. Using the critical condition parameters, a pattern of variation was found for the ratio y_2/H.

Table 8 shows the different values for y_2/H, when the crossing height is Δz_{max}, based on variations in stream flow and channel properties. In this table, the low range y_2/H ratios for each flow magnitude, Q_e, are associated with narrow streams. The high range ratio values are for streams with wide channels under the same conditions. The slope of the stream had very little impact on the differences in value. As shown in the table, $y_2/H = 0.6$ under specific conditions when Δz is maximum. Therefore, assumptions made in research by Barrett (1984) are applicable in certain circumstances.

Table 7. Change in y_2/H for $\Delta z < \Delta z_{max}$

1	2	3	4
Constants	**Δz (ft)**	**Hydraulic Principles y_2/H**	**Empirical Equation y_2/H**
Q=25 cfs	0.70	0.77	0.83
n=0.04	0.60	0.86	1.01
w=8 ft	0.50	0.90	1.16
S=0.002	0.30	0.94	1.41
Δz_{max}=0.73 ft	0.10	0.96	1.64
Q=25 cfs	0.34	0.73	0.66
n=0.04	0.25	0.84	0.90
w=20 ft	0.15	0.93	1.15
S=0.002	0.10	0.94	1.41
Δz_{max}=0.35 ft	0.05	0.96	1.34
Q=25 cfs	0.23	0.73	0.59
n=0.04	0.20	0.84	0.74
w=40 ft	0.16	0.91	0.88
S=0.002	0.12	0.94	0.99
Δz_{max}=0.24 ft	0.08	0.94	1.08
Q=150 cfs	2.70	0.84	1.09
n=0.04	2.50	0.88	1.97
w=8 ft	2.00	0.92	1.43
S=0.002	1.00	0.96	1.84
Δz_{max}=3.02 ft	0.50	0.97	2.01
Q=150 cfs	0.95	0.79	0.84
n=0.04	0.70	0.89	1.06
w=20 ft	0.50	0.92	1.20
S=0.002	0.30	0.94	1.23
Δz_{max}=1.03 ft	0.10	0.95	1.46
Q=150 cfs	0.61	0.72	0.65
n=0.04	0.50	0.84	0.84
w=40 ft	0.40	0.89	0.95
S=0.002	0.30	0.92	1.06
Δz_{max}=0.62 ft	0.20	0.94	1.15
Q=300 cfs	5.00	0.88	1.28
n=0.04	4.00	0.93	1.57
w=8 ft	2.00	0.97	2.09
S=0.002	1.00	0.97	2.33
Δz_{max}=5.91 ft	0.50	0.98	2.47

Table 7. (Continued)

1	2	3	4
Constants	**Δz (ft)**	**Hydraulic Principles y_2/H**	**Empirical Equation y_2/H**
Q=300 cfs	1.50	0.80	0.89
n=0.04	1.20	0.88	1.07
w=20 ft	0.90	0.92	1.22
S=0.002	0.60	0.93	1.35
Δz_{max}=1.64 ft	0.30	0.95	1.48
Q=300 cfs	0.90	0.72	0.68
n=0.04	0.70	0.86	0.90
w=40 ft	0.50	0.90	1.04
S=0.002	0.30	0.93	1.17
Δz_{max}=0.91 ft	0.10	0.95	1.30
Q=1500 cfs	6.00	0.72	0.86
n=0.04	4.00	0.91	1.33
w=20 ft	3.00	0.93	1.51
S=0.002	2.00	0.95	1.67
Δz_{max}=6.08 ft	1.00	0.96	1.82
Q=1500 cfs	2.40	0.74	0.79
n=0.04	2.00	0.84	0.95
w=40 ft	1.50	0.89	1.10
S=0.002	1.00	0.92	1.23
Δz_{max}=2.48 ft	0.50	0.94	1.35
Q=150 cfs	0.50	0.73	0.87
n=0.04	0.40	0.79	0.97
w=8 ft	0.30	0.82	1.04
S=0.01	0.20	0.85	1.10
Δz_{max}=0.53 ft	0.10	0.87	1.15
Q=150 cfs	0.14	0.71	0.72
n=0.04	0.12	0.75	0.77
w=20 ft	0.08	0.79	0.83
S=0.01	0.06	0.80	0.85
Δz_{max}=0.15 ft	0.04	0.83	0.88
Q=300 cfs	1.00	0.75	0.95
n=0.04	0.80	0.81	1.06
w=8 ft	0.60	0.84	1.14
S=0.01	0.40	0.86	1.21
Δz_{max}=1.10 ft	0.20	0.88	1.28

Table 7. (Continued)

1	2	3	4
Constants	**Δz (ft)**	**Hydraulic Principles y_2/H**	**Empirical Equation y_2/H**
Q=300 cfs	0.20	0.70	0.75
n=0.04	0.15	0.76	0.82
w=20 ft	0.10	0.79	0.87
S=0.01	0.05	0.81	0.90
Δz_{max}=0.21 ft	0.01	0.82	0.93
Q=1500 cfs	7.90	0.70	0.98
n=0.04	7.00	0.81	1.20
w=8 ft	5.00	0.89	1.47
S=0.01	3.00	0.93	1.70
Δz_{max}=8.00 ft	1.00	0.95	1.90
Q=1500 cfs	0.74	0.67	0.79
n=0.04	0.50	0.77	0.94
w=20 ft	0.30	0.80	1.00
S=0.01	0.10	0.83	1.05
Δz_{max}=0.75 ft	0.05	0.84	1.06
Q=1500 cfs	0.21	0.68	0.71
n=0.04	0.17	0.73	0.77
w=40 ft	0.13	0.75	0.79
S=0.01	0.10	0.76	0.81
Δz_{max}=0.22 ft	0.07	0.77	0.83
Q=3000 cfs	1.60	0.69	0.86
n=0.04	1.40	0.75	0.94
w=20 ft	1.20	0.78	0.99
S=0.01	1.00	0.80	1.03
Δz_{max}=1.65 ft	0.80	0.82	1.07
Q=3000 cfs	0.30	0.69	0.76
n=0.04	0.25	0.72	0.80
w=40 ft	0.20	0.74	0.83
S=0.01	0.15	0.76	0.85
Δz_{max}=0.33 ft	0.10	0.77	0.87

Table 8. Values for y_2/H when Δz is maximum

Q_e	y_c	Range for y_2/H
low flow	yc < 0.5-ft	0.45 - 0.60
LWSC design flow	yc = 0.5-ft	0.55 - 0.65
high flow	yc > 0.5-ft	0.60 - 0.75
extreme flow	yc >> 0.5-ft	0.80 - 0.95

5.1.4. Unvented Ford Design Procedure

The first step in designing an unvented ford requires estimation of the design flow, Q_e, as described in section 4. Next, flow regime should be analyzed to determine if the stream is under subcritical or supercritical flow conditions for the design flow. If the stream flow is subcritical:

1. Find the headwater depth, y_1, for the design flow using Equation (7) or (8).
    a. If $y_1 \leq$ 6-in, the unvented ford can be constructed on the streambed.
    b. If $y_1 >$ 6-in, a raised unvented ford should be considered.
2. For a raised unvented ford calculate the critical flow depth, y_c, using Equation (13).
    a. If $y_c \leq$ 6-in, the height of the crossing should be raised to Δz_{max}.
    b. If $y_c >$ 6-in, do not use an unvented ford and try considering a vented ford.
3. For a raised unvented ford calculate Δz_{max} using Equation (15).
4. The overtopping depth on a raised unvented ford is checked using Equation (12) or Equation (19) when the structure height is Δz_{max}.
5. The flow velocity at the crossing is calculated from the following equation:

$$V_2 = \frac{Q}{y_2 L} \tag{20}$$

where V_2 = flow velocity at the crossing, Q = stream discharge, y_2= overtopping flow depth, and L = length of overflow section.

If the stream flow is supercritical:

1. Find the headwater depth, y_1, for the design flow using Equation (7) or (8).
   a. If $y_1 \leq$ 4-in, the unvented ford can be constructed on the streambed.
   b. If $y_1 >$ 4-in, do not use an unvented ford and consider a vented ford.
2. The exit velocity, V_2, can be calculated using Equation (20).

5.2. Vented Fords

Vented fords should be considered when the design flow overtopping depth for an unvented or raised unvented ford exceeds the maximum allowable overflow depth. Vented fords are more appropriate for these situations because pipes built into the structure permit stream flow to pass through the structure. This allows them to handle larger stream discharges while meeting maximum allowable overtopping depth criteria.

5.2.1. Design Discharge

The design flow for a vented ford is a combination of culvert flow and overtopping flow at the structure. The pipe in vented fords is designed to have a flow capacity of Q_v such that:

$$Q_v = Q_e - Q_{top} \tag{21}$$

where, Q_e is the total design flow from hydrological analysis, and Q_{top} is the flow over the ford. Overtopping should be 0 to 6-in, but not exceed this level for the design flow. The following information will discuss two different methods for determining the overtopping flow depth, Q_{top}, so that the culvert design flow can be estimated and pipe size chosen for the structure.

5.2.2. Modified Broad Crested Weir Equation

Flow over a vented ford can be calculated from rearranging Equation (17),

$$Q_{top} = 4.83\ L_o^{0.823} H^{1.67} \tag{22}$$

Considering $H = y_2/0.6$, when the pipe cover is raised to Δz_{max}, and assuming a maximum allowable water depth, y_2, of 6-in over the ford, H becomes 0.833-ft and Equation (22) can be rearranged as

$$Q_{top} = 3.538\ L_o^{0.823} \tag{23}$$

where the overtopping flow, Q_{top}, can be estimated when the length of structure is known.

5.2.3. Critical Flow Equation

Using the relationships determined from Equations (9) and (13), and combining the equations can give a new expression that can be used to determine overtopping depth, Q_{top}. The equation is given as:

$$1.5y_c = y_c + \left(\frac{Q_{top}^{\ 2}}{2gw^2 y_c^{\ 2}} \right) \tag{24}$$

or
$$Q_{top} = \sqrt{y_c^{\ 3} g w^2} \tag{25}$$

when overtopping flow depth is equal to the design flow critical depth in the situation where the height of the pipe cover is Δz_{max}.

When comparing the value of Q_{top} determined using the empirical equation (Barrett 1984) to the Q_{top} estimated using the critical flow equation there is no significant difference between the values of calculated flows, if Δz is maximum. This means that when 6-in is used as the overtopping flow depth, and the pipe cover, C_o, is equal to Δz_{max}, either equation can be used effectively to obtain a value for the overtopping flow. It should be noted that culverts need a minimum cover of 1-ft to prevent loading damage to pipes (Lohnes et al., 2001). Therefore, at least 1-ft of cover should be used for a vented ford, even if Δz_{max} is less than 1-ft.

5.2.4. Analysis of Flow Conditions

The hydraulic design of a vented ford is similar to that of a culvert. In culvert hydraulics and flow equation derivation and analysis (Normann et al., 1985, Herr and Bossy 1965; Haestad Methods 1999; Gupta 2001), flow conditions of a culvert are usually divided into two categories: inlet control and outlet control. The entrance of a culvert can be above the water or submerged. When the inlet is submerged, the pipe is partially full under inlet control while the barrel is completely full under outlet control (Gupta 2001). This may imply that a larger size is required for a culvert that is operating under inlet control as compared with outlet control.

In a vented ford design, determination of number and size of pipes in a vented ford design is a trial and error process. In the design approach used by previous investigators, it was first assumed that the flow is governed by inlet control and then the design is checked for outlet control. When the inlet of a culvert is submerged, a larger size may be required under inlet control. Therefore, the design of a vented ford with a submerged entrance for inlet control flow may not need to be checked for outlet control; and the design procedure can be significantly simplified. An analysis is performed in this study to examine the validity of the assumption that inlet control flow condition requires a larger size of culvert barrel.

Inlet Control Hydraulics

Inlet control means that the discharge capacity of a culvert pipe is controlled at the entrance by headwater depth, y_1, and entrance geometry, including barrel shape and cross-sectional area, and the type of inlet edges. Under the assumption of inlet control, culvert barrel friction and other minor losses can be neglected. The practical significance of inlet control is that flow capacity of a culvert can be increased by improving entrance condition. When the inlet is submerged, the pipe is partially full under inlet control.

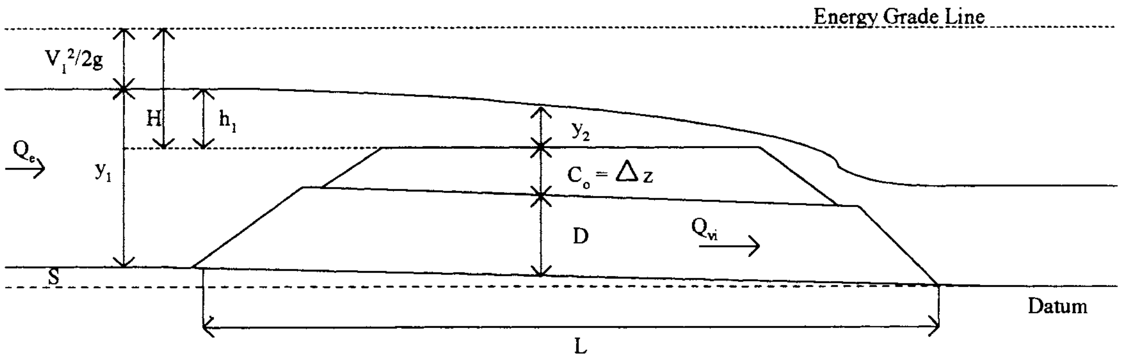

Figure 15. Diagram for a vented ford under inlet control

An example of a vented ford under inlet control is shown in Figure 15. Assuming the culvert has a submerged entrance, an equation for inlet control can be shown as (Haestad, 1999):

$$\frac{y_1}{D} = C\left[\frac{Q_{vi}}{A(D^{0.5})}\right]^2 + Y + f_s S \tag{26}$$

or

$$Q_{vi} = \frac{\pi}{4}\left(D^{2.5}\right)\left[\frac{\frac{y_1}{D} - Y - f_s S}{C}\right]^{0.5} \tag{27}$$

where Q_{vi} is the design flow for culvert under inlet control, D is the diameter of the culvert, y_1 is the depth from the inlet invert up to the water surface, or headwater depth, C and Y are constants that can be found in Haestad (1999) which are listed in Table B1 of Appendix B, f_s is the slope correction factor where $f_s = +0.7$ for mitered entrance and $f_s = -0.5$ for other entrance types, and S is the slope of the culvert. This equation can be used to develop D vs. Q_{vi} plots for inlet control.

In order to solve Equation (27), the headwater has to be estimated using the following equation:

$$y_1 = D + C_o + h_1 \tag{28}$$

where C_o is the cover over the pipe and h_1 is the overflow depth before the inlet. It can be assumed that velocity head is neglected in most circumstances because it is a relatively small component of the total head, therefore $h_1 \cong H$, and H can be substituted into Equation (28). The value of H can be estimated from Barrett's research, when pipe cover is equal to Δz_{max}, using:

$$H = \frac{y_2}{0.6} \tag{29}$$

where H is the upstream head above the raised structure and y_2 is the overflow depth over the crossing. When 6-in is used for the design overflow depth, Equation (28) can be solved for the required headwater depth for inlet control. Once that is calculated, Equation (27) is used to develop a relationship between D and Q_{vi} for vented fords with inlet control.

In situations where there are very shallow and narrow streams, the velocity head may need to be included in the equation such that

$$y_1 = D + C_o + H - \left(\frac{V_1^2}{2g}\right) \tag{30}$$

Where g is force due to gravity and V_1 is the upstream velocity that can be estimated as

$$V_1 = \frac{Q_e}{A_1} \tag{31}$$

or

$$V_1 = \frac{Q_e}{(D + C_o + H)w} \tag{32}$$

Where Q_e is the design flow and w is the width of the stream.

Outlet Control Hydraulics

Under outlet control barrel friction is the predominant head loss. Tail water conditions also have an important effect on culverts with outlet control flow. The entrance of a culvert can be above the water or submerged. When the inlet is submerged the barrel is completely full under outlet control. This implies that a larger size may be required for a culvert that is operating under inlet control as compared with outlet control.

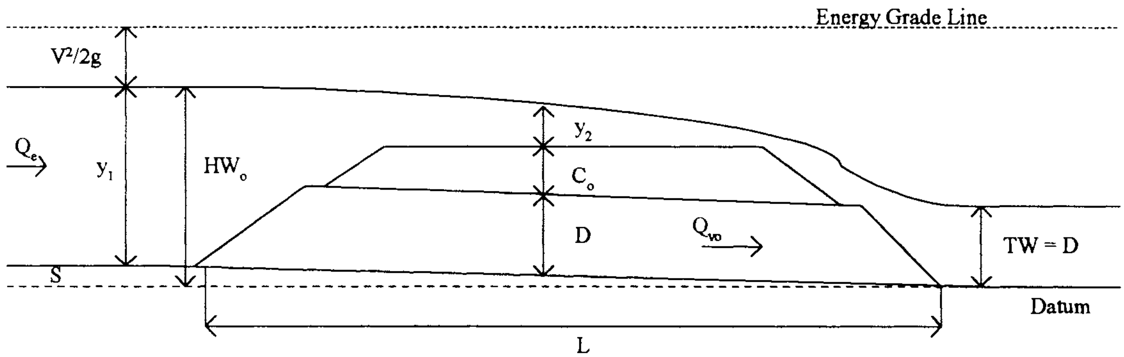

Figure 16. Diagram for a vented ford under outlet control

An example of outlet control is shown in Figure 16. With the assumptions that there is a submerged entrance, V_1 is equal to V_2, the tailwater (*TW*) depth is equal to D, the inlet approach distance is also equal to D, y_1 is the streambed up to the water surface, HW_o is the outlet invert up to the headwater surface, L is the culvert width, S is the culvert slope, n is Manning's roughness coefficient, and K_e is a factor found in Haestad (1999) which is listed in Table B2 in Appendix B, the following equation can be written for outlet control (Haestad, 1999; Gupta, 2001):

$$HW_o + \left(\frac{V_1^2}{2g}\right) = TW + \left(\frac{V_2^2}{2g}\right) + \left(\frac{K_e V_2^2}{2g}\right) + \left(\frac{n^2 V_1^2 (D+L)}{2.22\left(R^{\frac{4}{3}}\right)}\right) \tag{33}$$

Recognizing that

$$HW_o = y_1 + LS \tag{34}$$

and $$A_3 = \frac{D^2\pi}{4} \tag{35}$$

and $$R = \frac{A_3}{P} \tag{36}$$

where R is the hydraulic radius, A_3 is the cross sectional area, and P is the wetted perimeter, a new equation can be derived. Rearrangement of equations and substitution into Equation (33) gives the following:

$$Q_{vo} = \frac{(y_1 + LS - D)^{0.5}\left(\frac{D^2\pi}{4}\right)}{\left(\frac{K_e}{2g} + \left(\frac{n^2(D+L)}{2.22\left(\frac{D}{4}\right)^{\frac{4}{3}}}\right)\right)^{0.5}} \tag{37}$$

where Q_{vo} is the design flow for a culvert under outlet control. This equation is used to develop D vs. Q_{vo} plots for vented fords under outlet control flow condition.

5.2.5. Design Curves

Design curves (D vs. Q_v) are derived from culvert hydraulics and flow equations (Herr and Bossy, 1965; Normann et al., 1985, Haestad Methods, 1999; Gupta, 2001). These curves can be used to compare pipe sizing for inlet control and outlet control conditions. As mentioned earlier, Equations (27) and (37) can be used to find the relationship between culvert diameter and design flow for inlet and outlet control. Figures 17 through 28 show design curves generated in this research. These design curves represent inlet and outlet control for different types of submerged culverts placed at various slopes including mild, moderate, and steep sloping of the pipe.

After analyzing the design curves for different styles of pipe, it was concluded that inlet control conditions require larger pipe size the majority of the time. There were cases where outlet control required pipe size equal to or slightly larger than the inlet control requirement, but this only seemed to happen when the culvert slope is mild. The general trend shows that as slope of the pipe increases, culvert size for inlet control becomes much greater than culvert size required for outlet control for the same design flow. When outlet control pipe size was larger than the inlet control size requirement, the difference was small and negligible. Therefore, the assumption that inlet control requires a larger pipe is valid and culvert design for vented fords can be simplified, since outlet control does not have to be checked.

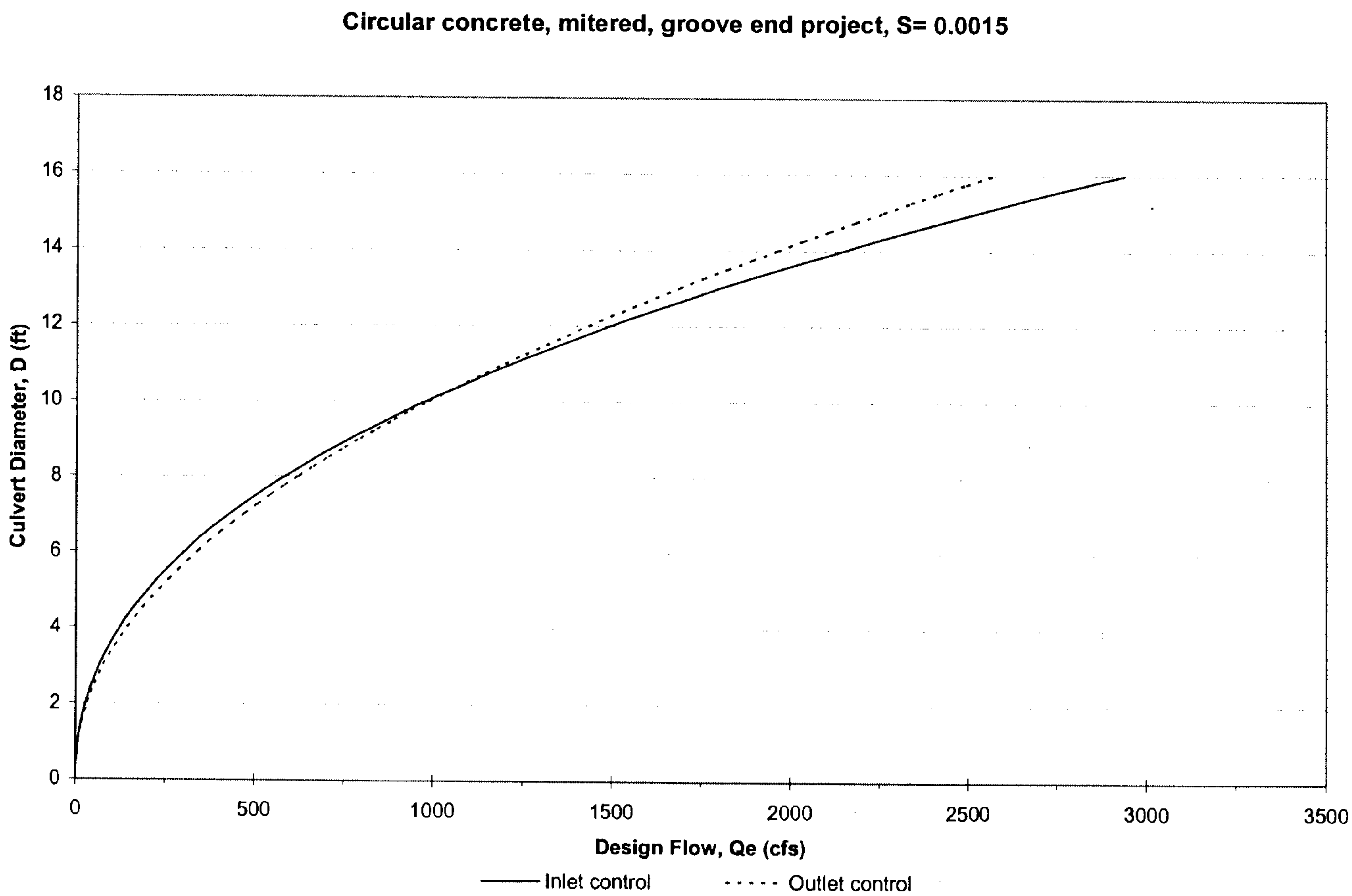

Figure 17. Circular concrete culvert design curves, S= 0.0015

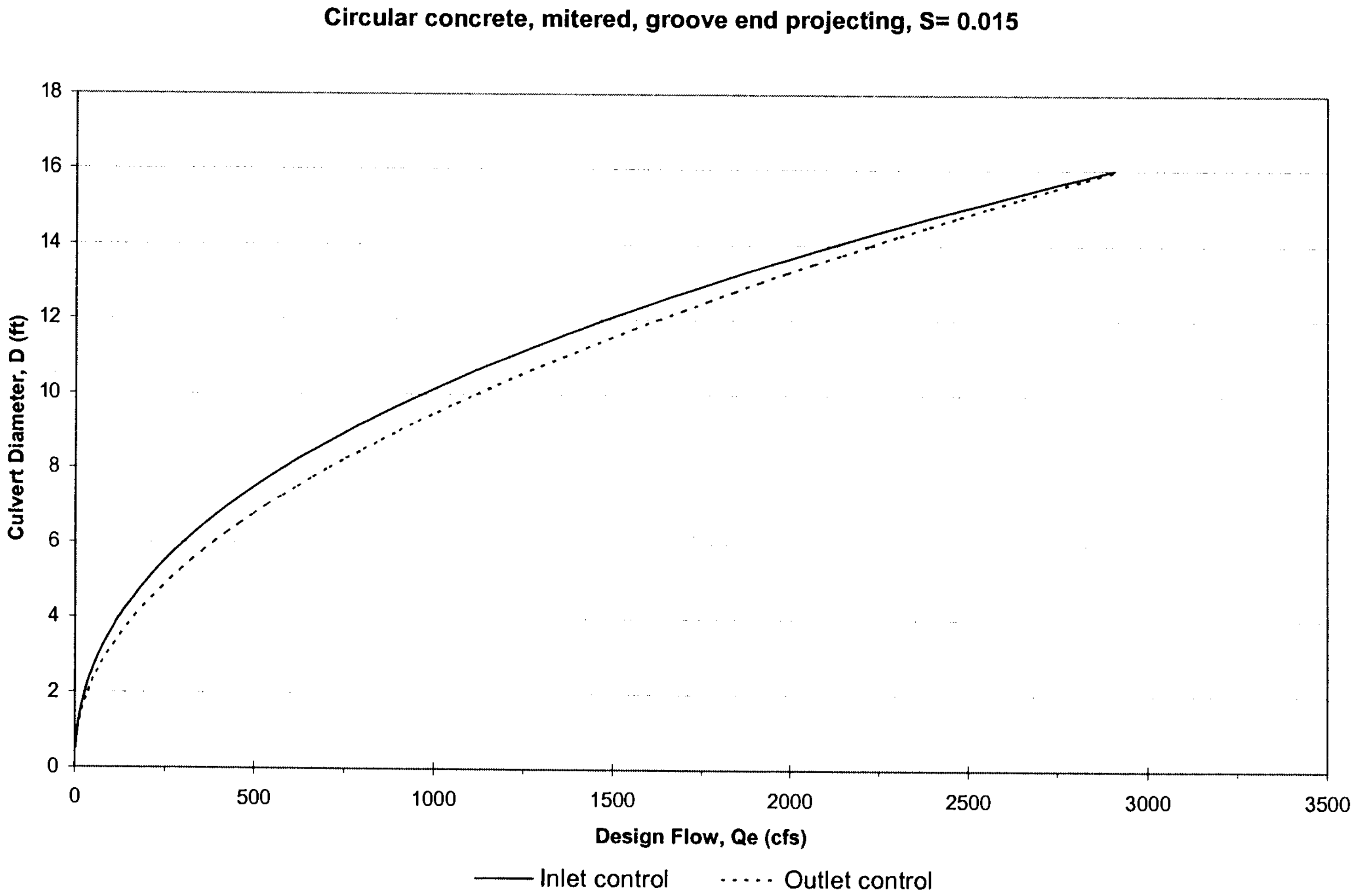

Figure 18. Circular concrete culvert design curves, S= 0.015

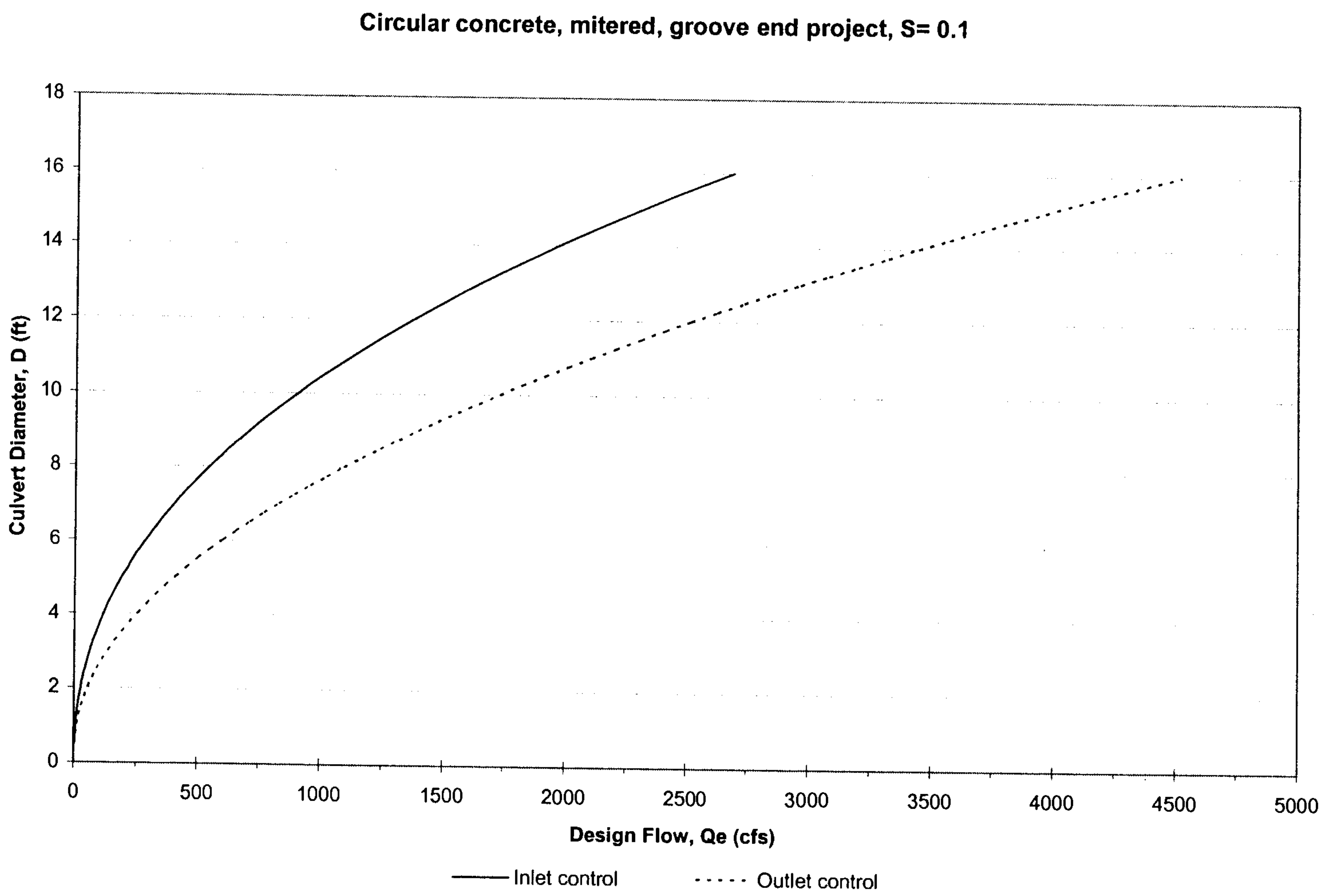

Figure 19. Circular concrete culvert design curves, S= 0.1

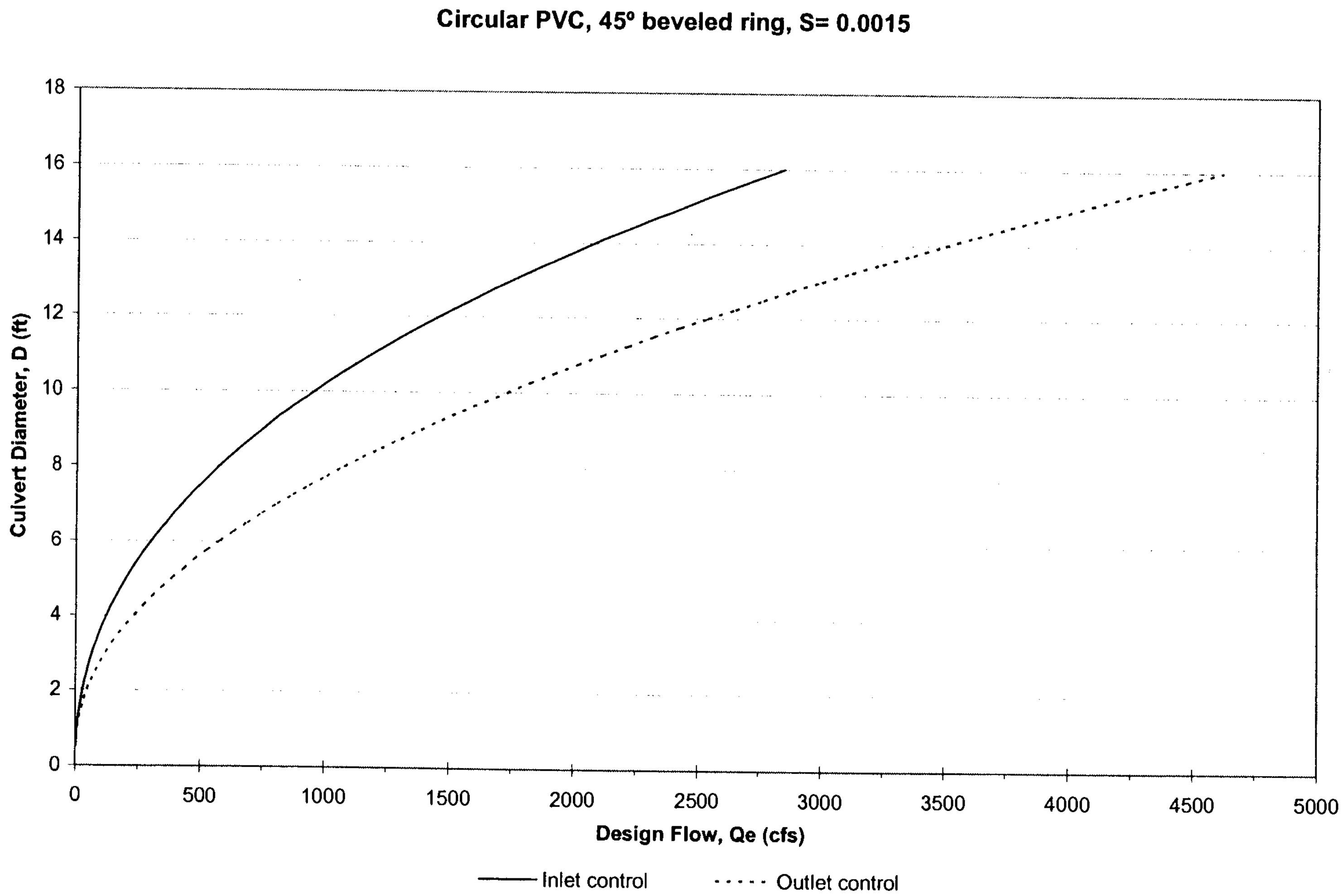

Figure 20. Circular Polyvinyl Chloride culvert design curves, S= 0.0015

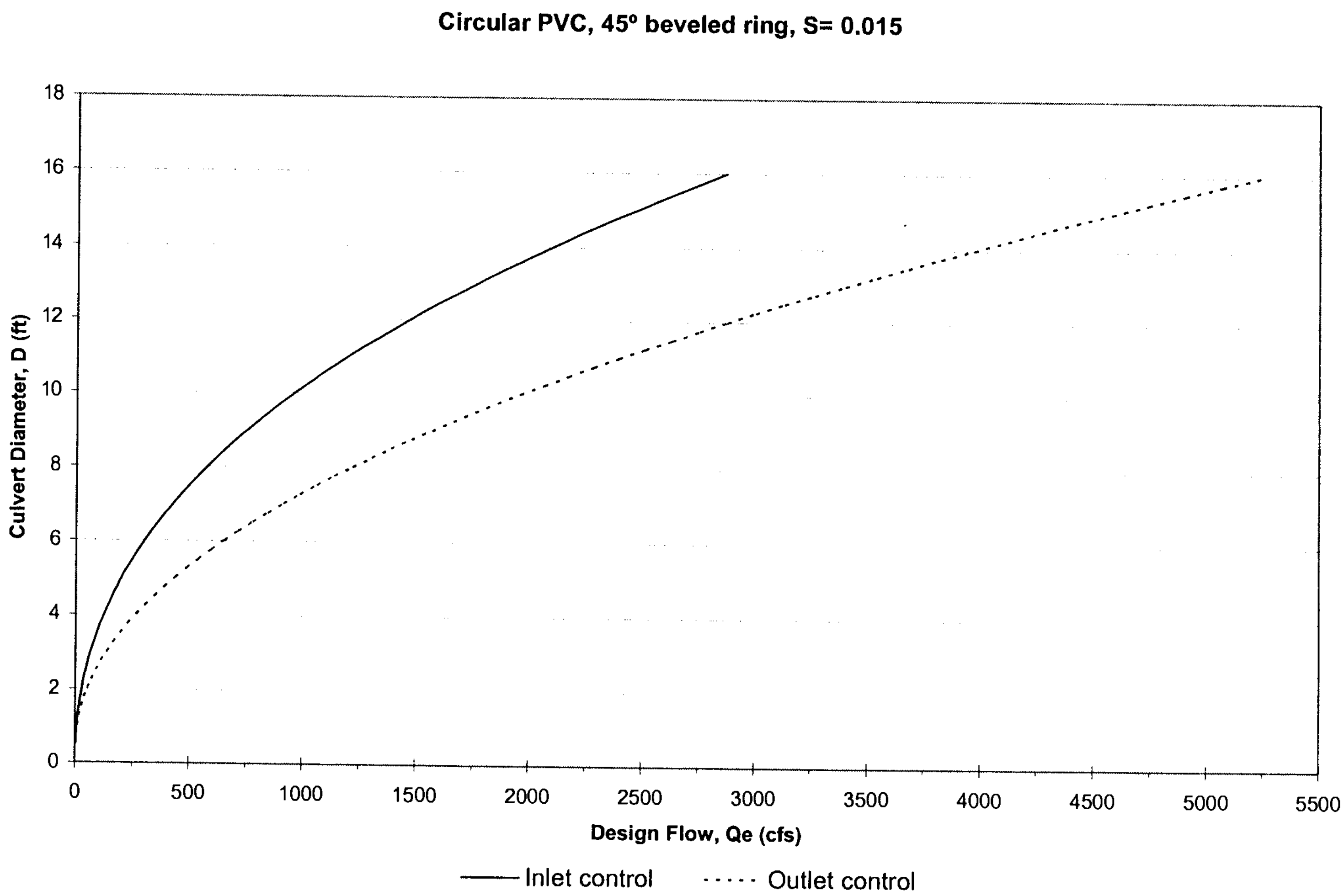

Figure 21. Circular Polyvinyl Chloride culvert design curves, S= 0.015

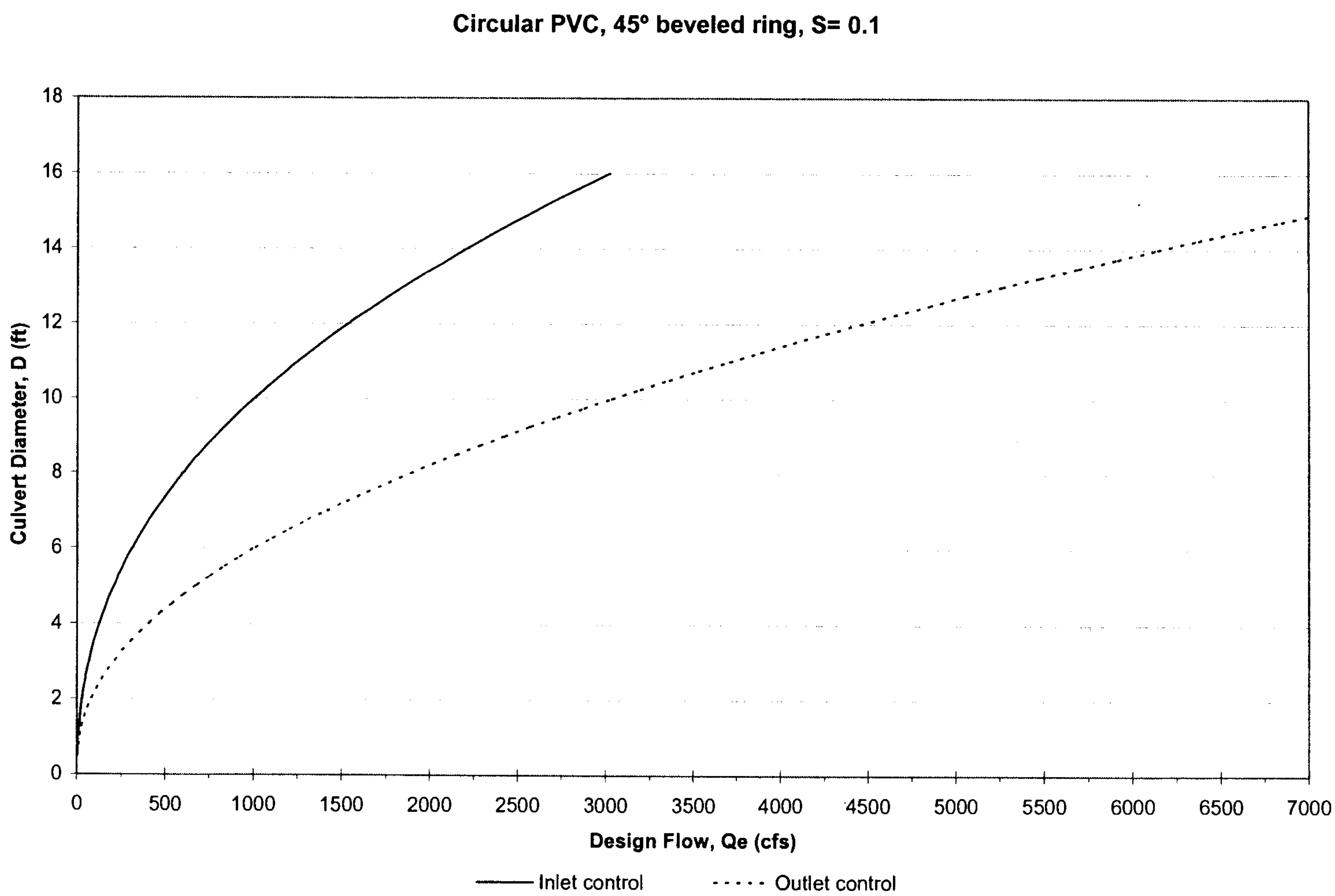

Figure 22. Circular Polyvinyl Chloride culvert design curves, S= 0.1

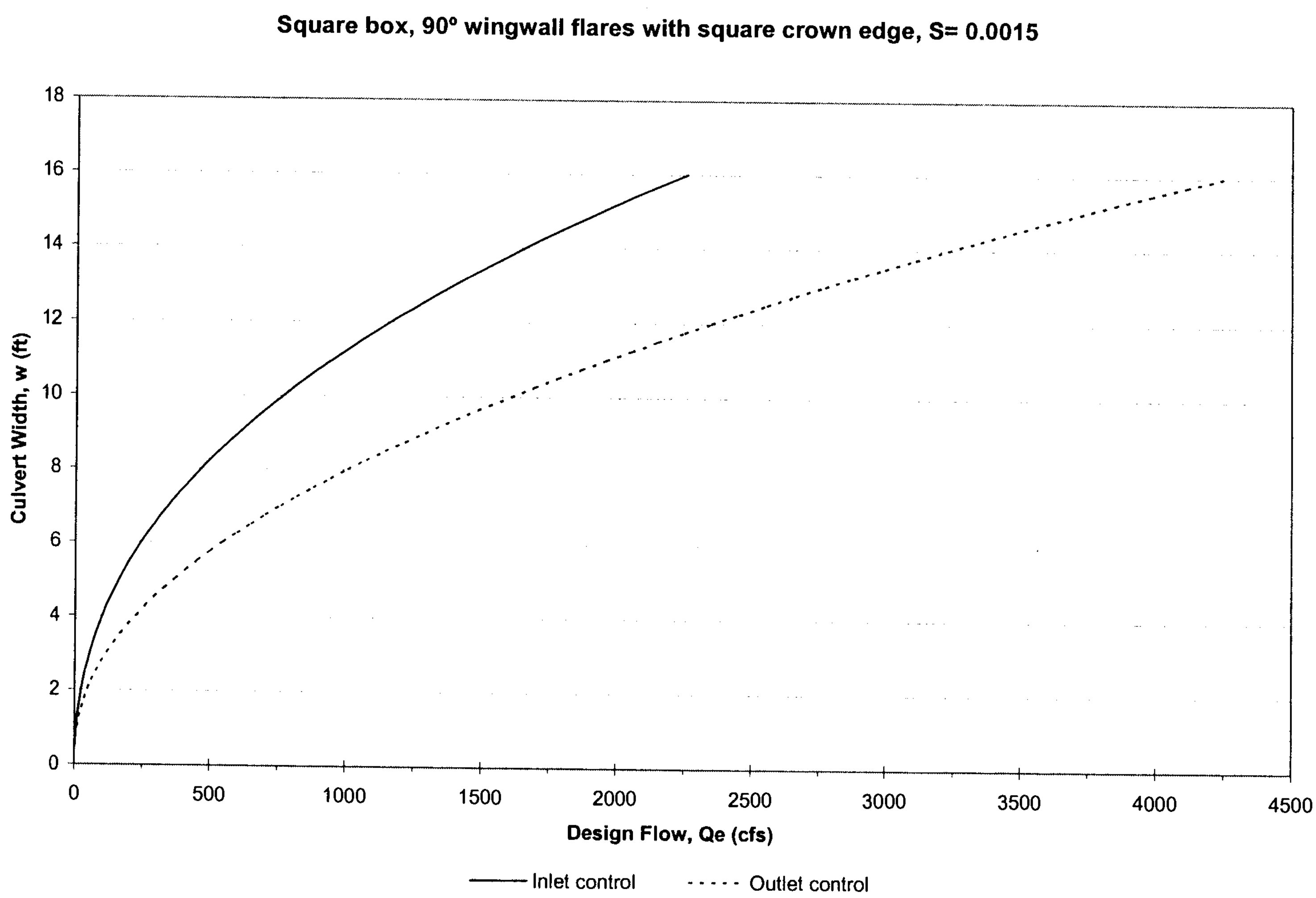

Figure 23. Square box culvert design curves, S= 0.0015

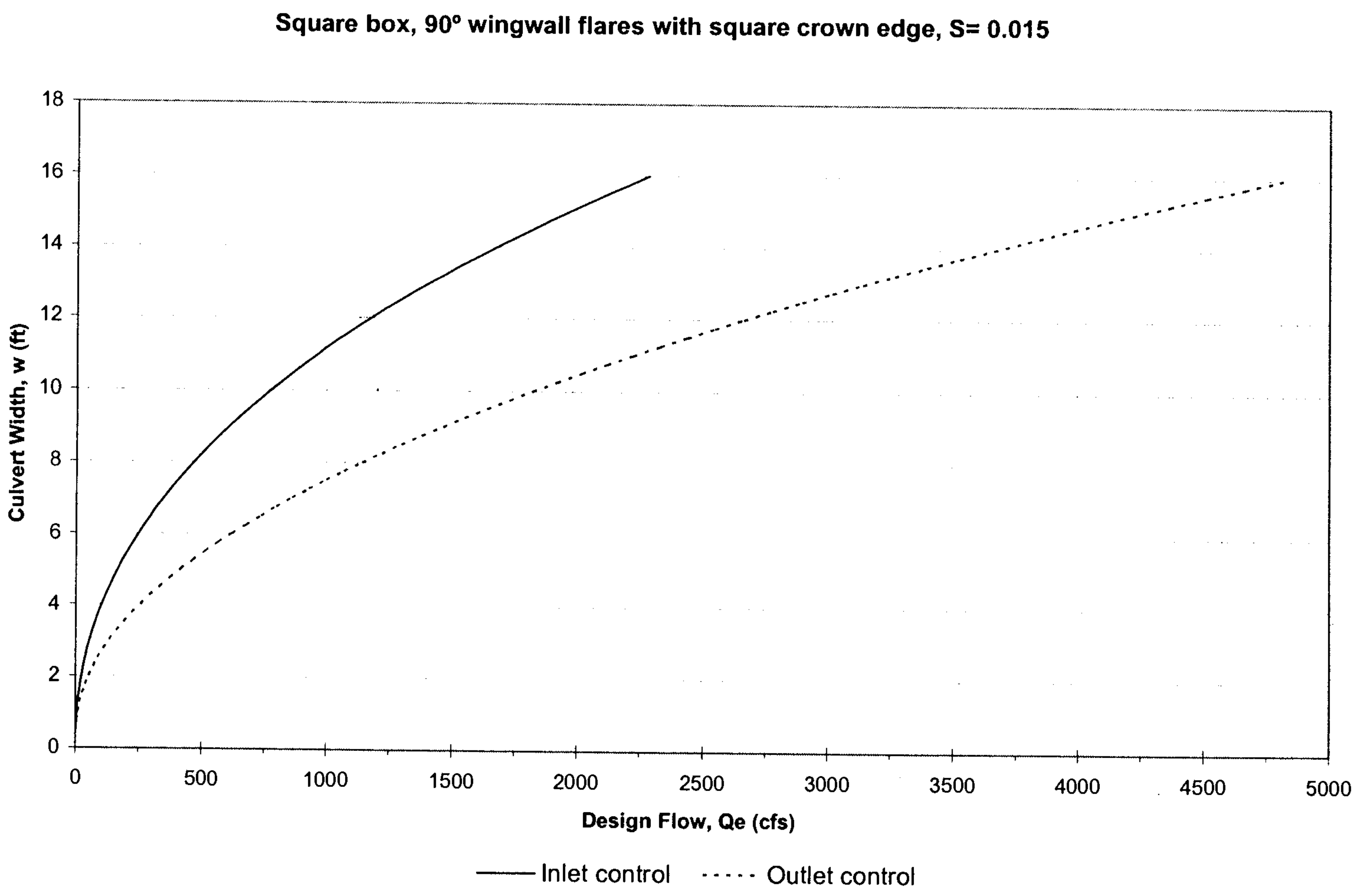

Figure 24. Square box culvert design curves, S= 0.015

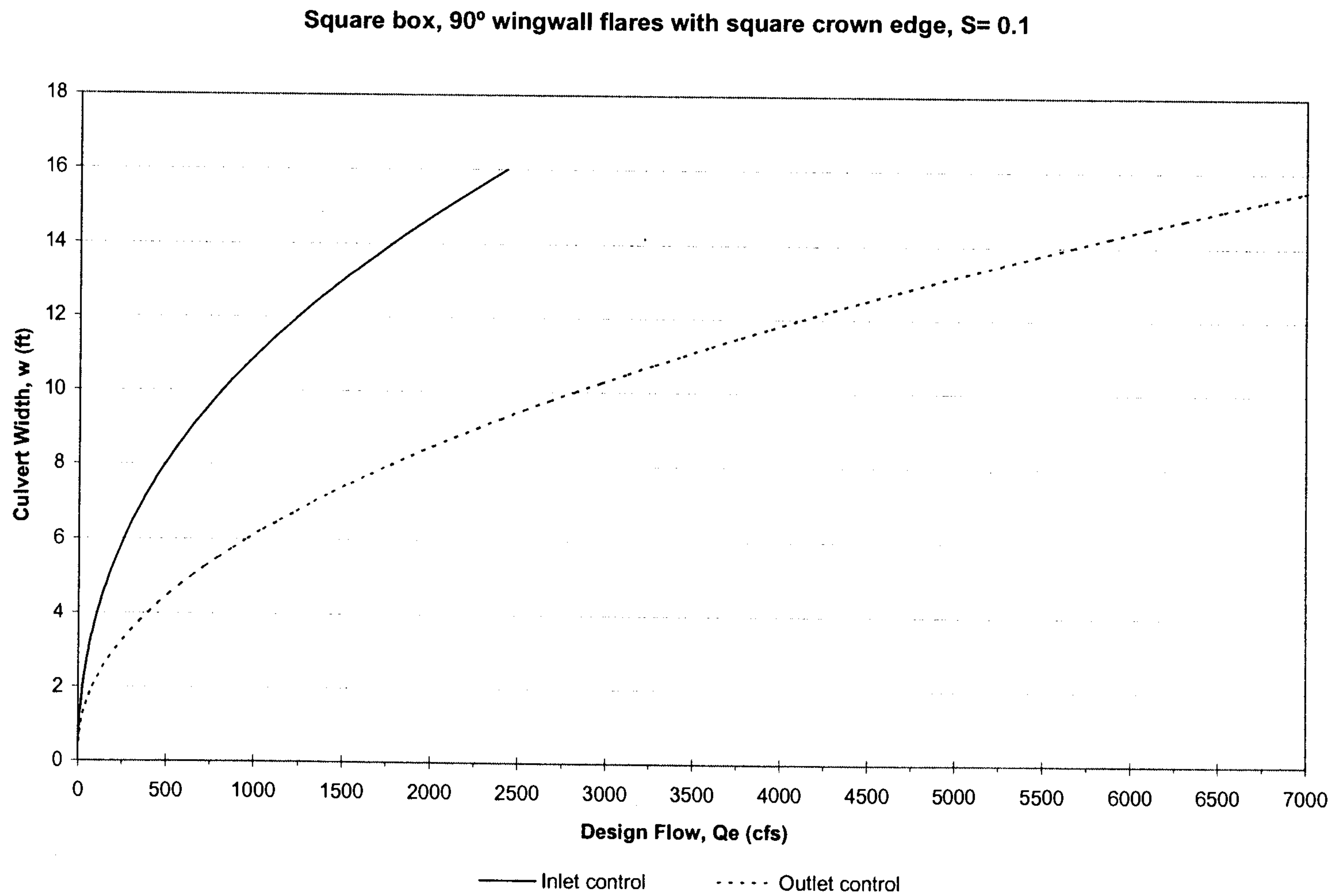

Figure 25. Square box culvert design curves, S= 0.1

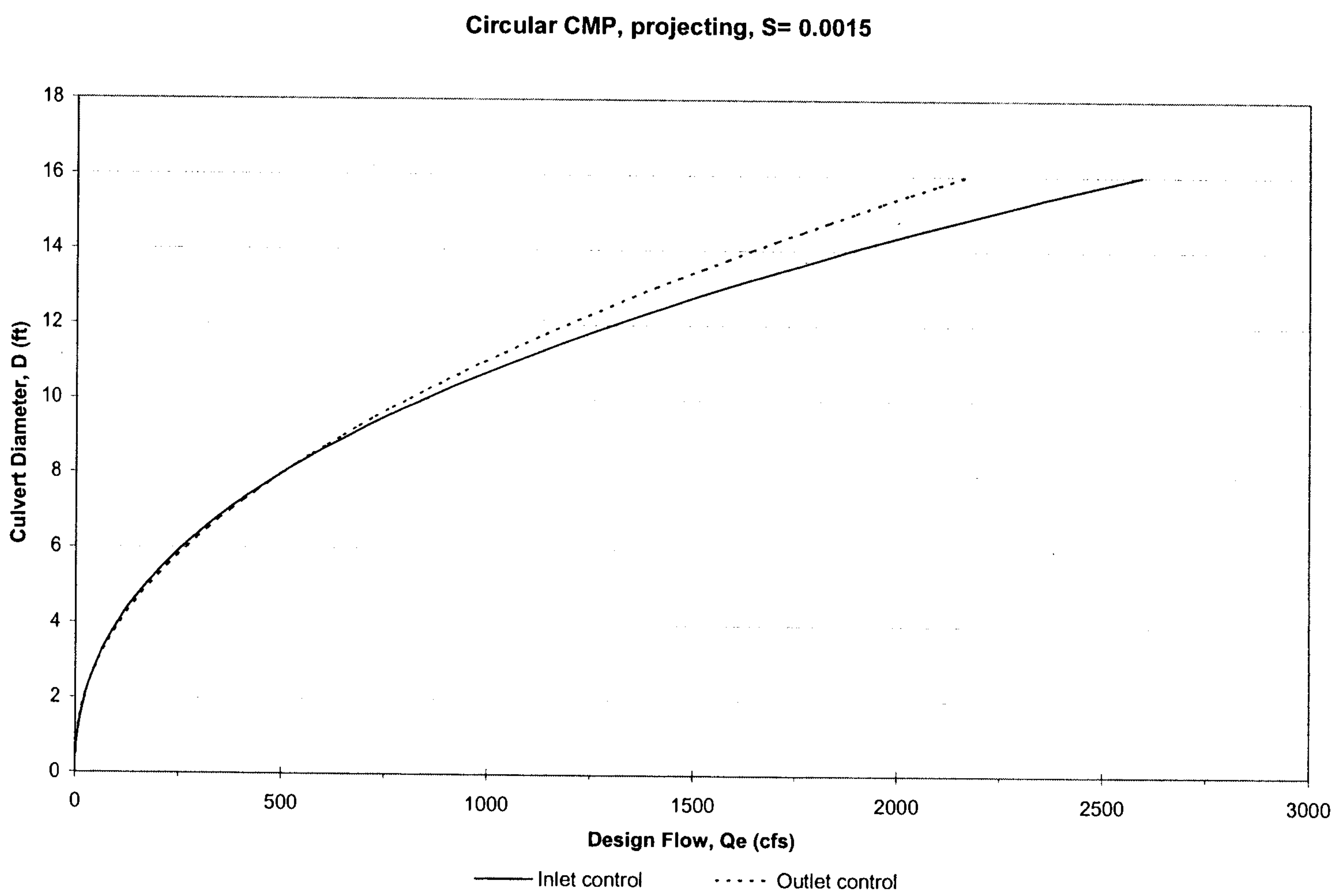

Figure 26. Circular corrugated metal pipe design curves, S= 0.0015

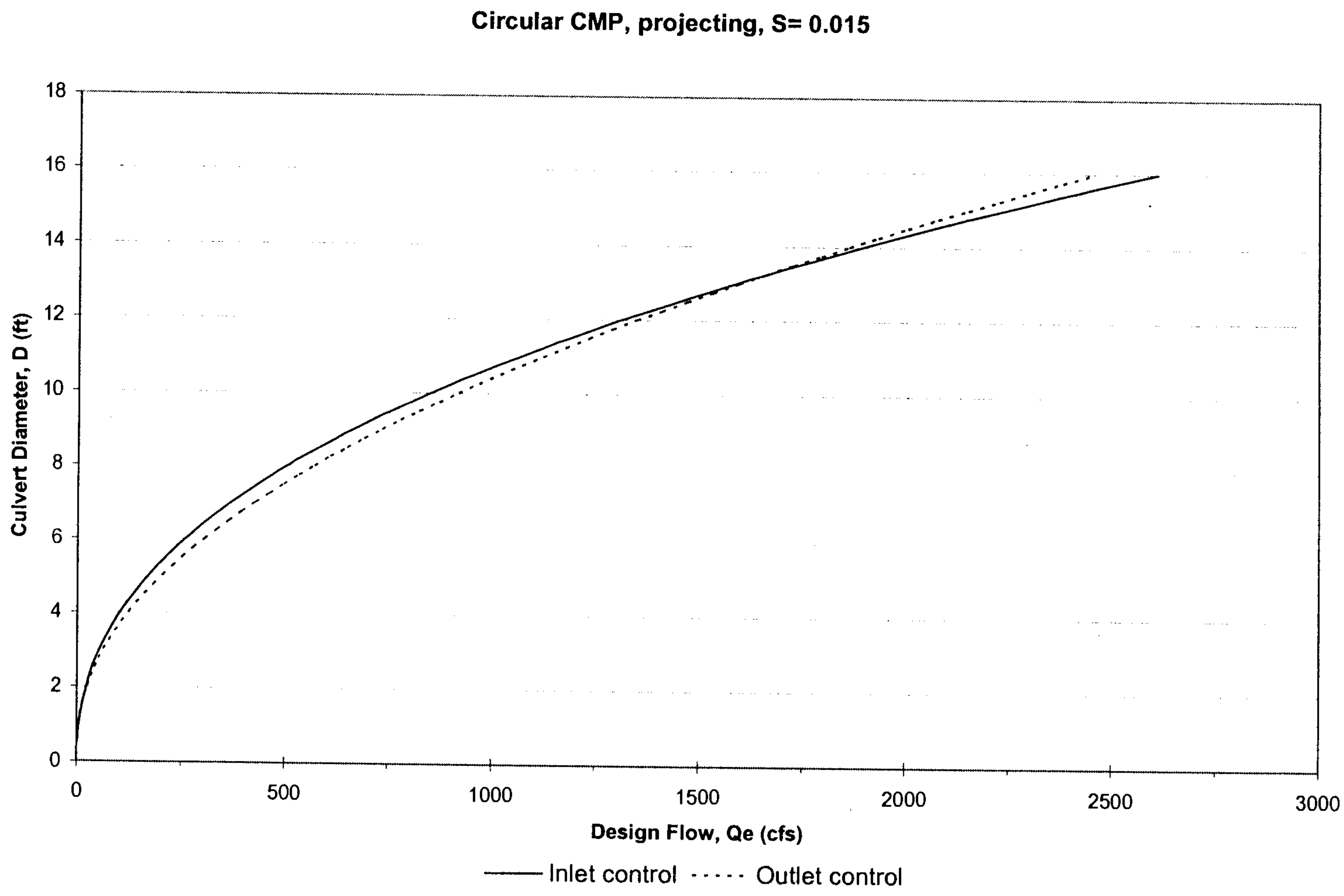

Figure 27. Circular corrugated metal pipe design curves, S= 0.015

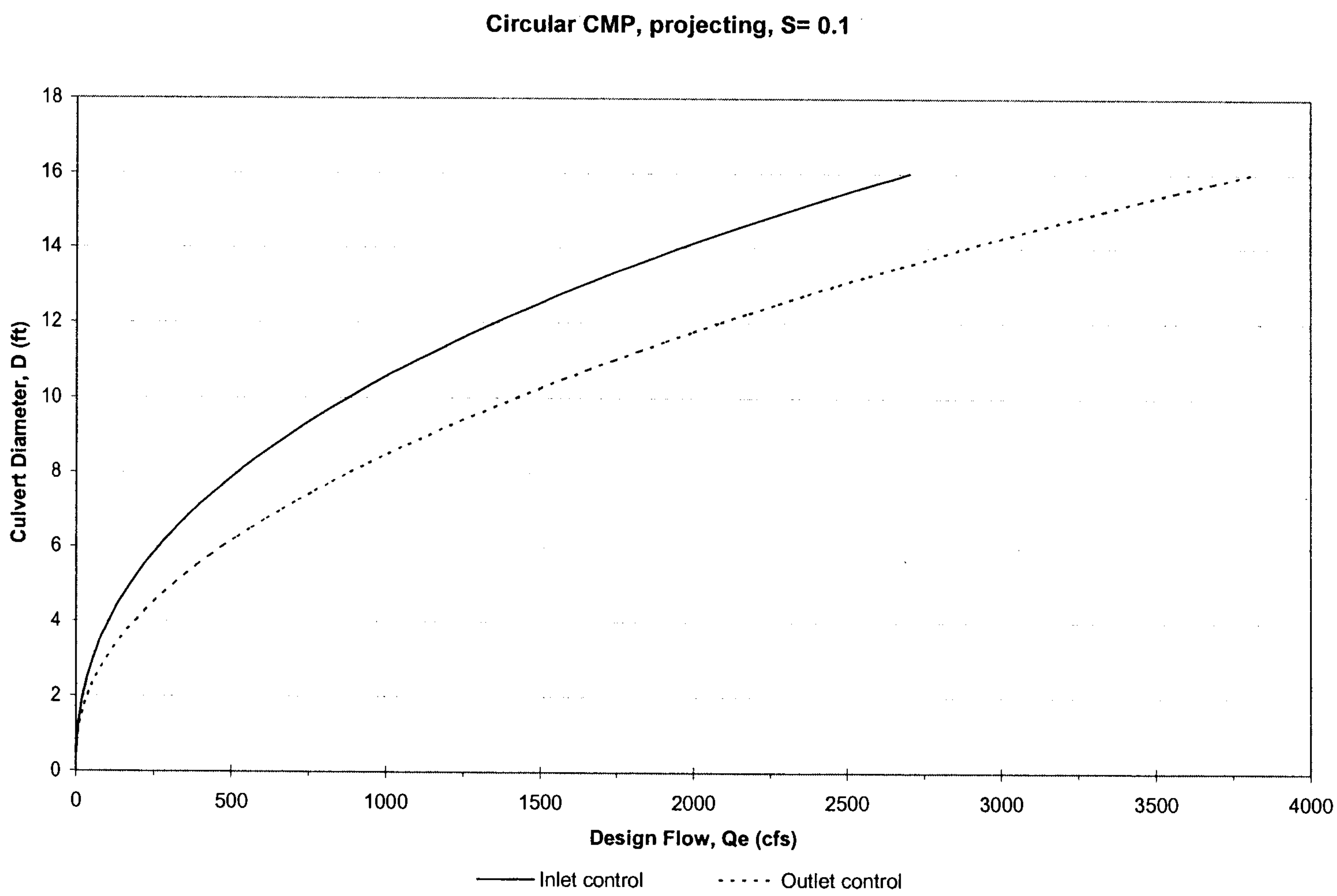

Figure 28. Circular corrugated metal pipe design curves, S= 0.1

5.2.6. Selection of Pipe

After discharge through the pipe, Q_v, is determined from Equation (21), the number and size of pipes should be selected. Single pipe may be considered first. If a computed trial size is larger than the design height of LWSC or availability of pipe size, multiple culverts should be used. The design discharge flowing through each pipe is equal to the total discharge through the vent divided by the number of pipes. Pipe diameter can be determined for the design discharge with design curves (D vs. Q_v) discussed earlier in this section. Other methods can be used to calculate pipe diameter as well, including those mentioned in the literature review.

5.2.7. Vented Ford Design Procedure

Once the design discharge is estimated, as discussed in section 4, design of a vented ford can be accomplished using the following steps:

1. Assume the pipe cover, C_o, is equal to Δz_{max} and the design overtopping flow depth, $y_2 = y_c$, is equal to 6-in to meet LWSC design criteria.
2. The overtopping design flow, Q_{top}, can be calculated using either Equation (22), where $H = y_2/0.6$, or Equation (25). Both equations give similar estimations.
3. Check to see if $\Delta z_{max} \geq 1$ to meet pipe cover requirements using Equation (15). If $\Delta z_{max} \leq 1$, depth should be increased to at least 1-ft.
4. Calculate the design culvert flow, Q_v, using Equation (21).
5. The design culvert flow, Q_v, can be used to determine pipe size. Either design

curves for inlet control conditions can be used, as discussed in section 5, or other pipe sizing methods mentioned in the literature review may be utilized.

6. The velocity through the culvert can be checked by dividing the culvert design flow by the culvert cross-sectional area.
7. For the final design, the pipe exit velocity should not exceed 10 ft/s (Motayed et al. 1982b) and the cover over the pipe(s) should be at least 1-ft thick.

5.3. Low Water Bridges

Low water bridges are generally considered where design stream discharge is large, debris potential is high, or when there are sensitive stream conditions such that stream disturbance must be avoided. Once in place, there are different types of stream flow that are possible at low water bridges. These are the same types of flows which occur at vented fords including: 1) open channel flow during low flows, 2) possibility for pressure flow as flow depths increase, and 3) weir flow over the structure and pressure flow underneath during overtopping. The upstream and downstream edges of the bridge deck are usually smoothly rounded to enhance the efficiency of discharge over the slab during overtopping.

Depending on the placement height of a low water bridge, the structure may cause changes in the stream width creating a channel transition at the crossing. According to Chaudhry (1993), the water depth decreases when the channel width constricts if the upstream flow is subcritical and depth increases if the flow upstream of the constriction is supercritical. Similar to a transition in the streambed elevation discussed in section 5.1, there is an upper limit, B_c, for the amount that a channel can be contracted without altering upstream flow. At this limit, critical flow conditions occur for stream flow passing by the structure. The following steps can be used to find the limiting width, B_c. First, critical flow depth for the design discharge can be calculated using

$$y_c = \frac{2}{3}\left(y_1 + \frac{V_1^2}{2g}\right) \tag{38}$$

Next, $$q = \sqrt{g y_c^3} \tag{39}$$

is derived from Equation (13) where q is the unit discharge corresponding to the critical depth, y_c. Finally, the limiting width can be computed using

$$B_c = \frac{Q_e}{q} \tag{40}$$

where B_c is the limiting width, Q_e is the design discharge, and q is the unit discharge calculated from Equation (39). Figures 29 and 30 show a plan and profile view respectively for a low water bridge that constricts a stream to the limiting width, B_c.

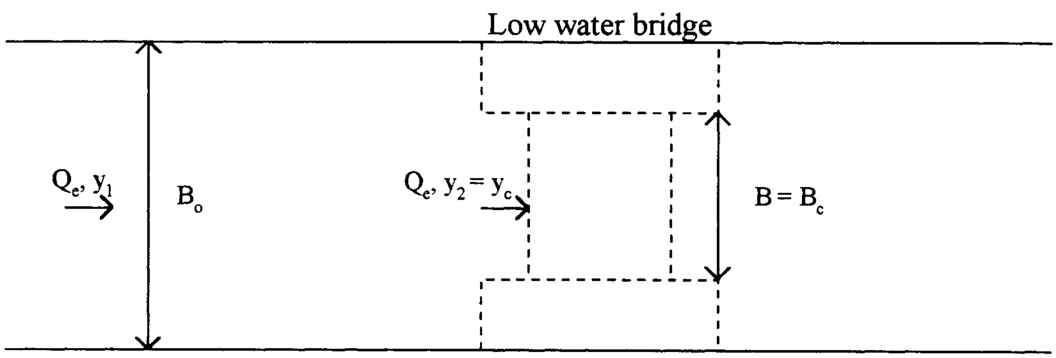

Figure 29. Plan view of a low water bridge causing critical flow conditions

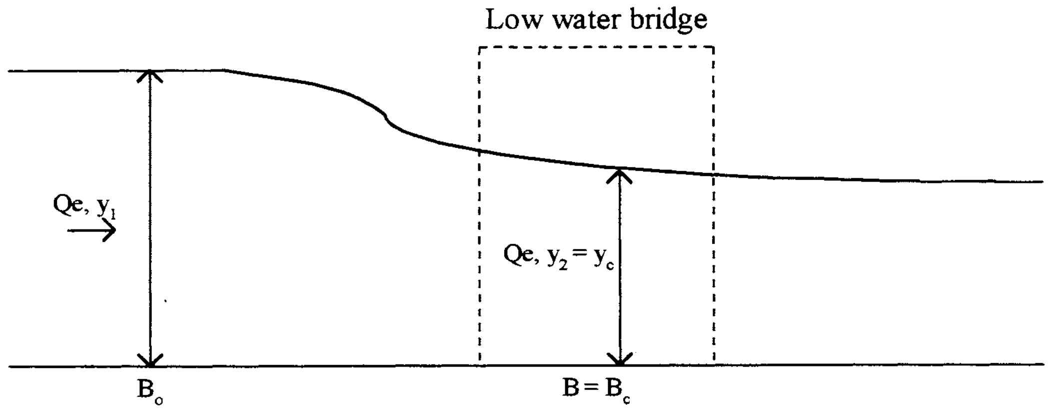

Figure 30. Profile view of a low water bridge causing critical flow conditions

If stream width is constricted by the addition of a low water bridge with length B, where B is greater than the limiting width, the transitional stream depth can be determined by rearranging Equation (11) giving

$$\frac{1}{y_2^2} = \frac{2gw_2^2}{Q_e^2}\left[y_1 + \left(\frac{Q_e^2}{2gw_1^2 y_1^2}\right) - y_2\right] \tag{41}$$

where $w_1 = B_o$, is the width of the undisturbed channel and $w_2 = B$, the constricted width at the crossing. Equation (41) is used to determine the depth of flow at the crossing, y_2, for a low water bridge with width B. This equation is solved using a mathematical equation solver or trial and error methods.

Any constriction of the channel width beyond the upper limit, B_c, causes choking conditions for the upstream water. In this situation, the backwater depth should be checked to assure that existing stream banks can contain increased water levels upstream. Discussed by Motayed et al. (1982b), depth upstream from the bridge can be computed by adding the flow depth without the bridge and the height of the backwater created by the bridge. The following equation can be used for a backwater approximation:

$$h' = C_D \left(\frac{A_s}{A_1} \right) \left(\frac{V^2}{2g} \right) \tag{42}$$

Where h' = backwater flow depth, C_D = drag coefficient ($C_D = 2$ for LWSCs), A_s = projected area of slab, pier, and abutments on a plane perpendicular to flow, A_1 = cross-sectional area of upstream flow, and V = average velocity of flow.

5.3.1. Low Water Bridge Design Procedure

The design of a low water bridge starts with estimation of design discharge as described in section 4. The structure height should be chosen so that there is no overtopping for the design flow, to reduce safety concerns, i.e. the bottom of the bridge slab is slightly higher than the surface water elevation at the crossing for the design flow. After finding the design flow, Q_e, the following steps can be used for design:

1. If upstream design flow is supercritical:
    a. The length of low water bridge, B, should equal the width of the channel, B_o, so that $y_2 = y_1$. Calculate flow depth, y_1, for the design discharge using Equation (7) or (8).
2. If the upstream flow is subcritical:
    a. The length of low water bridge, B, may be equal to the width of the channel, B_o, so that $y_2 = y_1$. Calculate flow depth, y_1, for the design discharge using Equation (7) or (8), or

b. The bridge length, B, may be reduced if needed.

- It is recommended that reduced bridge length should equal the limiting constriction width for the stream flow, $B = B_c$, so that flow depth $y_2 = y_c$. B_c is calculated using Equation (40).
- If $B_c < B < B_o$, Equation (41) can be used to determine flow depth, y_2.
- If $B < B_c$, upstream flow is altered and backwater elevation should be checked using Equation (42) to make sure the water surface elevation stays within the existing stream banks.

6. DISCUSSION AND CONCLUSIONS

LWSCs are economic alternatives for bridge replacement and new stream crossing projects on low volume roads. They are structures that can effectively be used in many different regions throughout the United States. Careful planning and detailed design are important in LWSC project development. Hydrologic and hydraulic analyses and designs have been conducted in this study, as they are critical components of the LWSC design process. The objective of developing a systematic approach for hydrologic and hydraulic design has been achieved by extensive review of previous studies, examining feedback from the on-line survey, and the investigation and improvement of existing design methodologies and techniques for LWSC hydrologic and hydraulic design.

In hydrologic design, the conventional method of using instantaneous annual maximum flows was compared to a design method based on daily flow data and flow-duration curves. The conventional method establishes relationships between design frequency and design peak flow. The advantage is the availability of data at gaged streams and ability to use USGS regression equations, which are available for locations throughout the nation, to estimate flow data at ungaged streams. Unfortunately, duration of flooding cannot be determined with this method, so it is not the best choice for LWSC design.

In contrast, flow-duration curves developed with daily flow data can be used to determine the percent of time in a year that a design flow is equaled or exceeded. The information obtained from the curves, including flow exceedence frequency and flood durations, is more suitable for LWSC design since acceptable duration of road closure during a given year can be considered. The only drawback is that when flow data are not available, regression equations need to be developed from available streamflow data coming from gaged sites that have watersheds with similar geophysical conditions.

The design discharge calculated in hydrologic design is required for the hydraulic design for each of the three types of LWSCs. The nature of flow in streams is also important for each design. Selection of structure types and design considerations are dependent on whether streamflow is at subcritical or supercritical state.

Unvented ford design is based on the depth and state of flow for the design discharge. It is concluded that the overtopping flow depth on a raised unvented ford is influenced by LWSC height. Analyses and results show that hydraulic design for raised unvented fords can be simplified by setting the LWSC height to a maximum level, Δz_{max}, where the overtopping flow is equal to the design flow critical depth. When this is assumed, hydraulic principles or empirical equations developed from laboratory experiments, which have been validated for specific conditions, can be used for design.

Vented ford design is similar to traditional culvert design and can be accomplished using many available methods. The first step is determining the overtopping flow, based on allowable overtopping depth, and culvert flow which combine to equal the design discharge. The overtopping flow is estimated with methods discussed for raised unvented ford design in which hydraulic principles or empirical equations can be used. The remaining flow, equal to culvert flow, is used for culvert pipe selection and sizing in which traditional methods are acceptable for design. In this study it was determined that inlet control design for culverts results with larger pipe size than outlet control design in most cases. Therefore, the culvert sizing process is simplified because outlet control design does not have to be checked.

Depending on the length and height of a low water bridge, alteration of the stream channel may occur. Disturbances to the width of a stream channel affect flow depths that are used to determine bridge placement height. A relationship was found between changes in channel width and change of flow depth. After analyzing these findings it was concluded that low water bridges should be placed at a height where channel disturbance can be avoided. If a bridge design is uneconomical due to its long length, a shorter structure which could cause stream width constriction may need to be considered. Under subcritical flow conditions, reducing the length of the structure, causing some constriction of the channel, could lead to a lower flow depth at the bridge. It is suggested that the length be limited to minimum channel width, B_c, where critical flow conditions occur at the crossing.

The hydrologic and hydraulic design procedures in this study have been evaluated. It is recommended that flow-duration curves or regression equations be developed for all ungaged streams in each state so that daily flow data can be utilized for LWSC projects. Regional regression equations that were developed for Iowa have been useful for the design of LWSCs. It would be necessary for other states to have the same type of resources available for future LWSC designs. With this information, more reasonable design flows can be estimated and the LWSC design procedures described in this report can be used effectively.

7. DESIGN EXAMPLES

7.1. Example 1

Data

Design discharge:	$Q_e = 30$ cfs
Stream slope:	$S = 0.005$
Streambed roughness:	$n = 0.04$
Stream width:	w = 15.0 ft

Design

1. Determine if the flow is subcritical or supercritical:

    Depth of design flow is calculated using Equation (8), $y_1 = 0.90$-ft.

    Froude number is calculated using Equation (3), $Fr = 0.41$.

    Fr < 1 and flow is subcritical.

2. Analyze the depth of design flow, y_1:

    $y_1 > 6.0$-in, therefore a raised unvented ford is needed.

3. Analyze the critical flow depth, y_c:

    Critical flow depth is calculated using Equation (13), $y_c = 6$-in.

    $y_c = 6.0$-in and meets criteria, therefore a raised unvented ford can be used.

4. Determine the height for the raised unvented ford:

    Structure height is calculated using Equation (15), $\Delta z_{max} = 0.23$-ft.

    The raised unvented ford can be constructed 0.23-ft above the streambed to allow a 6.0-in overtopping depth at design flow. If the unvented ford is raised to a greater height, overtopping flow will be less than 6-in for design flow and the upstream flow will be altered.

7.2. Example 2

<u>Data</u>

Waterbody:	Keg Creek
Project Site:	Iowa, Region II
Watershed Area:	$A = 30.4$ mi^2
Stream slope:	$S = 0.012$
Streambed roughness:	$n = 0.04$
Stream width:	w = 30.0 ft

<u>Design</u>

1. Estimate design flow, Q_e:

   Using 2% exceedence time for Region II in Iowa, Equation (1) is used to find Q_e.

   Table 5 gives: a = 6.78, b = 0.90, and

   $Q_e = 147.0$ cfs

2. Determine if the flow is subcritical or supercritical:

   Depth of design flow is calculated using Equation (8), $y_1 = 1.21$-ft.

   Froude number is calculated using Equation (3), Fr = 0.65.

   Fr < 1 and flow is subcritical.

3. Analyze the depth of design flow, y_1:

   $y_1 > 6.0$-in, therefore a raised unvented ford is considered.

4. Analyze the critical flow depth, y_c:

   Critical flow depth is calculated using Equation (13), $y_c = 0.91$-ft.

   $y_c > 6.0$-in, therefore a vented ford is needed.

5. Determine the overtopping flow for a vented ford:

   Assume pipe cover depth is Δz_{max}, $y_2 = y_c$, and $H = y_2/0.6$

   Equation (22) gives Q_{top} = 58-cfs and Equation (25) gives Q_{top} = 60-cfs.

   The results are similar and a decision is made to use Q_{top} = 58-cfs.

6. Determine the culvert flow:

   Design flow for the culvert(s) is calculated using Equation (21), Q_v = 89-cfs.

   This flow can be used for pipe selection and sizing.

7. Determine culvert pipe to use:

   A decision is made to use corrugated metal pipe with slope of 0.015.

   Figure 27 is used to determine pipe size for the culvert design flow, Q_v.

   Culvert diameter, D, for 1 pipe gives D = 4.0-ft, or

   Culvert diameter, D, for 4 pipes gives D = 1.0-ft, which is selected.

8. Check velocity, V:

   The velocity is equal to culvert design flow divided by pipe cross-sectional area.

   V = 7-ft/s, and V > 10-ft/s, so the design is good.

9. Check pipe cover, Δz_{max}:

   The depth of pipe cover is calculated using Equation (15), Δz_{max} = 0.05-ft.

   The pipe cover must be increased to at least 1.0-ft to meet the minimum requirement.

   In raising the pipe cover to a depth larger than Δz_{max}, the overflow depth will be smaller than 6.0-in for the design flow.

7.3. Example 3

Data

Waterbody:	North Fish Creek
Project Site:	Bayfield County, Wisconsin
Watershed Area:	$A = 65.4$ mi^2
Stream slope:	$S = 0.005$
Streambed roughness:	$n = 0.04$
Stream width:	w = 38.0 ft
USGS identification:	040263491

Design

1. Estimate design flow, Q_e:

Daily streamflow data are obtained from the U.S. Geological Survey which is available at the website www.usgs.org. The data are arranged into class intervals of ascending order. The percent of time during which flow was equal to or greater than each class interval is determined. The information is graphically presented in Figure 31 as a flow-duration curve for North Fish Creek.

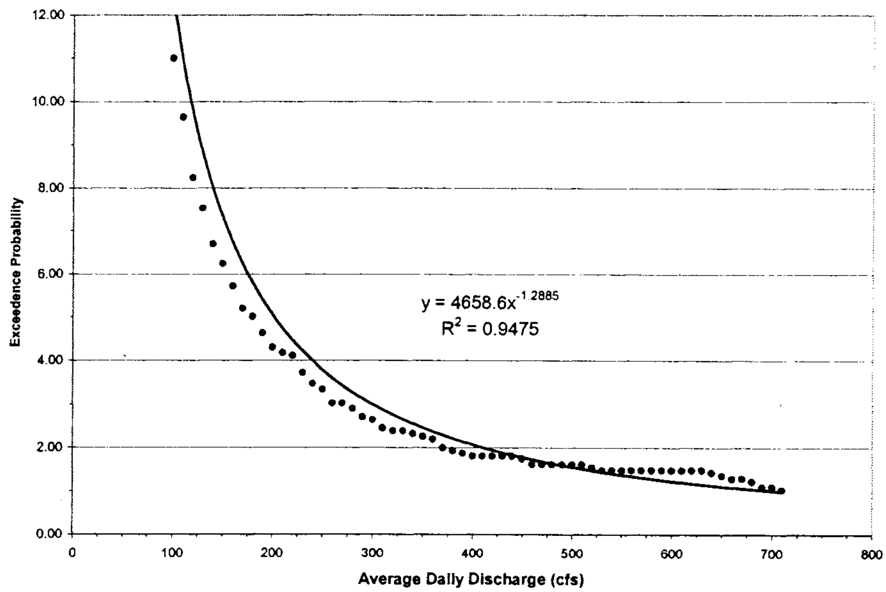

Figure 31. Flow duration curve for North Fish Creek in Wisconsin

At 2 percent exceedence time, Figure 31 gives Q_e = 375-cfs.

2. Determine if the flow is subcritical or supercritical:

   Depth of design flow is calculated using Equation (8), y_1 = 2.32-ft.

   Froude number is calculated using Equation (3), Fr = 0.49.

   Fr < 1 and flow is subcritical.

3. Analyze the depth of design flow, y_1:

   y_1 > 6.0-in, therefore a raised unvented ford is considered.

4. Analyze the critical flow depth, y_c:

   Critical flow depth is calculated using Equation (13), y_c = 1.45-ft.

   y_c > 6.0-in, therefore a vented ford is needed.

5. Determine the overtopping flow for a vented ford:

   Assume pipe cover depth is Δz_{max}, $y_2 = y_c$, and $H = y_2/0.6$

   Equation (22) gives Q_{top} = 71-cfs and Equation (25) gives Q_{top} = 76-cfs.

   The results are similar and a decision is made to use Q_{top} = 71-cfs.

6. Determine the culvert flow:

   Design flow for the culvert(s) is calculated using Equation (21), Q_v = 304-cfs.

   This flow can be used for pipe selection and sizing.

7. Determine culvert pipe to use:

   Use circular concrete pipe with slope of 0.015.

   Figure 18 is used to determine pipe size for the culvert design flow, Q_v.

   Culvert diameter, D, for 1 pipe gives D = 6.0-ft, or

   Culvert diameter, D, for 4 pipes gives D = 1.5-ft.

A low water bridge should be used if the pipe size/quantity is too large or if the vented ford design is uneconomical.

8. Determine low water bridge placement:

   A shorter structure is desirable, which would alter the channel width.

   The flow is subcritical and the limiting width, B_c, is chosen for design.

   The limiting width is calculated using Equation (40), $B_c = 37.8$-ft

   The limiting width is approximately the same as existing channel width.

   If this length of bridge is used, it would have to be at least 2.0- ft. above the streambed.

9. Compare the vented ford design to the low water bridge design:

   Cost estimation is generally a good method for comparison.

APPENDIX A

ON-LINE LWSC SURVEY

Survey on the Use of Low Water Stream Crossings (2002)

Iowa State University
Dept. of Civil and Construction Engineering

If you have any questions or if you wish to send anything to us please contact:

Roy R. Gu
Dept. of Civil and Construction Engineering
Iowa State University
Ames, IA 50011
roygu@iastate.edu

Please complete the information for the person filling out the questionnaire or someone we can contact should we require further information.

Name:

Your state/county/agency:

Your position/title:

E-mail:

Phone:

Fax:

Could we contact you if we require additional information?

Yes

No

Are you interested in receiving a copy of the executive summary of the design manual?

Yes

No

1. If your state/county/agency has any low water stream crossings (LWSCs), how many on each of the following roads?
Road surface type:

Primitive/field access (dirt):

Aggregate-surfaced (gravel)

Paved (or asphalt) road

Other (please describe):

2. Please list the number of LWSCs in your state/county/agency, average cost, road type/service level, and average daily traffic (ADT) count.

LWSC Type	Number	Average Cost $	Road Type / Service Level	ADT Count (vehicle/day)
Fords (no pipes)				
Vented Fords (with pipes)				
Low Bridges				
Other:				

3. Based on your experience with LWSCs, please indicate factors considered and specify constraints in choosing and designing LWSC structures.

- Average Daily Traffic (ADT): ______ vehicles or less
- Overtopping frequency: ______ times/days per year or less
- Overtopping flow depth: ______ inches or less
- Maximum cost: $______
- Cost saving compared to bridges/culverts: ______ % or more
- Drainage areas: ______ acres
- Stream-flow discharge: ______ cfs
- Stream-bank height: ______ ft or less
- Approach distance, i.e. sight distance for warning signs: ______ ft or longer
- Streambed slope: ______ % or milder
- Vertical curve at dip (cross slope, i.e. approach grade): ______ % or less
- Height of crossing above streambed: ______ ft or less
- Downstream slope of crossing structure: ______ % or less
- Extra time for alternate routes: ______ minutes or less
- Average duration of traffic interruption: ______ hours or less
- Maximum distance from maintenance facilities: ______ miles
- Other factors and constraints:

4. If your state/county/agency has constructed any LWSCs, what did you use as design standards/guidelines?

a. Existing references (please list or provide copy) or

b. In-house design (please describe specifications or provide copy)

5. In your experience with LWSCs,

a. How have they performed or what has been their integrity?

b. What problems, if any, did you have with LWSCs?

☐ Destroyed by erosion

☐ Too frequent overtopping

Other:

6. If your state/county/agency is going to build (more) LWSCs,

a. What pavement treatment and thickness would be specified?

b. Would you use inlet and outlet protection and erosion control?

○ Yes

○ No

Type of protection or method of erosion control:

c. What erosion and sediment control procedures would you use during the LWSC construction phase?

d. What marking and signing for traffic safety does your agency require?

e. What maintenance of LWSC structures (including scour prevention) would be required?

f. What data would you use for hydrologic analysis,

○ Daily flows

○ Annual peak flows

g. What methods of hydrologic analysis does your agency employ?

APPENDIX B

CONSTANTS FOR EQUATIONS DEVELOPED IN HAESTED (1999)

Table B1. List of inlet control submerged factors C and Y (Haestad, 1999)

SHAPE AND MATERIAL	INLET EDGE DESCRIPTION	SUBMERGED	
		C	Y
Circular Concrete	Square edge w/ headwall	0.0398	0.67
	Groove end w/ headwall	0.0292	0.74
	Groove end projecting	0.0317	0.69
Circular CMP	Headwall	0.0379	0.69
	Mitered to slope	0.0463	0.75
	Projecting	0.0553	0.54
Circular	Beveled ring, 45° bevels	0.0300	0.74
	Beveled ring, 33.7° bevels	0.0243	0.83
Rectangular Box	30° to 75° wingwall flares	0.0385	0.81
	90° and 15° wingwall flares	0.0400	0.80
	0° wingwall flares	0.0423	0.82
Rectangular Box	45° wingwall flare d=.043	0.0309	0.80
	18° to 33.7° wingwall flare d=.083	0.0249	0.83
Rectangular Box	90° headwall w/ 3/4" chamfers	0.0375	0.79
	90° headwall w/ 45° bevels	0.0314	0.82
	90° headwall w/ 33.7° bevels	0.0252	0.865
Rectangular Box	3/4" chamfers; 45° skewed headwall	0.0402	0.73
	3/4" chamfers; 30° skewed headwall	0.0425	0.705
	3/4" chamfers; 15° skewed headwall	0.04505	0.68
	45° bevels; 10°-45° skewed headwall	0.0327	0.75
Rectangular Box 3/4" Chamfers	45° non-offset wingwall flares	0.0339	0.803
	18.4° non-offset wingwall flares	0.0361	0.806
	18.4° non-offset wingwall flares, 30° skewed barrel	0.0386	0.71
Rectangular Box Top Bevels	45° wingwall flares- offset	0.0302	0.835
	33.7° wingwall flares- offset	0.0252	0.881
	18.4° wingwall flares- offset	0.0227	0.887
C M Boxes	90° headwall	0.0379	0.69
	Thick wall projecting	0.0419	0.64
	Thin wall projecting	0.0496	0.57
Horizontal Ellipse Concrete	Square edge w/ headwall	0.0398	0.67
	Groove end w/ headwall	0.0292	0.74
	Groove end projecting	0.0317	0.69
Vertical Ellipse Concrete	Square edge w/ headwall	0.0398	0.67
	Groove end w/ headwall	0.0292	0.74
	Groove end projecting	0.0317	0.69

Table B1. (Continued)

Pipe Arch 18" Corner Radius CM	90° headwall Mitered to slope Projecting	0.0379 0.0463 0.0496	0.69 0.75 0.57
Pipe Arch 18" Corner Radius CM	Projecting No bevels 33.7° bevels	0.0487 0.0361 0.0264	0.55 0.66 0.75
Pipe Arch 31" Corner Radius CM	Projecting No bevels 33.7° bevels	0.0487 0.0361 0.0264	0.55 0.66 0.75
Arch CM	90° headwall Mitered to slope Thin wall projecting	0.0379 0.0463 0.0496	0.69 0.75 0.57
Circular	Smooth tapered inlet throat Rough tapered inlet throat	0.0196 0.0289	0.89 0.90
Elliptical Inlet Face	Tapered inlet-beveled edges Tapered inlet-square edges Tapered inlet-thin edge projecting	0.0368 0.0478 0.0598	0.83 0.8 0.75
Rectangular	Tapered inlet throat	0.0179	0.97
Rectangular Concrete	Side tapered-less favorable edges Side tapered-more favorable edges	0.0466 0.0378	0.85 0.87
Rectangular Concrete	Slope tapered-less favorable edges Slope tapered-more favorable edges	0.0466 0.0378	0.65 0.71

Table B2. List of outlet control submerged factor K_e (Haestad, 1999)

CULVERT TYPE	ENTRANCE TYPE AND DESCRIPTION	LOSS COEFFICIENT, Ke
Pipe, Concrete	Projecting from fill, socket end (groove end)	0.2
	Projecting from fill, square cut end	0.5
	Headwall or Headwall with wingwalls	
	Socket end of pipe (groove end)	0.2
	Square edge	0.5
	Rounded (radius= 1/12D)	0.2
	Mitered to conform to fill slope	0.7
	End-Section conforming to fill slope	0.5
	Beveled edges, 33.7° or 45° bevels	0.2
	Side- or slope-tapered inlet	0.2
Pipe or Pipe Arch Corrugated Metal	Projecting from fill (no headwall)	0.9
	Headwall or headwall and wingwalls square-edge	0.5
	Mitered to conform to fill slope, paved or unpaved	0.7
	End-Section conforming to fill slope	0.5
	Beveled edges, 33.7° or 45° bevels	0.2
	Side- or slope-tapered inlet	0.2
Box, Reinforced Concrete	Headwall parallel to embankment (no wingwalls)	
	Square-edged on 3 edges	0.5
	Rounded on 3 edges to radius of 1/12 barrel dimension, or beveled edges on 3 sides	0.2
	Wingwalls at 30° to 75° to barrel	
	Square-edged at crown	0.4
	Crown edge rounded to radius of 1/12 barrel dimension, or beveled top edge	0.2
	Wingwall at 10° to 25° to barrel	
	Square-edged at crown	0.5
	Wingwalls parallel (extension of sides)	
	Square-edged at crown	0.7
	Side- or slope-tapered inlet	0.2

REFERENCES

Barrett, B.C., *Summary Report of Research on the Hydraulic Modeling of Low Water Stream Crossings*. Iowa State University, May, 1984.

Bedient, P.B., and W.C. Huber. *Hydrology and Floodplain Analysis*. 3rd Edition. Prentice Hall Inc., Upper Saddle River, New Jersey, 2002.

Carstens, R.L., and R.Y. Woo. *Liability and Traffic Control Considerations for Low Water Stream Crossings*. Engineering Research Institute Project 1470 Final Report, Iowa State University, Ames, 1981.

Chaudhry, M.H. *Open Channel Flow*. Prentice-Hall Inc., New Jersey. 1993.

Coghlan G., and N. Davis. *Low Water Crossings*. Transportation Research Record 702, pp. 98-103, TRB, National Research Council, Washington D.C., 1979.

Eriksson, M.O. *Cost-Effective Low-Volume-Road Stream Crossings*. Transportation Reserarch Record 898, pp. 227-232, TRB, National Research Council, Washington D.C., 1983.

Gupta, R.S. Hydrology and Hydraulic Systems. 2nd Edition. Waveland Press, Inc., Prospect Heights, Illinois, 2001.

Haestad Methods. *Computer Applications in Hydraulic Engineering*. 3rd Edition. Haestad Methods Inc., Haestad Methods Press, 1999.

Herr, L.A. and H.G. Bossy. Hydraulic *Charts for the Selection of Highway Culverts*. Hydraulic Engineering Circulation No. 5, GPO, Washington D.C., 1965.

Lohnes, R.A., R.R. Gu, T. McDonald, and M.K. Jha. *Low Water Stream Crossings: Design and Construction Recommendations*. Final Report to Iowa Dept. of Transportation, Project TR-453, Center for Transportation Research and Education Project 01-78, 2001.

Modi, P.N., and S.M. Seth. *Hydraulics and Fluid Mechanics*. Tenth Edition, Standard Book House, Delhi, 1991.

Motayed, A.K., F.M. Chang, and D.K. Mukherjee. *Design and Construction of Low Water Stream Crossings*. Federal Highway Administration Report No. FHWA/RD-82/163, U.S. Department of Transportation, 1982 a.

Motayed, A.K., F.M. Chang, and D.K. Mukherjee. *Design Guide: Low Water Stream Crossing*. Federal Highway Administration Report No. FHWA/RD-82/164, U.S. Department of Transportation, 1982 b.

Motayed, A.K., F.M. Chang, and D.K. Mukherjee. *Design and Construction of Low Water Stream Crossings: Executive Summary*. Federal Highway Administration Report No. FHWA/RD-83/015, U.S. Department of Transportation, 1983.

Normann J.M., R.J. Houghtalen, and W.J. Johnston. *Hydraulic Design of Highway Culverts*. US D.O.T. Report No. FHWA-IP-85-15, Hydraulic Design Series No. 5, 1985.

Pienaar, P.A., and A.T. Visser. *Methodology for Functional Design of Low-Level River Crossings in South Africa*. Transportation Research Record 1504, pp. 112-117, TRB, National Research Council, Washington D.C., 1995.

Ring, S.L. *The Design of Low Water Stream Crossings*. Transportation Research Record 1106, pp. 309-318, TRB, National Research Council, Washington D.C., 1987.

Rossmiller, R.L., R.A. Lohnes, S.L. Ring, J.M. Phillips, B.C. Barrett. *Design Manual for Low Water Stream Crossings*. Iowa Department of Transportation, Project No. HR 247, 1984.

Rossmiller, R.L. *Iowa Design Manual for Low Water Stream Crossings*. Transportation Research Record 995, pp. 35-42, TRB, National Research Council, Washington D.C., 1984.

Shen, H.W. *Opinion Survey for Selection of Low-Water Crossing Structures*. Transportation Research Record 898, pp. 221-227, TRB, National Research Council, Washington D.C., 1983.

United States Department of Agriculture (USDA) Forest Service. *Low Water Crossing Design (Low Standard Roads)*. 2002 a. http://www.fs.fed.us/rm/RRR/Applications/lwxinst.html. Accessed June, 2002.

United States Department of Agriculture (USDA) Forest Service. *Landowners Guide to Building Forest Access Roads- Stream Crossing Methods*. 2002 b. http://www.na.fs.fed.us/spfo/pubs/stewardship/accessroads/stream.htm. Accessed June, 2002.

United States Geological Survey (USGS). *Surface-Water Data for the Nation*. 2002. www.usgs.org. Accessed October, 2002.

Warhol, T., and M.R. Pyles. *Low Water Fords: An Alternative to Culverts on Forest Roads*. Proceedings, 12th Council on Forest Engineering Meeting, Coeur d'Alene, Idaho. August 27-30, 1989.

Wilent, S. *How to Build a Vented Ford*. The Forestry Source. 2002. http://www.safnet.org/archive/202_howto_ford.htm. Accessed February, 2002.