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Evaluation of Alternatives For Hydraulic Design of Bridges With HEC-RAS

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University of Tennessee, Knoxville

To the Graduate Council:

I am submitting herewith a thesis written by William Wesley Peck entitled "Evaluation of Alternatives For Hydraulic Design of Bridges With HEC-RAS." I have examined the final electronic copy of this thesis for form and content and recommend that it be accepted in partial fulfillment of the requirements for the degree of Master of Science, with a major in Environmental Engineering.

Bruce A. Tschantz, Major Professor

We have read this thesis and recommend its acceptance:

James L. Smoot, J. Hal Deatherage

Accepted for the Council:
Carolyn R. Hodges

Vice Provost and Dean of the Graduate School

(Original signatures are on file with official student records.)

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Accepted for the Council:

Interim Vice Provost and
Dean of The Graduate School

EVALUATION OF ALTERNATIVES FOR HYDRAULIC DESIGN OF BRIDGES WITH HEC-RAS

A Thesis
Presented for the
Master of Science
Degree
The University of Tennessee, Knoxville

William Wesley Peck, P.E.
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ABSTRACT

On average, flooding causes over five billion dollars of damage and 99 fatalities per year in the United States alone. These ever-increasing numbers indicate a need for rigorous design procedures for any structures which exacerbate flood risk. The U.S. Army Corps of Engineers Hydraulic Engineering Center (HEC) has developed a computer program called River Analysis System, and more commonly referred to as HEC-RAS or RAS.

HEC-RAS is the best and most recent method for hydraulic design of bridges, and available documentation provides guidance for its use. The author is experienced in hydraulic design of bridges and has developed this thesis in order to address certain generalities and gaps in the HEC-RAS documentation. Factors examined included: location of the transition and approach sections, effects of choice of bridge calculation methods for low flow events, effects of bridge calculation methods for high flow events, effects of interpolated cross-sections, and effects of choice of inappropriate boundary conditions.

It was found that transition reach lengths recommended by HEC-RAS documentation were the most accurate, and recommendations made as part of the HEC-2 program result in over calculation of water surface elevations. It was also found that use of the energy method for bridge calculations during high flow events which experience only pressure flow results in calculated water surface elevations that are much higher than observed data indicates.

TABLE OF CONTENTS

CHAPTER PAGE

I. INTRODUCTION....1
Objectives....12
Summary....14

II. LITERATURE REVIEW....16
The Bridge Reach....18
Transition Lengths....21
Cross-Section Spacing....24
Selection of Bridge Modeling Method....26
Summary....27

III. THEORY AND LIMITATIONS OF RAS....28
Open Channel Flow Theory....29
Open Channel Flow Regimes....31
Normal Depth....34
Energy Equation and The Standard Step Method....36
Critical Depth....39
Momentum Equation....41
RAS Limiting Assumptions....44
Steady, Gradually Varied Flow Assumption....44
One-Dimensional Flow Assumption....46
RAS Bridge Cross-Sections....50
RAS Low Flow Bridge Computations....53
Energy Method....54
Momentum Balance Method....54
Yarnell Method....55
WSPRO Method....56
RAS High Flow Computations....62
Energy Method....63
Pressure/Weir Method....64

IV. MODELING REQUIREMENTS AND SITE SELECTION....67
Data Requirements....67
Study Site Selection....76

V. RESULTS....81
Transition Lengths....83
Low Flow Bridge Analysis Methods....89

CHAPTER PAGE

High Flow Bridge Analysis Methods....97
Perched Bridge Analysis....99
Effects of Cross-Section Interpolation....100
Influence of Boundary Conditions....104

VI. SUMMARY & CONCLUSIONS....108
Conclusions....109
Recommendations For Future Work....112

BIBLIOGRAPHY....115

VITA....119

LIST OF TABLES

TABLE PAGE

1. Bridge Failure Modes..........5

2. Computed and Observed Water Surface Elevations..........19

3. Flow Classification Types..........34

4. Description of Bridge Study Locations..........80

5. Expansion and Contraction Lengths..........83

6. Average Errors of Water Surface Elevation Computations At Cross-Section..........87

7. Range of Water Surface Elevation Errors..........87

8. Negative Water Surface Elevation Errors for Various Transition Length Recommendations..........87

9. Bridge Locations and Flow Data for Low Flow Events..........90

10. Average Water Surface Elevation for Low Flow Computation Methods..........92

11. Range of Water Surface Elevation Errors for Low Flow Computation Methods..........92

12. Negative Water Surface Elevation Errors for Low Flow Computation Methods..........93

13. Summary of Low Flow Events..........94

14. Average Water Surface Elevation Errors At Each Cross-Section For High Flow Computation Methods..........97

LIST OF FIGURES

FIGURE PAGE

1. Bridge Reach..20

2. Pressure Flow Versus Open Channel Flow..30

3. Steady And Unsteady Flow Regimes...32

4. Uniform And Varied Flow Regimes...33

5. Gradually And Rapidly Varied Flow..33

6. Variables of The Energy Equation...37

7. Specific Energy at a Cross-Section..40

8. Momentum Equation Definition Sketch...42

9. Dimensions of Flow..46

10. Typical Velocity Distribution For Various Cross-Section Shapes..................48

11. Bridge Reach Cross-Sections...50

12. Bridge Cross-Sections..52

13. Flow Types..53

14. Coefficient of Discharge For Type 3 Bridge Opening..............................59

15. Definition Sketch of WSPRO Assumed Streamlines.................................61

16. Low Flow, Pressure Flow, and Weir Flow Through a Bridge Opening............63

17. Typical RAS Cross-Section Data Editor...68

18. Cross-Section Plot Generated By RAS...72

19. RAS Boundary Condition Editor...74

FIGURE PAGE

20. RAS Bridge Data Editor..76

21. Typical Study Site Topography..78

22. Typical Water Surface Profile..81

23. Expansion Lengths..84

24. Typical Multiple Pile Bent Installation..91

25. Typical Perched Bridge Section.. ..99

26. Changes in water surface profiles due to interpolated sections....................103

27. Water Surface Profiles for Various Boundary Conditions.......................106

28. Effects of Interpolated Cross-Sections on Computations..........................107

Chapter I

Introduction

Flooding is the most common of natural disasters, with the exception of fire. The Federal Emergency Management Agency (FEMA) states that approximately ninety percent of all presidential disaster declarations involve flooding as a major component (FEMA, 1996).

The U.S. Army Corps of Engineers (Corps) compiles flood damage statistics and reports annually to the U.S. Congress. The Corps reports that, for the ten-year period from 1990 through 1999, floods caused an average of five billion dollars of damage per year (Corps, 2000). This is a significant increase over the 2.1 billion dollar average for the years 1983 to 1990. The Corps also reports that, on average 99 people lose their lives every year due to flooding.

The reasons for the increasing dangers of flooding and flood damage are linked with population growth in the United States. Analysis of census figures for the past 40 years show an average population growth of ten percent per decade (U.S. Census Bureau, 2000). This population growth is causing increased development in most cities.

The continued population growth throughout the United States leads to more development and increasing encroachment in areas that were considered impossible to develop. Areas that were formerly too complex or costly to build on are now candidates for continued development of land and resources. To sum it up, all the good and easy locations for development are taken. This results in development in areas that require roads and bridges to be designed to operate under conditions that in some locations were previously considered unacceptable.

A similar situation linked to population growth has caused another major issue in hydraulic bridge design. Flood prone areas that were previously unutilized or underutilized are now becoming valuable real estate. Buildings and other infrastructure improvements are being placed in floodplains. This means that land which previously had little value has become expensive real estate. Increases in flooding due to bridges or highway encroachments suddenly become more important because the flood-prone areas are now quite valuable.

Population growth places an ever-increasing demand, and value, on the nation's transportation infrastructure. As the traffic volumes experienced on bridges continues to increase, bridge failures become more and more costly in the form of lost time and extra mileage traveled for an increasing number of drivers.

The increasing danger of flooding requires vigilance during the design process for engineering projects that impact waterways and floodplains. By far, the most common manmade structures in this type of environment are bridges and culverts. Structures which are designed with inadequate capacity to pass floods can cause a significant increase in upstream flooding. Several under-designed bridges or culverts in series compound the problem. Increased flooding causes increased damage to upstream development. There is direct economic motivation for optimum design of bridges. Under-designed bridges cause increased flooding, while over-designed bridges require resources from finite sources that may be more properly utilized elsewhere.

The Mirriam-Webster's Collegiate Dictionary (Mirriam-Webster, 2001) defines a bridge as 'a structure carrying a pathway or roadway over a depression or obstacle'. When this is applied to roadways three basic obstacles come to mind: waterways, railroads, and other roadways. Ironically, of the three basic obstacles that require bridges, the most common are also the most difficult to design. Of the 19,010 state or locally owned bridges in the state of Tennessee, 16,506 of them cross over waterways of some kind (Leatherwood, 2000). This means that 87% of the state's bridges span some type of waterway.

Most bridge designs have three basic components as discussed below. All facets must be coordinated to provide an optimum design.

- Roadway Design: This component consists of the horizontal and vertical alignments of the approach roadway and the bridge itself. The roadway geometry must be coordinated closely with the hydraulic design. This ensures an optimum hydraulic design that also meets current AASHTO safety requirements.

- Hydraulic Design: This portion of the design process ensures that the bridge is of sufficient size to pass the flood flow for the desired recurrence interval for the waterway it spans. This involves determining the abutment locations (which set the length of the structure), roadway grade elevations, clearance under the bridge, individual span lengths, and foundation depths.
- Structural Design: This involves the structural design of the various bridge components. The structural designer determines the construction material of the bridge, and designs individual members of the bridge such as the deck, beams, piers, and foundation to support the desired loads.

Hydraulic design is much more inexact than the other components of the design. The behavior of a concrete deck, or a steel beam is much easier to predict than the behavior of a dynamic river-watershed system that is constantly changing. The diverse nature of contributing watersheds and local topography makes it extremely difficult to predict flood levels, stream meandering, bank and bed erosion, sediment loading, and other important parameters.

This discussion is centered on the hydraulic design of the bridge, specifically, the modeling methods used to determine hydraulic data for the bridge. As previously discussed, rigorous hydraulic design is necessary in order to prevent flood damage to upstream or downstream properties. However, hydraulic design is also necessary to prevent failure of the bridge itself.

By far, the majority of all bridge failures in the United States are due to flood events. This means that the hydraulic design is crucial to the success of the bridge. The New York State Department of Transportation compiled a nationwide list of bridge failures and reported them by categories (Shirole and Holt, 1991). Table 1 presents these categories. More than half of all bridge failures were caused by hydraulic factors. The next highest factor, collision, led to less than one fourth of the number of failures caused by hydraulics.

Modes of hydraulic failure include scour, channel movement, debris or ice jam buildup, and embankment erosion due to overtopping. Scour is the removal of streambed material around the bridge opening and around piers and abutments due to high water velocities induced by contraction of flow at the bridge. Channel movement may be natural or induced by human activities within the watershed. Channel movement may cause failure

Table 1: Bridge Failure Modes

Failure Type	Number of Failures	Percentage
Hydraulics	494	60%
Collision	108	13%
Overload	84	10%
Fire	24	3%
Earthquake	14	2%
Other	99	12%
Total	**823**	**100%**

if the migration encroaches on a portion of the bridge substructure (i.e. the channel moves into a pier and undermines its foundation). Debris or ice jam buildup decrease the capacity of the bridge to convey floodwaters. This causes a buildup of water behind the blocked bridge opening and may create forces large enough to cause failure in the bridge piers or deck. Floating debris or ice may also cause impact damage to the bridge. Any of these methods may cause a bridge to fail.

As discussed above, the importance of hydraulic design has heightened over the past century. The rule of thumb methods used in early bridge hydraulic designs have evolved into procedures involving mathematical modeling of ever-increasing complexity.

The evolution of hydraulic design in Tennessee illustrates the complexity of this facet of the bridge design process. In the earliest days of bridge engineering, the bridge length was determined by the judgement and experience of the builder or engineer. This often involved simply spanning the main channel with little opening for relief on the floodplain. This design method resulted in bridges sufficient for normal flow, but with great potential for damage during moderate or heavy flood events. Upstream flooding was not a major problem in most cases, however insufficient flow capacity caused many bridges to simply wash out.

Another guideline from the early twentieth century was to place any bridge piers parallel to the stream flow pattern. Many early bridge designers preferred to place bridge piers on a 90° skew to the bridge deck. This often resulted in some unusual curves in the roadway approaches to the bridge. In some cases, the stream was relocated in order to provide a 90° skew. This practice resulted in straightening of stream bends or construction of meander cutoffs in many cases. This forced stream condition in turn causes degradation of the streambed, undermining of the channel banks, and stream widening.

In 1897 Dr. Arthur Talbot published an eight page paper titled "The Determination of Water-Way For Bridges And Culverts". This paper provided one of the first mathematical methods for estimating the area of bridge opening required. Talbot's equation is shown below (Tennessee Metal Culvert Company, 1937).

$$A = C * \sqrt[4]{M^3} \tag{1}$$

Where:
- A = required area of bridge opening (ft^2).
- M = watershed drainage area (acres).
- C = coefficient = 1/5 for flat watershed.
  - = 1/3 for rolling watershed.
  - = 1 for mountainous watershed.

The effectiveness of this method varied between bridge sites. Talbot also made no mention of the concept of frequency of design storms. Despite these limitations, Talbot's equation provided a guideline to bridge and culvert designers for almost half a century, and was an improvement over simply using the engineer's judgement.

Talbot's equation and engineering judgment were the two methods used for bridge design until the 1950's. During the late 1950's the need for improved methods was recognized.

During the late 1950's and into the 1960's the United States Geological Survey (USGS) provided support to the Tennessee Department of Transportation (TDOT). The USGS provided structure lengths and flow velocities for major structures, while Talbot's equation was used for routine projects.

In 1970 TDOT formed a Hydraulic Design Section. This section took over much of the work previously contracted to the USGS. Through most of the 1970's bridge design was done by hand. Bridge analysis was conducted using one surveyed cross-section. The standard step method was used for step backwater calculations and the effects of the roadway embankment encroaching onto the floodplain were computed by modifying the survey section to reflect proposed conditions. Energy losses due to the expansion and contraction of water flowing through the bridge and losses due to bridge piers were determined by empirical methods.

In the late 1970's and early 1980's hydraulic engineers began to use computers to assist in their design work. The Corps of Engineers and the Federal Highways Administration (FHWA) both introduced programs for computing flood profiles through bridges.

In 1976 the Corps Hydraulic Engineering Center (HEC) introduced HEC-2 'Water Surface Profiles' (HEC, 1982). This computer program was designed to compute water surface elevations along a stream or river reach. It was designed to accommodate bridges, culverts, dams, and weirs, as well as unconstricted reaches. HEC-2 provided two methods for computing flow profiles through bridges: the normal bridge and special bridge methods.

The normal bridge method computes a water surface profile through bridges by use of the energy equations and the standard step method. This method assumes energy losses are caused by flow contraction and expansion upstream and downstream of the bridge, and by friction. Water surfaces are computed by use of the standard step method while energy losses are added at the required places. Empirical methods are used to compute losses due to contraction and expansion of flow and friction losses are computed using Manning's n factor. The normal bridge method requires six river cross-sections to compute a water surface profile through the bridge.

The special bridge method uses a method developed by Yarnell for factoring in the hydraulic effects of bridge piers. This empirical method was developed based upon over 2,100 flume experiments utilizing various shapes and sizes of bridge piers. Based upon these experiments, pier coefficients were developed to account for the most common shapes of bridge piers. This method requires only four cross-sections for computations. The bridge opening is approximated by a trapezoid.

The energy equations used as part of the normal bridge method, and Yarnell's methodology will be discussed in some detail in Chapter III.

HEC-2 was the first widely used computer program for hydraulic design of bridges. It has been used extensively in the National Flood Insurance Program for developing flood elevations, mapping floodplains, and designating floodplain widths to be used in the production of flood hazard maps.

In 1986, the Federal Highways Administration introduced a new methodology for hydraulic calculations at bridges (Shearman, 1986) and a computer program, Water Surface PROfiles (WSPRO), based upon this methodology. WSPRO is similar to HEC-2, but while HEC-2 is intended for general flood profiles, WSPRO was specifically developed for bridge design applications. WSPRO utilizes the standard step method for unconstricted sections. At bridge locations WSPRO uses special empirical methods for determining bridge losses. These methods were developed by the USGS for specific use in WSPRO and are somewhat different from the methods used by HEC-2 (Shearman et al, 1986).

TDOT hydraulic designers adopted WSPRO in the early 1980's (Bennett, 2001). It proved very useful by automating part of the design process previously done by hand. However, it is not without its drawbacks. WSPRO and HEC-2 were developed originally for the punchcards used with early mainframe computers. With the advent of personal computers both were modified to use with personal computer operating systems. They utilize text only and are deficient in the area of graphical viewing of cross-sections and results. Debugging these can be daunting when faced with page after page crammed with text and numbers.

In the early 1990's computer software manufacturers introduced the concept of a graphical user interface. This type of interface represents files and objects as graphical icons. Introduction and popularization of the graphical user interface made it possible for software to use graphics extensively. As a consequence, software in general, and engineering software in particular, became much more user-friendly.

The Corps was quick to take advantage of this technological improvement. In 1995 HEC introduced the River Analysis System (RAS) (HEC, 1995). HEC's stated intention is for RAS to replace HEC-2. RAS provides capabilities similar to HEC-2. The major improvement however, is the addition of a graphical user interface. While requirements for data input by the user are similar between HEC-2 and RAS, the graphical capabilities of RAS provide great assistance in detecting bugs and errors in data input. Graphic capabilities for output data are much improved as well. Users can plot cross-sections and bridges and overlay water surface elevations as needed. This provides extensive help in visualizing situations and comparing alternatives. RAS also provides improved computation methods based upon new advances in hydraulic engineering theory since the introduction of HEC-2. Data requirements for RAS will be discussed in Chapter IV.

RAS has been part of the standard bridge design process at TDOT since late 1998. Reaction of TDOT engineers is mixed. RAS is much praised for its graphical capabilities, however developing a RAS bridge model is more time-consuming than with WSPRO.

Due to its increased flexibility and user-friendly graphics, RAS is becoming the method of choice for hydraulic bridge design. Based upon an informal survey conducted by the author of state Departments of Transportation in the southeast United States, WSPRO was the software of choice for the 1980's and early 1990's. The majority of state DOTs contacted are now using or considering the use of RAS for bridge designs.

Objectives

There are now several tools available for use in hydraulic modeling of bridges. Each of these various methods provides its own set of guidelines and assumptions for operation. The purpose of this work is assist the bridge engineer in determining the best of the numerous and sometimes conflicting guidelines to use. Since RAS is the newest and seemingly most popular hydraulic design tool, all analysis discussed here will be conducted using this method.

In the course of using any new software, the designer often encounters questions about the best way to utilize the capabilities provided. RAS provides many capabilities not previously available. Some guidance will be provided as to the proper selection of boundary conditions, cross-section spacing, and other variables. RAS also provides six distinct methods for computing water surface elevations through a bridge reach along with guidelines for which method to use. Each method will be evaluated in order to expand the guidelines provided by HEC and assist the user in determining the validity of each method.

As with any new design method, RAS has questions associated with it. The new methods used by RAS create questions concerning validity and the proper ways to utilize them. An example of this is the question of expansion and contraction of flow as it passes through a bridge. HEC-2, WSPRO, and RAS all have their own recommendations for determining where expansion and contraction begin and end. This can have a significant impact on design computations. Each of these recommendations will be evaluated to determine which is most accurate.

Recommendations concerning spacing of cross-sections for modeling are vague and inadequate. By analyzing the results of modeling an individual bridge reach with various different cross-sectional spacing some guidance may be provided as to the optimum spacing.

Selection of boundary conditions is an important part of any hydraulic modeling effort. RAS provides several possible boundary conditions for the user to choose from. The effects of selection of boundary values will be judged.

Data collected by the USGS will be utilized to assist in answering these questions. The USGS has provided hydrologic atlases for several bridge sites in the southern U.S. These atlases contain the information required to develop RAS models of each site. Once the RAS models are constructed, the various methods and recommendations discussed above will be tested. Validity of each method or technique will be determined by comparing model output to actual water surface elevations surveyed in the field.

Summary

This thesis is based upon the author's experiences in hydraulic design of bridges for TDOT. The author has discovered areas where further guidance in the proper use of RAS for bridge modeling is needed. The areas discussed by this thesis are:

- Determination of the expansion and contraction length and proper location of the approach and exit sections of the bridge reach.
- Proper use of the various methods of low and high-flow bridge analysis provided by RAS.
- Effects of the use of the cross-section interpolation feature provided by RAS.

- Selection of proper boundary conditions for sub-critical flow

The primary objective of this thesis is to provide guidance in the most effective use of RAS to hydraulic bridge designers.

Chapter II

Literature Review

The 1950's saw a boom in federal transportation funding with the beginnings of the Interstate system. In the interests of protecting its highway investment, the U.S. Department of Transportation began investing in research. This research carried over into all areas of highway design including bridge hydraulics. The Bureau of Public Roads (BPR), which is now known as the FHWA, became heavily involved in bridge and hydraulics research. FHWA conducts much of its research under contract with its sister agency, the USGS. These contracts led to the development of WSPRO. Many state Departments of Transportation (DOTs) also sponsor or conduct hydraulic research.

The U.S. Army Corps of Engineers is responsible for flood control in most watersheds throughout the country. The Corps has a broad mission in the area of flood control and floodplain management. In pursuit of this mission, the Corps has conducted much research in the area of river hydraulics. This has led to the creation of HEC-2, RAS, and several related programs. Corps research and applications have, of necessity, been of a more general nature than that of the FHWA.

These two agencies have been the major sponsors of hydraulic research since the 1950's. The final, or most current, results of this research are two flexible hydraulic modeling programs that are used and accepted by engineers throughout the country: RAS, and WSPRO.

Extensive documentation concerning RAS is available from HEC. An experienced RAS user will find the Hydraulic Reference Manual (HEC, 1997) the most useful of these. HEC provides a detailed discussion of the theory of RAS in this manual. It also contains recommendations for dealing with various modeling situations the user may encounter. Further details and discussion can be found within the course notes provided as part of RAS training classes offered by HEC and the National Highways Institute.

While HEC (1997) provides an overview of RAS's application of the WSPRO method, Sherman et al (1986) discuss the WSPRO methodology in detail as it was originally implemented. Shearman provides theoretical background and data requirements for using this method for bridge analysis.

Shearman also provides charts and tables for assistance in determining the discharge coefficient (K') which is required for the WSPRO analysis method. The concept of the discharge coefficient was first presented by Kindsvater, Carter, and Tracy (1953) for use in indirect measurement of flow through bridges. The authors present four categories of bridge constriction based upon the type of bridge abutment and roadway embankment. The base coefficient is determined based upon the type of bridge opening and the degree of floodplain constriction caused by the bridge. The base coefficient is then modified for several factors based upon charts developed empirically from laboratory data. These charts were later modified by Matthai (1967) to reflect additional data. A more detailed discussion of the discharge coefficient and its applications to bridge modeling may be found in Chapter III.

Brunner and Hunt (1995) performed a comparison of RAS, WSPRO, and HEC-2. Their study contains a discussion of the similarities and differences of the fundamental computational methods of each. Using a sample consisting of thirteen bridge sites located in Louisiana, Alabama, and Mississippi with seventeen flood flows they determined the mean average absolute error for each computation method by comparing calculated water surface elevations to observed field data. Table 2 below is excerpted from Brunner and Hunt. Based on these results they concluded that all three programs computed water surface elevations "within the tolerances of observed data".

The Bridge Reach

One element common to nearly all literature concerning bridge hydraulics is the concept of the bridge reach. The bridge reach is the portion of the river that contains the bridge. It is normally defined by a four to six cross-sections. Energy losses within the bridge reach are greater and much harder to predict than in an unconstricted river reach. As a consequence, water surface elevations may vary greatly within the bridge reach.

The Corps (1959) divides the bridge reach into three sections based upon the primary method of energy loss. These are the transition length downstream of the bridge

Table 2: Computed and Observed Water Surface Elevations
per Brunner and Hunt (1995).

Site Location	Flow (cfs)	Obs. WS Elev	RAS		HEC-2		WSPRO	
			WSEL (ft)	Error (ft)	WSEL (ft)	Error (ft)	WSEL (ft)	Error (ft)
Alexander Cr.	5508	88.4	88.2	-0.2	88.1	-0.3	88.3	-0.1
Alexander Cr.	9500	90.2	90.1	-0.1	90.0	-0.2	90.1	-0.1
Beaver Cr.	14000	217.8	217.9	0.1	217.8	0.0	217.3	-0.5
Bogue Chitto	25000	337.3	337.8	0.5	337.5	0.2	337.6	0.3
Bogue Chitto	31500	338.3	338.9	0.6	338.5	0.2	338.8	0.5
Buckhorn Cr.	4150	322.0	322.1	0.1	322.2	0.0	322.3	0.3
Cypress Cr.	1500	116.1	115.8	-0.3	115.7	-0.4	115.9	-0.2
Flagon Bayou	4730	76.3	76.2	-0.1	76.2	-0.1	76.9	0.6
Okatam Cr. Near Magee	16100	367.2	367.3	0.1	367.1	-0.1	367.3	0.1
Okatama Cr. East of Magee	12100	371.9	371.5	-0.4	371.5	-0.4	372.6	0.7
Pea Cr.	1780	359.1	358.9	-0.2	358.8	-0.3	359.4	0.3
Poley Cr.	1900	234.8	234.7	-0.1	234.6	-0.2	235.0	0.2
Poley Cr.	4600	237.2	237.2	0.0	237.2	0.0	237.6	0.4
Tenmile Cr.	6400	110.9	11.0	0.1	111.0	0.1	110.9	0.0
Thompson Cr.	3800	200.3	200.6	0.3	200.6	0.3	200.9	0.6
Yellow Riv.	2000	234.2	234.2	0.0	234.1	-0.1	234.3	0.1
Yellow Riv.	6603	237.3	237.7	0.4	237.5	0.2	237.8	0.5
Average Absolute Error:				**0.21**		**0.18**		**0.32**

crossing, the transition length upstream of the bridge crossing, and the width of the bridge. The Corps concluded that energy losses in the downstream reach are primarily due to expansion as the active flow area expands from the constricted bridge area to the larger unconstricted floodplain flow area. This downstream transition is referred to as the expansion reach. Losses in transition area upstream of the bridge are caused by contraction of the active flow area from the large floodplain into the smaller bridge area. This upstream bridge reach is called the contraction reach. Losses within the bridge area itself are due primarily to friction, impact, and eddies caused by the bridge piers and abutments. Exact computation of losses in the bridge length depends upon the method used. Figure 1 illustrates the sections of the bridge reach.

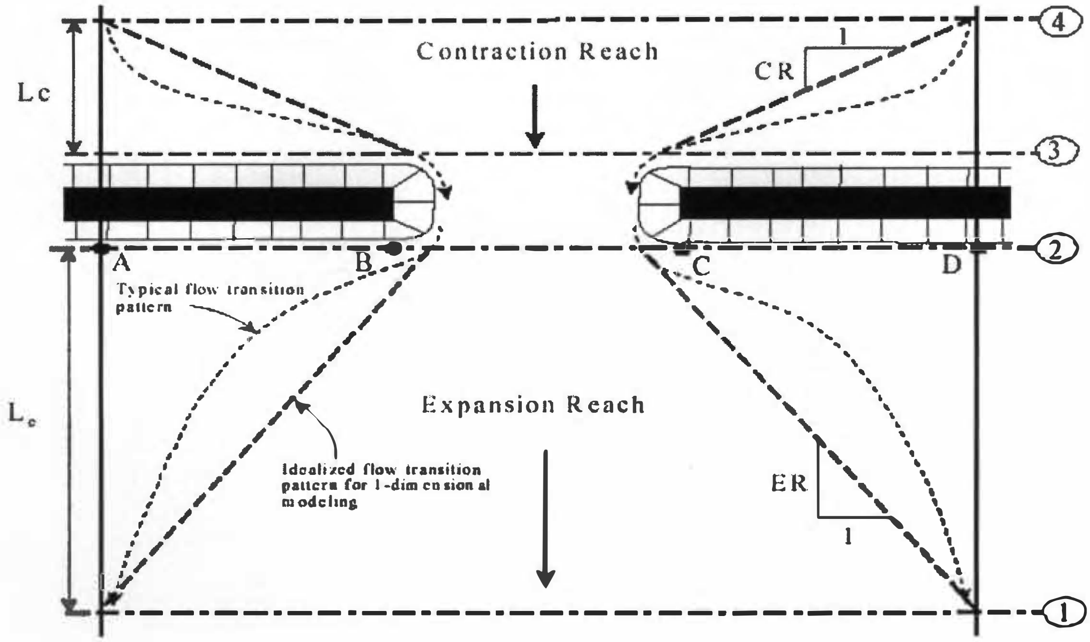


Figure 1: Bridge Reach (HEC, 1997)

As previously mentioned, a bridge reach is defined by a minimum of four cross-sections. The most downstream cross-section is located at the point where the active flow area has expanded to the full, unconstricted floodplain width. This is called the exit section (1). The most upstream section is located at the point where flow is just about to begin to contract from the full floodplain width to the width of the constricted bridge opening. This is referred to as the approach section (4). This topic will be discussed in additional detail in later chapters.

When modeling a bridge, it is generally advisable to include cross-sections some distance upstream and downstream of the bridge reach. This ensures that all other influences on the local water surface elevations are included.

Transition Lengths

The length of the bridge reach is a subject of much debate. All literature reviewed seems to be in agreement that the approach section should be located just before flow begins to contract and the exit section should be located just after flow has fully expanded. Flow lines should be approximately parallel at these sections. However, there are conflicting recommendations as to just where this occurs in relation to the bridge.

Matthai (1967), and Shearman (1986) recommend locating the approach section one bridge length above the upstream bridge face. This is the convention used by WSPRO.

Shearman considers this location to be the point of maximum backwater as well. Chow (1959) recommends the approach section be located at "the upstream end point of the backwater curve", but he does not provide guidance as to where this point is.

Shearman (1986) recommends the exit section be located one bridge length below the downstream face. Mathai (1967) does not require an exit section in his procedure. Chow (1959) defines the exit section as above, but does not provide quantitative guidance as to its location.

HEC takes an approach different from that of the FHWA work by Shearman. HEC recommendations are based upon the length of floodplain constricted by the bridge opening, referred to as the obstructed width. In Figure 1, above, the obstructed width is the distance from point A to point B or point C to point D, whichever is greater.

Documentation for the HEC-2 software (HEC, 1982) provides specific recommendations for locating the approach and exit sections: (1) the approach section should be located at a distance upstream equal to the obstructed length, (2) the exit section should be located downstream at a distance four times the obstructed length. Following this rule at sites with flat topography and wide floodplains may require the exit section to be located a mile or more downstream.

More recent studies sponsored by HEC have declared the method of four times the constriction length to be inaccurate. Hunt and Brunner (1995) feel that using a factor of four causes the expansion reach to be overestimated by a large amount. They developed regression equations to use for determining expansion and contraction lengths. The equations were developed based upon numerous generic two-dimensional models. Expansion and contraction lengths were determined based upon flow vectors within the two-dimensional models and regression analysis performed to obtain Equations 2 and 3. Equation 2 is for expansion length, and Equation 3 represents contraction length. Equations 2 and 3 are shown as developed by Hunt and Brunner in English units. If these are used for a S.I. unit system all values should be converted to the indicated units. Solution of these equations require an iterative process as obstructed length, Froude numbers, and overbank flow are dependent upon the expansion and contraction lengths.

$$L_e = -298 + 257\left(\frac{F_{c2}}{F_{c1}}\right) + 0.918(L_{obs}) + 0.00479Q \qquad (2)$$

Where: L_e = length of expansion reach (ft).
F_{c2} = main channel Froude number at section downstream of embankment.
F_{c1} = main channel Froude number at Exit section.
L_{obs} = average length of obstruction by roadway embankment (ft).
Q = total discharge (cfs).

$$L_c = 263 + 38.8\left(\frac{F_{c3}}{F_{c1}}\right) + 257\left(\frac{Q_{ob}}{Q}\right)^2 - 58.7\left(\frac{n_{ob}}{n_c}\right)^{0.5} + 0.161(L_{obs}) \qquad (3)$$

Where: L_c = length of contraction reach (ft).
Q_{ob} = discharge conveyed on overbanks (cfs) at approach section.
n_{ob} = the Manning n value for overbank at approach section.
n_c = the Manning n value for main channel at approach section.

As shown in the equations above, Hunt and Bonner found that lengths appeared to correlate well with Froude numbers in the channel. They recommend use of the provided regression equations for determining expansion and contraction lengths. They state that the contraction length should fall between 0.3 and 2.5 times the constriction and the expansion length should be between 0.5 and 4. The author's examination of numerous designs performed by the Tennessee Department of Transportation using these equations indicate that the expansion ratio is normally between 0.5 and 2.0 and the contraction ratio is normally from 0.5 to 1.5.

One purpose of this study is to determine which of these conflicting recommendations yields the most accurate results. RAS models were developed at multiple bridges. Separate analyses were developed using the recommendations made by RAS, HEC-2, and WSPRO. Comparison of the results and the observed data will help determine which of these methods is most valid.

Cross-section Spacing

Spacing of cross-sections within a hydraulic model is an issue of some importance. HEC and FHWA recommend that cross-sections be placed where the channel experiences some significant change (i.e. sudden channel widening or constriction). RAS has built-in provisions for monitoring this and a successful RAS run can often have numerous warnings concerning cross-section spacing. A large change in depth, conveyance, or velocity head triggers a warning to the user that cross-section spacing may be too great.

There is some discussion as to how often cross-sections should be placed. In their comparison of modeling software types Brunner and Hunt (1995) find location of cross-sections to be more important than the type of model used, however, they do not provide guidance concerning this. Gates et al (1998) performed a study of this issue. Numerous cross-sections on a river reach were surveyed. They then did a statistical analysis of how the various cross-section properties varied with different sampling resolutions. Average slope, cross-section area, and other hydraulic parameters were determined using cross-sections at various spacing resolutions. Average slope was shown to vary significantly when using small spacing increments, but this stabilized quickly at larger increments. They also found that differences in elevations over long distance appear to be influenced by large-scale trends, but differences over small distances appear to be nearly random.

Because physical surveying and mapping of a river reach is expensive, RAS has a built-in procedure for interpolating new cross-sections based upon the surveyed sections. An objective of this study is to determine the effects of using this interpolation routine to add sections. Some guidance concerning the effects of interpolating cross-sections at varied spacings will be provided based upon modeling of the study reaches with cross-sections spaced at varying intervals.

Selection of Bridge Modeling Method

RAS provides four different methods for modeling bridges in low-flow situations, and two for bridges in high-flow situations. Each of these will be discussed in detail in Chapter III.

RAS documentation (HEC, 1997) provides the following guidelines for selection of a low flow modeling method:

- Where losses are predominately friction and piers are a small obstruction, the energy, momentum and WSPRO methods may all be used accurately.
- Where pier losses are experienced in addition to friction, the momentum method is recommended.
- The Yarnell and WSPRO method are capable of modeling only subcritical flow. The energy or momentum methods must be used if flow passes through critical depth within the bridge reach.
- For supercritical flow, the momentum method is recommended where pier impact and drag losses are large.
- At bridges where piers are the major cause of energy loss the Yarnell and momentum methods are best.
- During high flows, when flow through the bridge is not pressurized the energy method is recommended.

- When the bridge deck and roadway embankments are a large obstruction the pressure and weir method should be used.
- When flow over the bridge and embankment is large, the energy method is best.

Further guidance concerning the selection of bridge modeling methods will be provided during the course of this study. The most valid method for each bridge site will be determined by comparing results of all methods to observed data. These results should provide some recommendations for selection of low and high flow methods.

Summary

Literature concerning hydraulic design and analysis with RAS is extensive. Mohammad et al (1998) discuss the application of RAS to a situation involving bridge construction and maintenance. The main purpose of Mohammad's work is discussion of the results of the modeling effort. This is typical of the available literature concerning RAS. Only results are discussed, with no real comments concerning development of the model. This thesis will attempt to correct that deficiency by providing practical guidance to hydraulic bridge designers.

Chapter III

Theory and Limitations of RAS

Mathematical modeling involves using a system of mathematical equations to represent a physical system. The user inputs data describing the various important components of the system and the data is processed through the model to determine the results. The software packages discussed in Chapter I are mathematical models specifically developed for open channel flow and hydraulic structures.

Mathematical models are a direct result of physical modeling. Theoretical and empirical equations used during mathematical modeling are based upon the results of physical modeling. It is important to realize that the controlling principals of a situation must be properly understood through theory, physical modeling, and observation of existing structures before valid mathematical modeling of the situation can be accomplished.

Mathematical modeling often requires making assumptions to simplify the system under consideration. These assumptions generally simplify calculations by eliminating factors which do not significantly affect the outcome. When developing a mathematical model extreme care must be taken in order to ensure that all assumptions are valid. This requires a good knowledge of hydraulic theory and its application to the situation being modeled.

Simplifying assumptions may also limit the situations in which a mathematical model may be used with validity. When using a mathematical model developed by others (i.e. RAS, or WSPRO), the user must understand the limiting assumptions and their effect upon the model results. This is a key factor when choosing which available modeling software to use for various applications. The user must understand the limiting assumptions in order to correctly apply the modeling software and interpret its results.

RAS is a state-of-the-art mathematical modeling program. This work explores the validity of some assumptions made during the development of RAS as well as resolving some conflicting recommendations on its use. A discussion of the limiting assumptions and the theoretical framework of RAS are provided below for the reader's benefit. This discussion is not comprehensive, and the reader should refer to the RAS Hydraulic Reference Manual (HEC, 1997) for more detailed information.

Open Channel Flow Theory

Chow (1959) divides the flow of water in a conduit into two major categories: open channel flow, and pipe (or pressure) flow. The differences between these two types of flow are based upon the principal forces experienced by each. Open channel flow has a free surface open to the atmosphere and is subject to only atmospheric pressure while pressure flow is subject to some external pressure. The principal forces acting upon open channel flow are gravity and inertia. The principal forces influencing pressure flow are inertia and shear forces.

The governing theory and equations for both categories are similar in nature. Figure 2 compares the two types of flow. Pressure flow is shown to the left in Figure 2. Piezometers are used to monitor the pressure within the pipe system. When a piezometer is inserted into the closed pipe system water rises to some depth above the pipe. This depth is dependent upon pressure within the system and is an indicator of pressure head. The water levels within the piezometer represent the hydraulic grade line which is a combination of pressure head and elevation head (gravitational forces). The energy grade line represents the total energy within the conduit due to pressure, elevation, and velocity of flow.

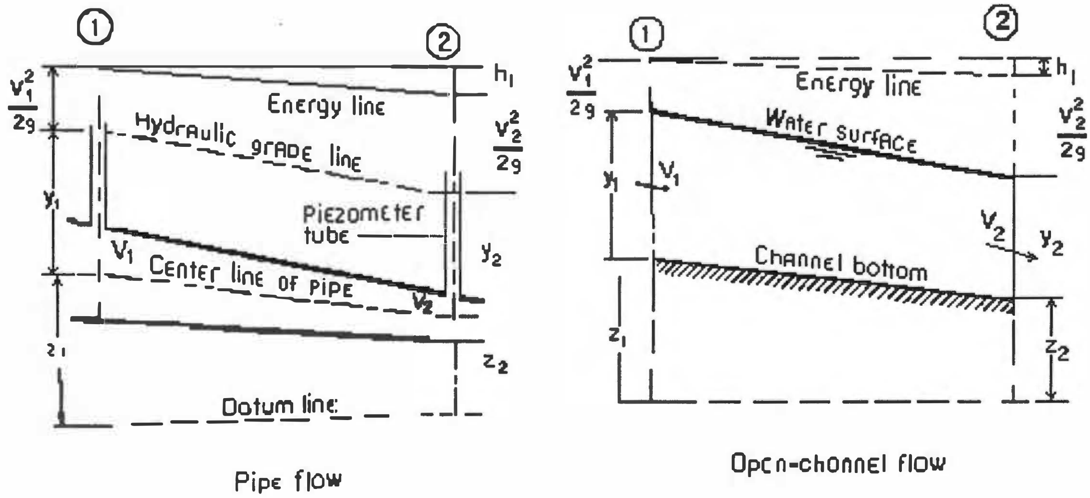


Figure 2: Pressure flow versus open channel flow (Chow, 1959)

Figure 2 also shows a similar diagram for open channel flow. Since open channel flow is subject only to atmospheric pressure, the depth of water represents the depth in the piezometers and the hydraulic grade line corresponds to the water surface.

The same governing principles apply to each, but open channel flow presents a much more complex problem than pipe flow. This is because the free surface of an open channel will likely change over time and distance along the channel reach. An additional complicating factor is introduced because flow, depth, and slopes of the channel bottom and free surface are closely related and the behavior of one will affect the behavior of the others.

Open channel flow is of primarily interest when designing bridges and RAS was developed to analyze open channel flow as it occurs in natural rivers and streams.

Tschantz (2000) provides the following definition of open channel flow:

> *Liquid flow (usually water) in a conduit and having a free surface open to the atmosphere & influenced by gravity, inertia, and some viscous forces.*

Open Channel Flow Regimes

Open channel flow is further classified depending upon whether depth and velocity change over time or space. If flow depth and velocity at a given location are unchanged over time then steady flow is present, if depth and velocity vary with time then flow is unsteady. Practically speaking, if flow does not change over a time period, then it is considered to be steady. Figure 3 illustrates steady and unsteady flow.

If flow depth and velocity at a given time are constant along the length of the channel then flow is classified as uniform. If, however, depth and velocity change with distance along the channel, flow is varied. Varied flow may be further classified by how rapidly depth changes over distance. Depth changes over a short distance, such as hydraulic jumps, are termed rapidly varied flow, and if changes occur over long distances flow is gradually varied. Figure 4 shows varied and uniform flow and Figure 5 illustrates the differences between gradually varied flow (GVF) and rapidly varied flow (RVF).

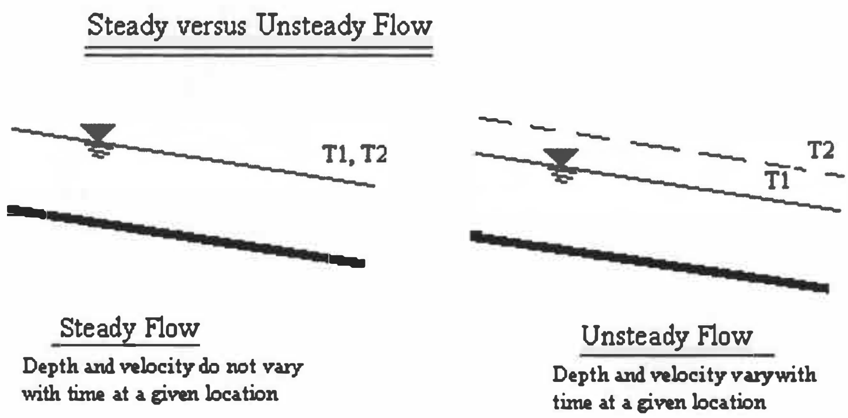


Figure 3: Steady And Unsteady Flow Regimes (HEC, 1997)

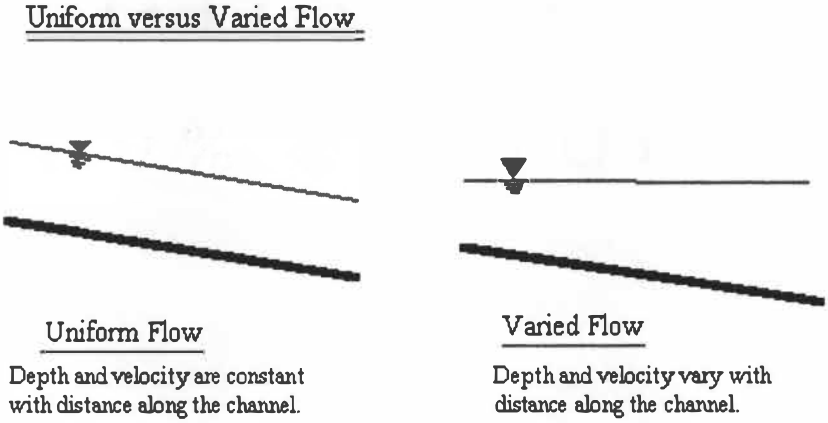


Figure 4: Uniform And Varied Flow Regimes (HEC, 1997)

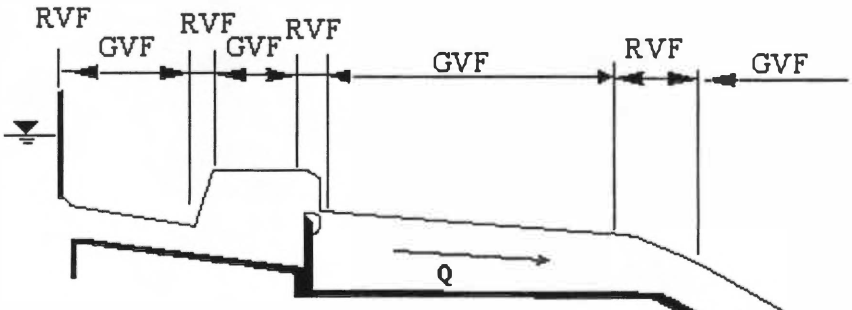


Figure 5: Gradually And Rapidly Varied Flow (Chow, 1959)

Open channel flow may be classified as any combination of steady versus unsteady and uniform versus varied (i.e. steady uniform flow or unsteady varied flow). Table 3 presents the various flow classifications.

Normal Depth

Normal depth is the depth in uniform flow (Daugherty et al, 1985). RAS uses normal depth for a boundary condition when specified by the user. The concept of boundary conditions is discussed further later in this chapter.

Table 3: Flow Classification Types (Tschantz, 2000)

Flow Type	**Uniform**	**Non-uniform**	
		GVF	**RVF**
Steady	Common assumption. Rarely found	Most common condition	Common local phenomenon
Unsteady	Rare condition	Fairly common	Less common than GVF

(GVF= gradually varied flow, RVF= rapidly varied flow)

RAS calculates normal depth with Manning's equation shown below.

$$Q = \frac{1}{n} A R^{2/3} s^{1/2} \tag{4}$$

Where: Q	=	discharge (m^3/sec).
n	=	Manning's roughness coefficient.
A	=	area of flow (m^2).
R	=	hydraulic radius (m) = A/P.
P	=	wetted perimeter of cross-section (m).
S	=	slope of energy grade line (often approximated by bed slope).

In order to simplify calculations, the concept of conveyance (K) is used. Conveyance is a measure of the carrying capacity of the channel independent of slope as shown below.

$$K = \frac{1.49}{n} A R^{2/3} \tag{5}$$

Solving Manning's equation for flow with a given depth is a simple procedure where the user calculates area and wetted perimeter at the given depth and substitutes them into Equation 4. Solving for depth for a given discharge is a much more complex process involving iterations and multiple guesses. The user must assume a depth, solve for area and wetted perimeter, and use these to solve Equation 4 for discharge. The calculated discharge is then compared to the given discharge. If the two are within a specific tolerance, then the assumed depth is normal depth. If not, then a new depth must be assumed and the process repeated until the calculated and given discharge are the same.

Energy Equation and The Standard Step Method

The total energy at a given river cross-section for a given discharge is generally expressed as a total head in feet of water (Chow, 1959). Energy in a cross-section consists of three basic elements: elevation head, pressure head, and velocity head. Elevation head is represented as the distance from the channel bottom to a horizontal datum. In open channel flow pressure head is simply the depth of the water. Velocity head is equal to $V^2/2g$. Figure 6 shows each of these factors. Equation 6 shows the energy head at a give cross-section.

$$H = z + y + \frac{V^2}{2g} \tag{6}$$

Where: H	=	total energy at cross-section (m of water).
z	=	distance from channel bottom to horizontal datum (m).
y	=	depth of flow (m).
V	=	velocity of flow (m^2/sec).
g	=	gravitational acceleration (9.81 m/sec^2).

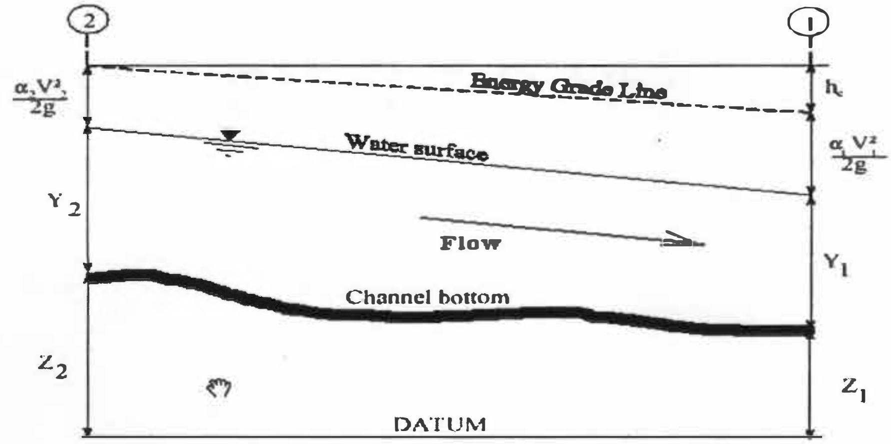


Figure 6: Variables Of The Energy Equation (HEC, 1997)

RAS uses the energy equation to solve for water surface elevations in a stream channel unconstricted by hydraulic structures. The energy equation states that the total energy in an upstream cross-section is equal to the total energy of the next downstream cross-section plus total energy losses. This concept is shown as Equation 8 below. Figure 6 represents this situation graphically.

$$Y_2 + Z_2 + \frac{\alpha_2 V_2^2}{2g} = Y_1 + Z_1 + \frac{\alpha_2 V_2^2}{2g} + h_e \tag{8}$$

Where: Y_1, Y_2	=	Depth of water at respective cross-section (m).
Z_1, Z_2	=	Channel invert elevations (m).
α_1, α_2	=	Velocity head weighting coefficients.
g	=	Gravitational acceleration constant (9.81 m/sec^2).
h_e	=	energy loss (m).
1,2	=	River cross-sections.

Energy losses are due to a combination of friction and contraction or expansion as shown:

$$h_e = L\bar{S}_f + C\left|\frac{\alpha_2 V_2^2}{2g} - \frac{\alpha_1 V_1^2}{2g}\right| \tag{9}$$

Where: L	=	Slope length (m).
S_f	=	Friction slope between sections 1 and 2.
C	=	Expansion or contraction loss coefficient.
α_1, α_2	=	Velocity head weighting coefficients.

Friction slope is computed by solving Manning's equation for slope. Equation 10 shows this calculation in terms of conveyance.

$$S_f = \left(\frac{Q}{K}\right)^2 \tag{10}$$

Solving the energy equation to determine y_2 requires an iterative process since several of the variables are interdependent. RAS solves Equation 8 through a process known as the standard step method. This process is summarized below based on information from RAS documentation (HEC, 1997).

(1) A depth at the upstream cross-section is assumed.

(2) Conveyance and velocity head are calculated based upon the depth assumed in (1) above.

(3) S_f is computed with Equation 10 and Equation 9 is solved to determine h_e.

(4) Equation 8 is solved for y_2, using values computed from (1) through (3) above.

(5) The computed and assumed depths are compared. If they agree to within 0.003 meter or a tolerance defined by the user, then the process may move to the next cross-section. If computed and assumed depths do not agree, then the process is repeated from step (1).

The process of determining water surface elevations in a river reach one of the cross-sections begins at the most downstream section and proceeds upstream for subcritical flow, and vice versa for supercritical flow. Since standard step calculations require one water surface to be known prior to beginning calculations, the user must specify flow and depth conditions at the boundary cross-section. This user-specified condition enables RAS to determine the depth at the boundary cross-section so that the standard step method may be used. The concept of boundary conditions will be discussed further later in this chapter.

Critical Depth

Open channel flow may also be divided into subcritical and supercritical flow. Subcritical flow is generally deeper and slower flowing than supercritical flow. In order to understand the difference between sub and supercrtical flow the concept of specific energy must be discussed.

Chow (1959) defines specific energy as "the energy at any section of a channel measured with respect to the channel bottom". In practical terms, this is the depth of flow plus velocity head as shown in Equation 11 and is expressed in feet or meters of water.

$$E = y + \frac{V^2}{2g} \tag{11}$$

Where: E	=	specific energy (m).
y	=	depth of flow (m).
V	=	flow velocity (m/sec).
g	=	gravitational constant (9.81 m/sec^2).

For any give energy, Equation 11 can be solved for two separate but equally valid depths. A smaller depth results in a larger velocity, while deeper flow is slower moving. Figure 7 illustrates this phenomenon by plotting flow depth versus specific energy for a given cross-section geometry and discharge. As Figure 7 indicates, the specific energy curve is a parabola asymptotic to lines representing Y = E and Y=0.

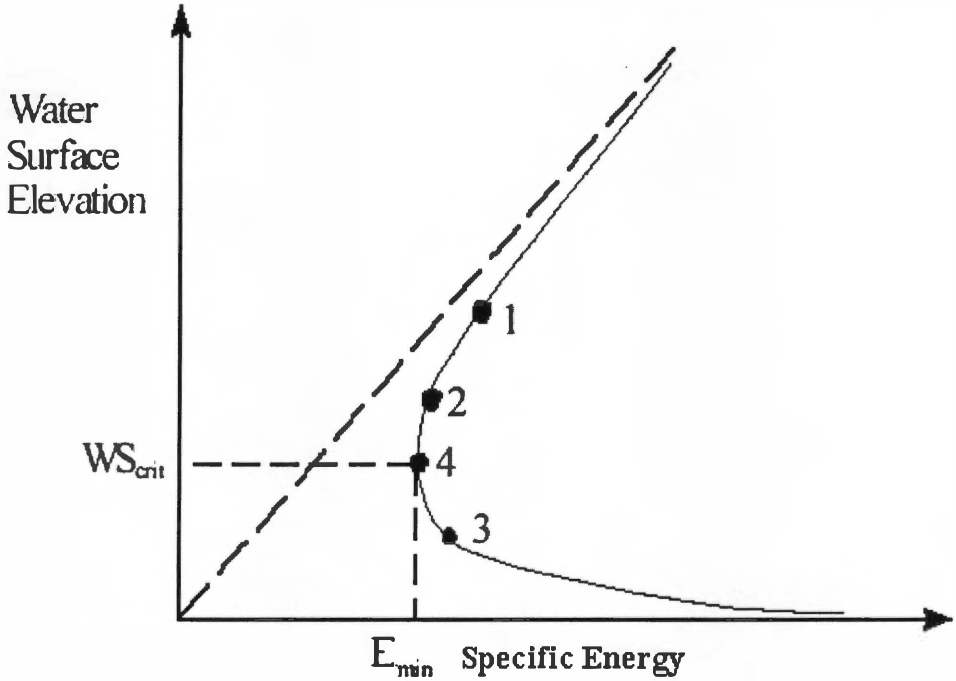


Figure 7: Specific Energy At A Cross-Section (modified from HEC, 1997)

For a given discharge and cross-section geometry, the depth of flow at minimum specific energy is called critical depth. If flow depth is greater than the critical depth, flow is subcritical. If flow depth is less than critical, then flow is supercritical. The majority of natural channels stay in a subcritical flow regime. Supercritical flow may be caused by very steep bed slopes, sudden channel constriction, or other radical changes in hydraulic parameters.

RAS calculates critical depth at a cross-section through an iterative process. A depth is assumed and the corresponding value of specific energy is calculated. This procedure is repeated until a depth which corresponds to the minimum specific energy is determined.

Choice of a flow regime is very important within RAS. As previously discussed, solution of the standard step method must proceed upstream for subcritical flow and downstream for supercritical flow. This means that the user must accurately indicate the type of flow for the river reach being analyzed. RAS performs calculations of critical depth at the reach boundaries in order to ensure the correct flow regime has been specified.

Momentum Equation

RAS uses the energy equation for most computations, but the energy equation is invalid if flow passes through critical depth. The energy equation is also not applicable in areas of rapidly varied flow. In these situations RAS must use empirical methods or the momentum equation. Empirical methods are available at hydraulic structures such as bridges, drop structures, or weirs. In other situations the momentum equation must be used.

This equation is based upon Newton's second law of motion: *force equals mass times acceleration.* Applying this to a body of water bounded by two cross-sections means that the mass flow rate is equal to the sum of external forces acting upon the body. This is shown in Figure 8 and Equation 12.

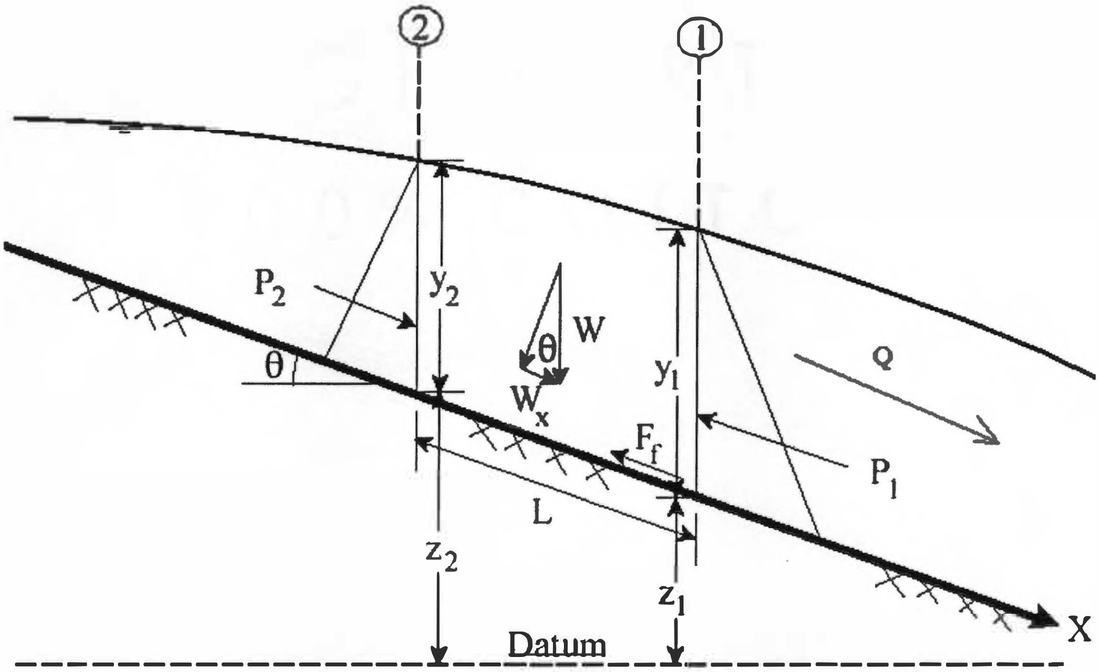


Figure 8: Momentum Definition Sketch (HEC, 1997)

$$P_2 - P_1 + W_x - F_f = Q\rho\Delta V_x \qquad (12)$$

Where: P	=	hydrostatic force at sections 1 and 2.
W_x	=	force due to the weight of water in X direction.
F_f	=	force due to external friction losses from 2 to 1.
Q	=	discharge.
ΔV_x	=	change in velocity in the X direction from 2 to 1.

The assumption of hydrostatic pressure is valid for channels with a slope less than 1:10. This includes nearly all natural channels, therefore P is represented by Equation 13.

$$P = \gamma A Y_{ce} \tag{13}$$

Where: A = Wetted area of cross-section (m^2).
γ = Unit weight of water (999.6 kg/m^3).
Y_{ce} = Depth from water surface to centroid of cross-section area (m).

Weight of water (W) is the unit weight of water multiplied by the volume of water.

$$W_x = \gamma\left(\frac{A_1 + A_2}{2}\right) L S_o \tag{14}$$

Where: L = Distance between sections 1 & 2 (m).
S_O = Slope of channel

External friction force (F_f):

$$F_f = \gamma\left(\frac{A_1 - A_2}{2}\right) S_f L \tag{15}$$

Where: S_f = Slope of energy grade line (friction slope).

Mass times acceleration:

$$ma = Q\rho \Delta V_x = \frac{Q\gamma}{g}(\beta_2 V_2 - \beta_1 V_1) \tag{16}$$

Where: β = momentum correction coefficient for varying velocity distribution in irregular channels.

Equations 13 through 16 are substituted into Equation 12 with the assumption that flow may vary from section 1 to section 2. This leads to Equation 17 which is the functional momentum equation as used by RAS.

$$\gamma A_1 Y_1 - \gamma A_2 Y_2 + \gamma\left(\frac{A_1 + A_2}{2}\right) L S_o - \gamma\left(\frac{A_1 + A_2}{2}\right) L S_f = \frac{Q_2 \gamma}{g} \beta_2 V_2 - \frac{Q_1 \gamma}{g} \beta_1 V_1 \qquad (17)$$

Application of the momentum method at a bridge is discussed further later in this chapter.

RAS Limiting Assumptions

Steady, Gradually Varied Flow Assumption

The most basic assumption of RAS is that all flow is steady and gradually varied except at bridges, culverts, or other hydraulic structures (HEC, 1997).

The assumption of steady flow means that the user may only input one flow per profile computation. This does not accurately represent the actual hydrology of a natural flood. Floods begin with lower base flows, increase to a peak, and then recede back to base flow. When using RAS to model a natural flood, the normal procedure is to model the only the peak flow for a design-driven recurrence interval. The peak flow is determined from a study of the watershed hydrology. This assumes that the highest water surface elevations are obtained at the highest flow. RAS may be used to model natural flow events by using this peak flow method, but care must be taken to determine that this is an accurate method for each river reach modeled.

At the time of this writing the most current version of RAS is 2.2. HEC is currently developing version 3.0, which will have a new module for performing unsteady flow analysis. This discussion applies only to the steady flow module of RAS.

The assumption of gradually varied flow requires that the streamlines be approximately parallel. If this condition is met, then the energy equations discussed above for uniform flow equations discussed above are a reasonable approximation for the gradually varied flow conditions. RAS uses the energy equation and standard step method as previously discussed to solve depths for gradually varied flow.

In a natural river reach this condition may be satisfied by properly spacing cross-sections. Sections should be spaced so that changes in cross-sectional area and depth are small. If a large change in cross-section hydraulic properties occurs between sections, then additional sections should be inserted so that changes are small from one section to the next. RAS assumes that the energy and momentum equations developed for uniform flow are also are a valid approximation for gradually varied flow.

The assumption of gradually varied flow becomes invalid at bridges or other hydraulic structures. Structures cause rapidly changing flow depths and velocities. In many cases flow transitions between sub-critical and super-critical. RAS uses empirical equations, such as the weir equations, at these locations. If empirical equations are not valid, then a momentum balance method is used. These are discussed in further detail throughout this chapter.

One-Dimensional Flow Assumption

Various hydraulic modeling software packages model one, two, or three dimensions. The three dimensions are described below and illustrated in Figure 9.

- The X dimension is the direction of longitudinal flow along the stream flow line.
- The Y dimension represents width of the cross-section perpendicular to flow.
- The Z-axis represents depth of flow.

One-dimensional models perform calculations only in the longitudinal flow direction (X dimension). The water surface elevation and velocity are assumed to be constant

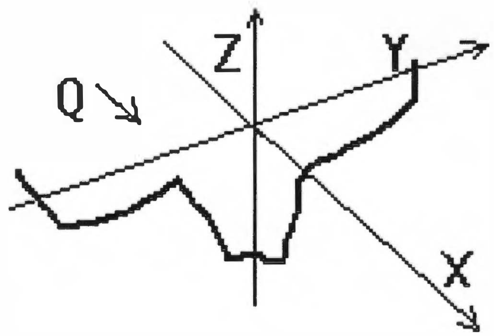


Figure 9: Dimensions Of Flow

throughout the cross-section. This condition is rarely found in nature as river and floodplain sections. Changes in natural topography cause contractions and expansions of flow. Flow velocity in a stream cross-section varies depending upon the location within the section similar to that shown in Figure 10. Boundary roughness causes flow near the ground surface to be slower than flow in the center of the channel. This is especially significant in situations involving river bends, where flow depth and velocity are much higher near the outside bank.

Two-dimensional models are able to calculate changes in velocity and elevation at different points perpendicular to flow (Y dimension) within a cross-section as well as in the longitudinal (X dimension) direction along the stream reach. These types of models also provide velocity vectors showing the direction of flow at each computational point. Two-dimensional models are a better approximation of actual conditions, but velocity direction and magnitude are considered to be the same at all depths.

The most common two-dimensional models available currently are FESWMS and RMA-2. FESWMS is a two-dimensional hydraulic model available from the FHWA that uses a finite element solution method. RMA-2 uses a finite difference solution method and is available from the Corps. The Surfacewater Modeling System (SMS) available from Brigham Young University provides excellent input and output processing and graphic capabilities for both FESWMS and RMA-2.

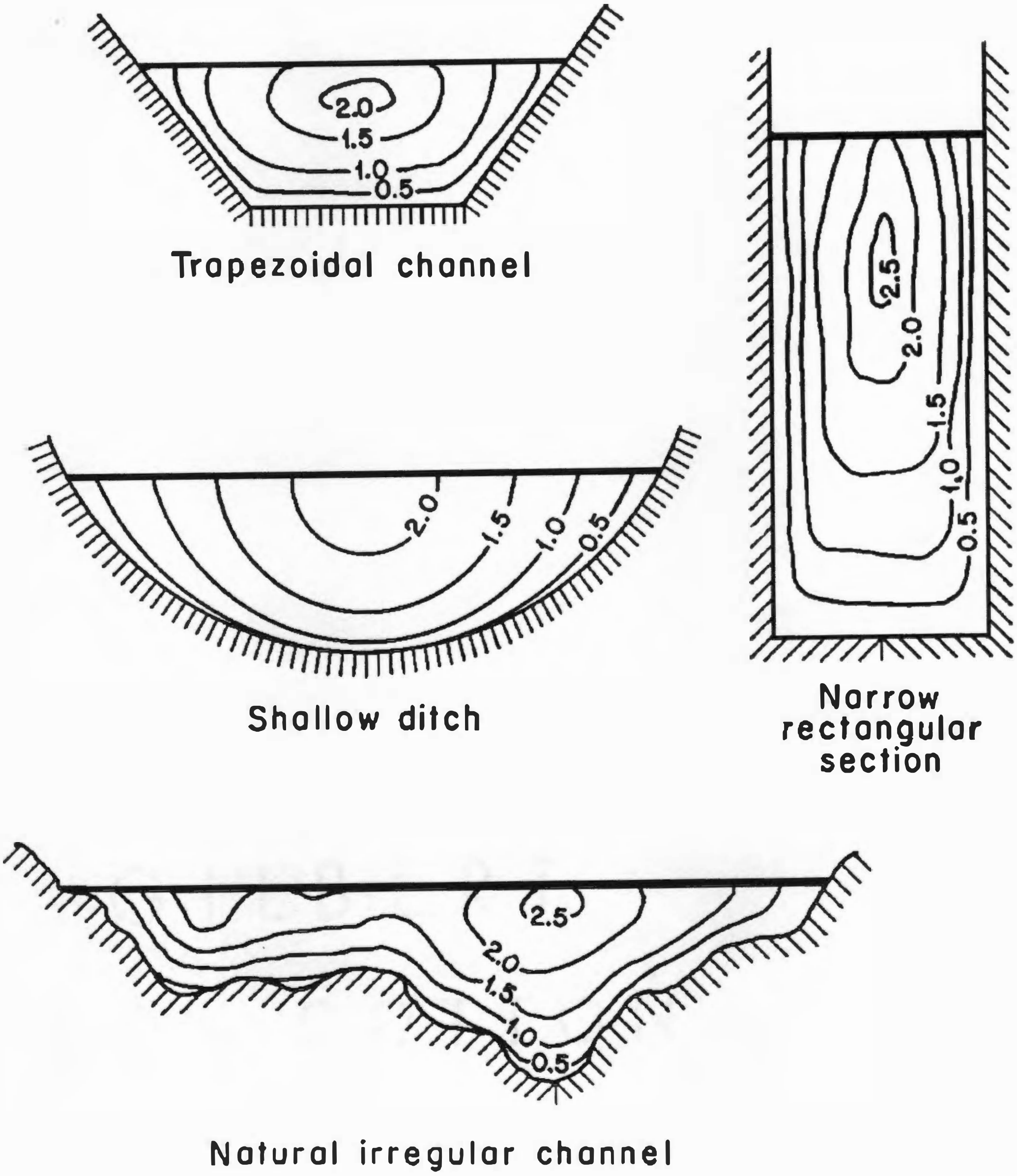


Figure 10: Typical Velocity Distribution For Cross-Section Shapes (Chow, 1959)

If variations with depth (Z dimension) are required as well, then a 3-dimensional model must be developed. Three-dimensional models predict changes in velocity and elevation in all three dimensions. The author is unaware of any software packages commercially available for three-dimensional hydraulic modeling.

Each additional dimension increases the complexity of the modeling calculations. Each one requires additional resources for data gathering and computing power. The increasing complexity requires a larger investment of time from the designer as well. As a result, one-dimensional models are the easiest and cheapest to develop. For most design situations one-dimensional models are adequate

Due to economic reasons, the majority of all bridge modeling is done using 1-dimensional modeling software, such as RAS. This may introduce some error in water surface elevations; however, in most cases this is considered insignificant. Two-dimensional models may be warranted in situations with large longitudinal variations in flow such as sinuous river channels or where curved channels create centrifugal effects on transverse water surface. Bridge crossings with unusual flow patterns or multiple openings may also require two-dimensional modeling. Three-dimensional modeling is especially useful for small-scale studies around piers or abutments, but is rarely used for design purposes.

RAS Bridge Cross-sections

Four separate cross-sections are required for modeling of bridges in RAS. Figure 11 shows recommended cross-section locations. These sections are collectively referred to as the bridge reach. Cross-sections are required at the downstream toe (2) and upstream toe (3) of the road embankment. A cross-section is required upstream at the point (4) where flow is just about to begin contraction to enter the bridge. The final required cross-section is located downstream at the point (1) where flow has expanded to its full effective width after passing through the bridge contraction. Cross-section 1 is commonly referred to as the exit section, and cross-section 4 is referred to as the approach section. These names are due to the fact that flow is approaching or exiting the bridge reach at each section.

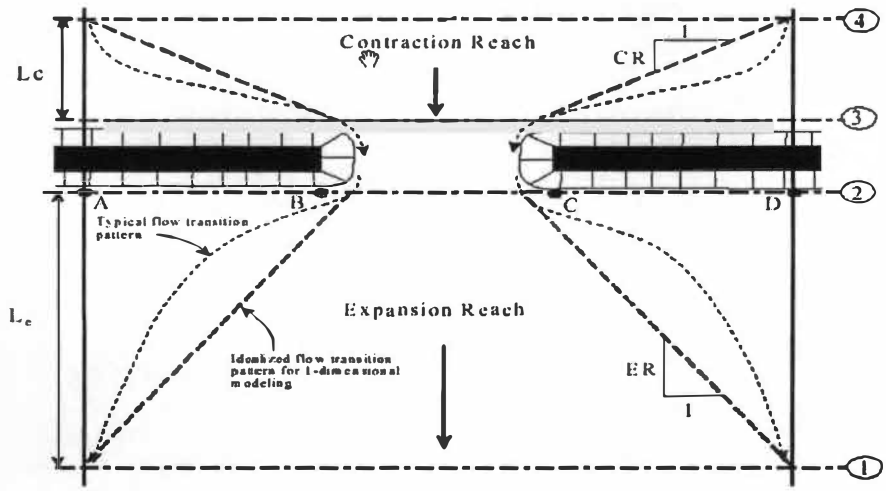


Figure 11: Bridge Reach Cross-Sections (HEC, 1997)

Location of the approach and exit sections is a questionable issue. Many different recommendations for locating the approach and exit section are available. HEC-2, WSPRO, and RAS all have their own recommendations for locating these sections. These recommendations conflict with each other and are discussed in detail in Chapter II. These best of these conflicting recommendations will be determined as part of this thesis.

Contraction and expansion as flow passes through a bridge causes portions of the cross-sections within the bridge reach to be ineffective. Ineffective flow areas do not conduct flow, but do provide flood storage. When inserting cross-sections, the user must represent the ineffective flow areas as shown by the dashed lines in Figure 11. RAS provides several methods for delineating ineffective flow areas. These areas may become effective if the water surface increases above a user specified elevation.

Two cross-sections are required in addition to the four cross-sections provided by the user. The program develops these two additional cross-sections just inside the upstream and downstream face of the bridge. These are developed by using sections 2 & 3 discussed previously, and overlaying bridge features upon these. See Figure 12 for details. These cross-sections are referred to as BD (bridge downstream) and BU (bridge upstream).

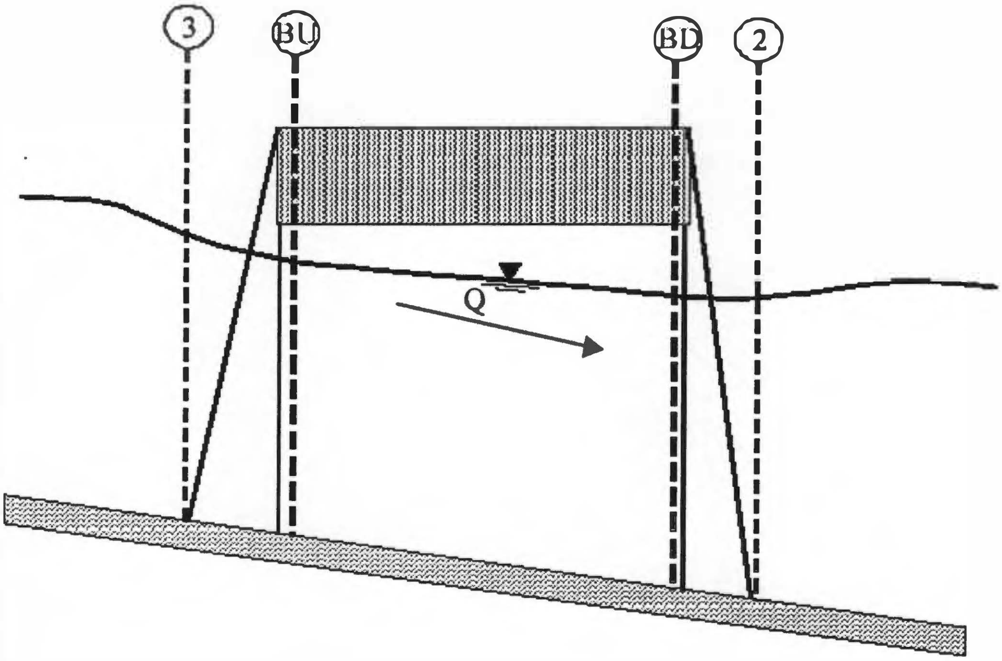


Figure 12: Bridge Cross-Sections (HEC, 1997).

Total energy losses through a bridge reach are a combination of loss in the expansion length, loss through the bridge, and loss in the contraction length. Energy losses in the expansion and contraction sections are calculated as frictional losses and expansion/ contraction losses. Expansion and contraction losses are calculated based on empirical coefficients input by the user. Friction losses are calculated based upon a weighted friction slope and weighted length between cross-sections. Methods for calculating bridge losses are discussed below.

RAS Low Flow Bridge Computations

RAS provides several different methods for analysis of reaches containing bridges. Separate methods are provided for low flow and high flow. A low flow condition exists when all flow through the bridge passes under the bridge low chord as shown in Figure 13 below. If flow comes into contact with the bridge deck, then a high flow condition exists. RAS provides four methods for computing bridge losses during low flow, and two for high flow. The discussion below applies only to Class A low flow, which is the most commonly experienced low flow condition. Class A low flow is defined as a situation where flow through the bridge is subcritical throughout the bridge reach. HEC provides guidance for situations which involve critical or supercritical flow (HEC, 1997).

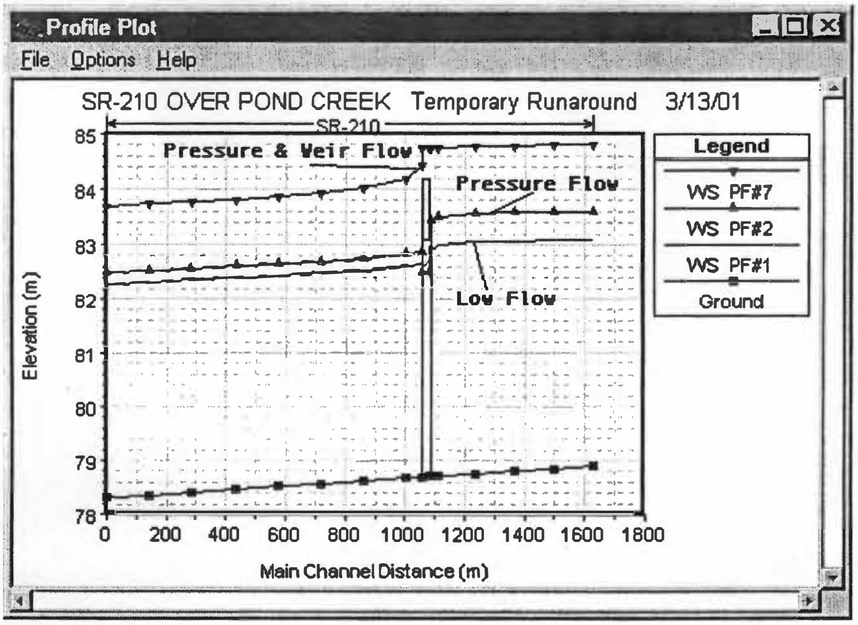


Figure 13: Flow Types

Energy Method

The energy method is essentially the same as the previously discussed standard step method. The cross-section parameters are modified based on the presence of the bridge. The flow area and wetted perimeter are modified based on portions of the bridge that are submerged. This method does not account for pier drag losses or pier and abutment shapes.

Momentum Balance Method

The momentum balance method performs a momentum balance through the bridge based on Equation 17 as previously discussed. The balance is conducted from section 2 to section BD, then from section BD to section BU, and finally from section BU to section 3. For a more details concerning the momentum balance method, see the previous discussion.

During the final momentum balance from section BU to section 3, the pier drag coefficient (C_D), is introduced. This is an empirical coefficient dependent upon pier shape and is used to determine energy losses due to drag and flow separation caused by the bridge piers. Recommended values for this coefficient range from 0.29 for elliptical piers to 2.00 for square nose piers. Pier losses as determined by the momentum method are shown below.

$$H_P = \frac{1}{2} C_D \frac{A_{P_{BU}} Q_3^2}{g A_3^2} \tag{18}$$

Where: H_P = Energy loss due to pier (m).
A_3 = Effective flow area at respective sections (m^2).
Q_3 = Discharge at section 3 (m^3/sec).
A_{PBU} = Area obstructed by bridge pier on upstream side (m^2).
g = Gravitational acceleration constant (9.81 m/sec^2).
C_D = Drag coefficient for flow at piers.

As Equation 18 shows, pier losses are heavily dependent upon flow, and flow area through the bridge opening. This means that pier loss is heavily dependent upon velocity through the bridge, and area obstructed by the bridge.

Yarnell Method

The Yarnell method is based upon an empirical equation developed from approximately 2,600 laboratory experiments conducted with various pier shapes, pier widths and lengths, and angles of attack. In this method change in water surface between section 2 and section 3 is calculated by Yarnell's equation as shown below.

$$H_{3-2} = 2K(K + 10\omega - 0.6)(\alpha + 15\alpha^4)\frac{V_2^2}{2g} \tag{19}$$

Where: H_{3-2} = Change in water surface from section 3 to 2 (m).
K = Yarnell's pier shape coefficient.
ω = Ratio of velocity head to depth at section 2.
α = Area obstructed by piers divided by total unobstructed area at section 2.
V_2 = Velocity downstream at section 2 (m^2/sec).

Yarnell's equation is especially sensitive to the pier shape coefficient, K, which varies from 0.90 for piers with semi-circular nose and tail to as much as 2.50 for ten pile trestle bents. Yarnell's equation does not take into account bridge width, shape of bridge opening, or bridge abutments. HEC recommends that the Yarnell method only be used in situations where the majority of energy losses are caused by the bridge piers.

WSPRO Method

The fourth low flow computation method in RAS, the WSPRO method, was adapted from the WSPRO computer program previously discussed. Several modifications were required in order to adapt WSPRO methods to the RAS conceptual framework. The WSPRO energy balance equation from section one to section four is shown below as Equation 20.

$$h_4 + \frac{\alpha_4 V_4^2}{2g} = h_1 + \frac{\alpha_1 V_1^2}{2g} + h_{L(4-1)} \tag{20}$$

Where: h	=	Water surface elevation at section 1 or 4 (m).
V	=	Velocity at section 1 or 4 (m^2/sec).
$H_{L(4-1)}$	=	Energy losses from section 4 to section 1.
α_1, α_4	=	Velocity head weighting coefficients.

Energy losses are calculated between each of the six bridge reach sections and summed in order to solve Equation 20.

Equation 21 shows the frictional loss computation between sections one and two, and Equation 22 shows expansion loss computations. Total losses between sections one and two are a combination of each of these.

$$h_{f(1-2)} = \frac{BQ^2}{K_2 K_1} \tag{21}$$

Where: $h_{f(1-2)}$ = Total friction losses (m).
B = Flow distance (m).
Q = Flow (m^3/sec).
K_2, K_1 = Conveyance at sections 1 and 2.

$$h_e = \frac{Q^2}{2gA_1^2}\left[2\beta_1 - \alpha_1 - 2\beta_1\left(\frac{A_1}{A_2}\right) + \alpha_2\left(\frac{A_1}{A_2}\right)^2\right] \quad (22)$$

Where: A_1, A_2 = Flow area at section 1 and 2(m^2).
α, β = Momentum correction factors for nonuniform flow.

The momentum correction factors are calculated as a function of conveyance and area in open channel sections. However, WSPRO utilizes a special method for relating these correction factors to bridge geometry. An empirical coefficient of discharge, C, is used as shown below. Computation of the discharge coefficient is discussed later in this chapter.

$$\alpha_1 = \frac{1}{C^2} \quad (23) \qquad \beta_1 = \frac{1}{C} \quad (24)$$

Energy losses between sections two and three are due to friction only and computed from section two to BD, from BD to BU, and from BU to section three. Equation 25 shows the computation as applied between BD and BU.

$$h_{f(BU-BD)} = \frac{L_B Q^2}{K_{BU} K_{BD}} \quad (25)$$

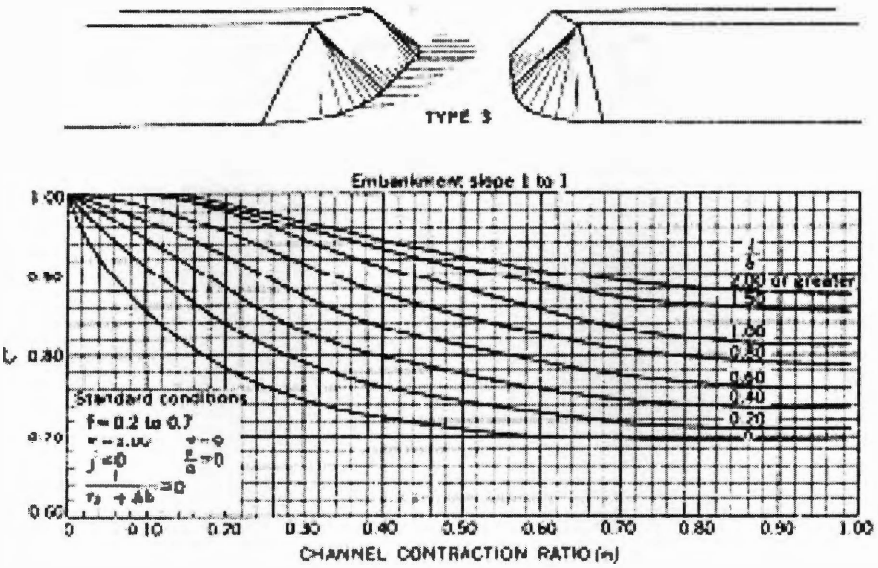
Where: K_{BU}, K_{BD} = Conveyance at respective sections.
L_B = Length between sections (m).
Q = Flow (m^3/sec).

Energy losses from section three to section four are also due to friction only. They are computed as shown above in Equation 25. However, the contraction flow length is computed in a complex procedure as discussed below.

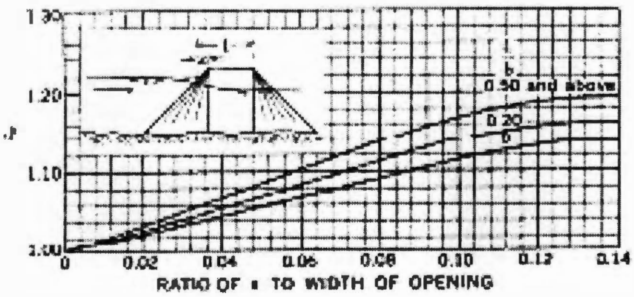
After energy losses between each of the six cross-sections in the bridge length have been computed, the total change in water surface elevation is computed using Equation 20. The most significant differences between the WSPRO method and the other computational methods provided by RAS are the use of a coefficient of discharge, and computation of the contraction flow length.

The base coefficient of discharge is determined by the channel contraction ratio and a ratio of flow length through the bridge to the bridge opening width. Once this base coefficient is determined, several correction factors are applied. Which correction factors are required is dependent upon the type of bridge opening. Four opening types are defined based upon embankment and abutment geometry. Most bridges fit into one of the four type categories listed below. Figure 14 is a chart used to determine coefficient of discharge for type 3 bridge openings.

- Type 1 bridge openings have vertical embankments and abutments with or without wingwalls.
- Type 2 openings have sloping embankments with vertical abutments and no wingwalls.



a) Base coefficient of discharge.



b) Unwetted abutment adjustment factor.

Figure 14: Coefficient Of Discharge Chart
For Type 3 Bridge Opening (Shearman et al, 1986)

- Type 3 openings have sloping embankments and spillthrough (sloping) abutments.
- Type 4 bridge openings have vertical abutments, sloping embankments and wingwalls.

Factors for correcting the base coefficient of discharge are provided for wingwall lengths, wingwall angles, average depth of flow, and entrance geometry.

Calculation of the coefficient of discharge is a complex process only briefly discussed here. This discussion borrows heavily from FHWA (FHWA, 1986) and the reader is referred to that publication for further details.

Contraction flow length is based upon a method not used in other RAS bridge computation schemes. The contraction reach is divided into twenty equal conveyance stream tubes. Width of the stream tubes becomes smaller as they pass through the bridge opening. Stream tubes near the edge of the floodplain must be longer due to the contraction to enter the bridge. The contraction length used to determine friction losses is an average of all twenty stream tube lengths. Figure 15 shows a definition sketch of the streamline concept. In Figure 15, b is the bridge width, while B is the floodplain width. This method of determining contraction length results in much greater contraction losses at bridges experiencing higher degrees of contraction.

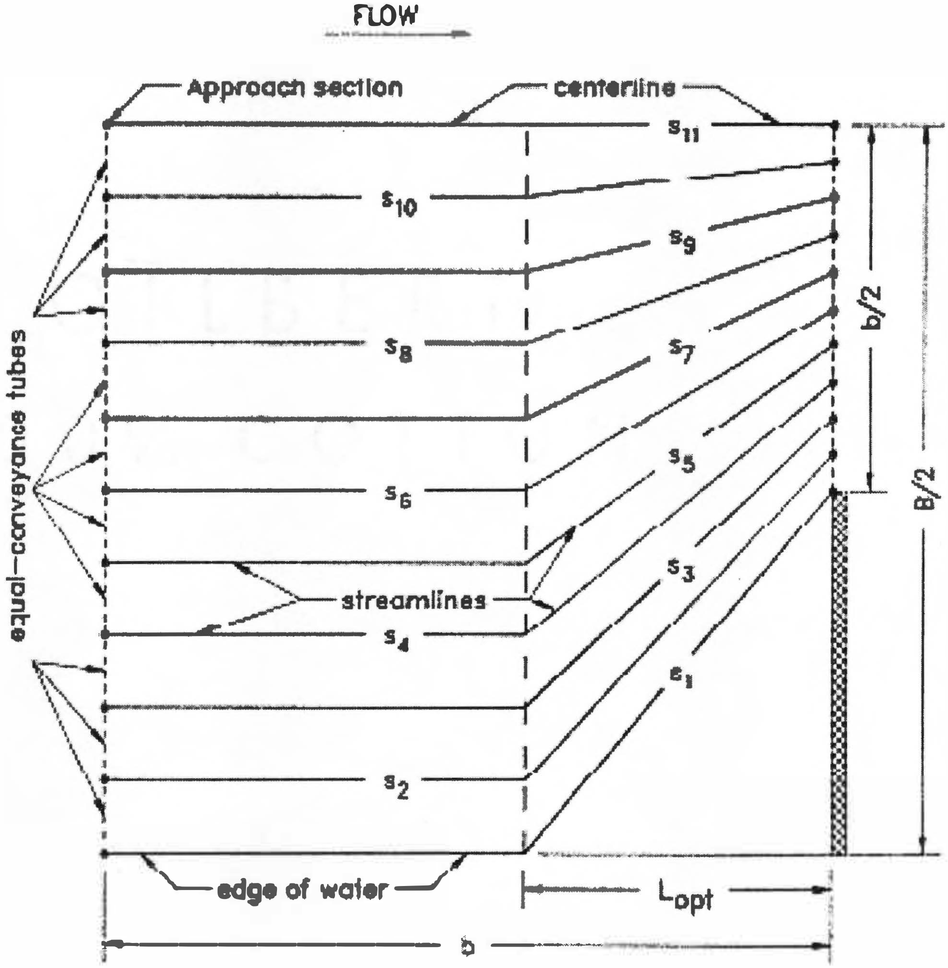


Figure 15: Definition Sketch Of WSPRO Assumed Streamlines (Shearman, 1986)

RAS High Flow Bridge Computations

RAS defines high flow as flow which comes into contact with the maximum low chord of a bridge deck. There are two separate types of flow that may occur: pressure flow, and weir flow. These may occur separately or together, or weir flow may occur along with low flow. Figure 13 illustrates both pressure and weir flow in addition to low flow.

When the water surface comes into contact with the upstream low chord of the bridge pressure flow begins. This contact causes increased backwater and pressure flow is caused by the weight of water above the low chord elevation. Figure 16 shows the depths at which various flow regimes will occur.

Weir flow begins when the water surface rises above the lowest point of the embankment of the approach roadway. The roadway surface acts as a weir conducting water across the embankment and downstream of the bridge. Figure 16 illustrates weir flow over a bridge and roadway embankment.

These types of flow may occur independent of each other, or at the same time. Sites where the approach roadway is lower than the bridge may experience weir flow across the embankment and low flow at the bridge opening. Pressure flow may be experienced alone in situations where the water surface rises above the bridge low girder but not does not overtop the roadway.

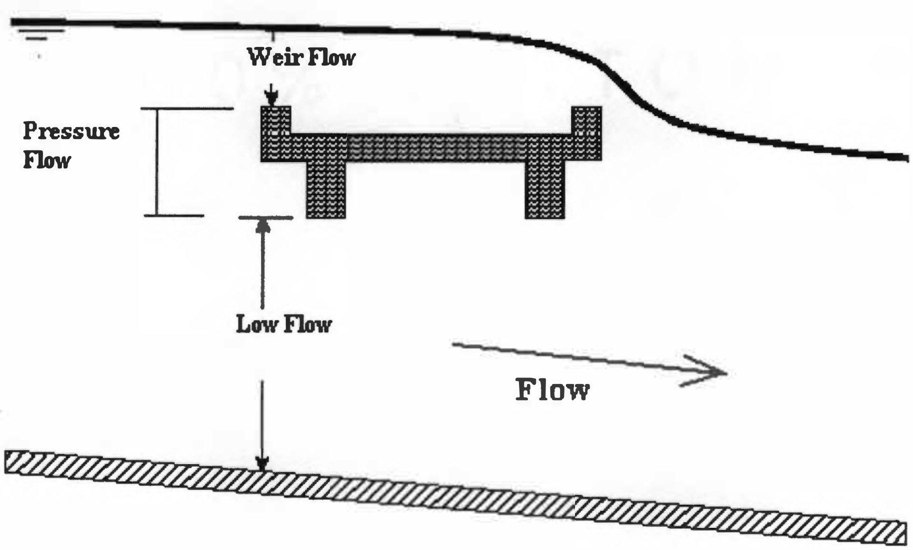


Figure 16: Low Flow, Pressure Flow, and Weir Flow
Through A Bridge Opening (adapted from HEC, 1997)

RAS provides two computation methods for high flow events with pressure and/or weir flow. The energy equation may be solved by the standard step method, or the designer may elect to use separate hydraulic equations with the pressure/weir method.

Energy Method

The energy method for high flow is essentially the same as previously discussed for low flows. Equation 8 is balanced in three steps through the bridge. Energy losses are due to friction, contraction, and expansion. Computations are performed as open channel flow. Hydraulic radius and flow area are altered to reflect the submerged portions of the bridge deck and substructure.

Pressure/Weir Method

The pressure/weir method uses separate hydraulic equations for pressure and weir flow. Both types of calculations are discussed below.

As previously discussed, pressure flow occurs through the bridge opening when the water surface raises above the bridge low chord elevation. If water contacts only the upstream side then the equation for flow through a sluice gate is used (26). If the water surface contacts both the upstream and downstream low chords then it is assumed the bridge opening is flowing full and the equation for flow through an orifice is used (27).

$$Q = C_d A_{BU}\left[2g\left(Y_3 - \frac{Z}{2} + \frac{\alpha_3 V_3^2}{2g}\right)\right]^{1/2} \tag{26}$$

Where: Q	=	Flow through bridge (m^3/sec).
C_d	=	Coefficient of discharge for pressure flow.
A_{BU}	=	Net area of bridge opening at section BU (m^3).
Y_3	=	Hydraulic depth at section 3 (m).
Z	=	Vertical distance from max bridge low chord to mean river bed elevation at section BU (m).

$$Q = CA\sqrt{2gH} \tag{27}$$

Where: C	=	Coefficient of discharge for fully submerged pressure flow.
H	=	Difference between the energy gradient elevation upstream of the bridge and the water surface elevation downstream of the bridge (m).
A	=	Net area of bridge opening (m^2).

Weir flow occurs when the water surface elevation rises above the lowest point on the approach roadway. The user must input a set of coordinates describing the roadway grades. From this RAS determines at what elevation weir flow begins and computes the effective length of the weir. The weir discharge coefficient may be modified to account for the effects of tailwater submergence on the weir. Equation 28 is the standard weir equation as used in RAS. At sites with very high tailwater the weir may become completely submerged causing the weir equation to be inaccurate. At this point RAS automatically switches to the energy method.

$$Q = CLH^{0.5} \tag{28}$$

Where: Q	=	Total flow over weir (m^3/sec).
C	=	Weir coefficient of discharge.
L	=	Length of submerged roadway (m).
H	=	Difference between energy upstream and road crest (m).

Selection of Bridge Analysis Method

In summary, RAS provides the user with the option of using energy equation, momentum balance, Yarnell equation, or WSPRO method for analyzing low flow. High flow, in which the water surface rises above the low girder elevation, may be analyzed by using the energy equation, or the pressure/weir method.

Each of these analysis methods has advantages and disadvantages. HEC provides guidelines and recommendations for selection of the most valid method. These guidelines were discussed in Chapter II. This work evaluates those recommendations in a real world situation. RAS models of eight different bridge sites have been created.

Each bridge was modeled with the analysis methods discussed above in order to evaluate the validity of each method and the guidelines for choosing an appropriate method as provided by HEC.

Chapter IV

Modeling Requirements and Site Selection

In order to accomplish the objectives previously discussed, several bridges were analyzed using RAS. The previous chapter provided a discussion of the theoretical basis of RAS. However, in order to properly understand and apply the results of hydraulic modeling with RAS the user must also understand the data input requirements.

It is also important that the reader understand the process of site development and model development. The field sites used were chosen based upon the available data at each location. Data was required in order to develop the model, and observed water surface elevations and flows were required in order to evaluate the results.

Data Requirements

Data required by RAS for bridge modeling falls into three basic categories: cross-section data, flow data, and structure geometry data. Cross-section data is gathered by field surveys of the reach to be modeled. Flow data consists of flow rates for various recurrence intervals along with data on boundary conditions for each flow rate. Flow data is generally obtained from a study of the watershed hydrology. Figure 17 shows the RAS cross-section data editor with all the required data fields. Structure geometry data is obtained from field measurements and as-built or design plans.

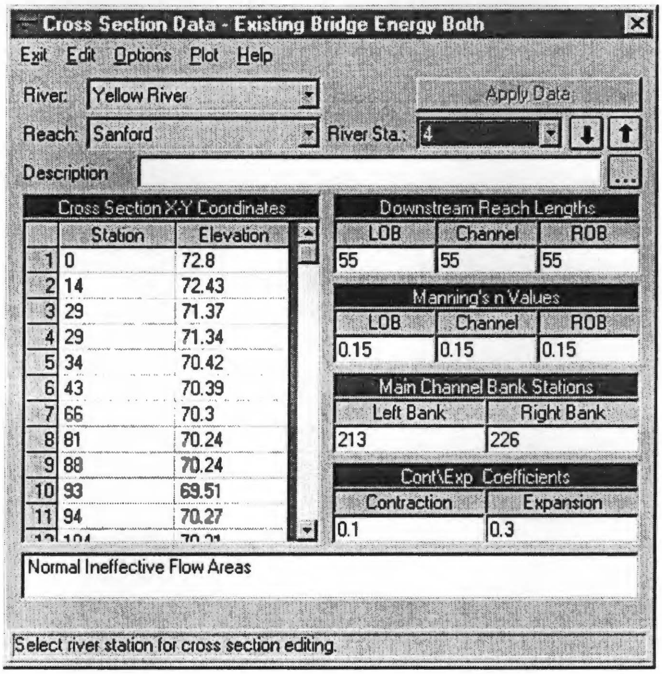


Figure 17: Typical RAS Cross-Section Data Editor

As previously discussed, RAS requires the user to furnish a minimum of four channel cross-sections in order to properly represent a bridge. It is highly advisable to provide other cross-sections outside the influence of the bridge. These sections help to include conditions that influence the water surface elevations at the bridge and ensure accurate results.

Location of the boundary cross-section is extremely important. The term boundary cross-section refers to the section at which computations begin. This is the most downstream cross-section for sub-critical flow and the most upstream cross-section for super-critical flow. This should be located well before flow enters the bridge reach. Boundary conditions are always uncertain. The effect of boundary conditions upon water surface elevation calculations will be examined in greater detail later.

Inclusion of cross-sections upstream of the bridge reach help to determine the effects of bridge backwater. Backwater is an increase in water depth caused by the bridge when compared to pre-bridge conditions. Backwater is a key parameter in the hydraulic design of bridges and aggravates existing floods. It extends well beyond the bridge reach.

Cross-sections should be obtained from a field survey. Cross-section data is entered into RAS as a series of stations and corresponding elevations. The channel section must be defined and the ends of the cross-section should extend beyond the expected floodplain limits. The sections must be aligned normal to the direction of flow. RAS provides methods for interpolating and adjusting cross-sections as required. If a channel reach is relatively constant then only a few cross-sections need be surveyed in the field. These sections can then be copied and located as required. Elevations must be modified based upon the channel slope or a surveyed river profile. This practice introduces very little calculation error while providing significant economic savings.

RAS also provides an automatic routine for interpolating cross-sections. In some cases this method may be utilized as a more economical alternative to a field survey. New cross-sections are automatically interpolated between two existing sections as specified by the user. This can be done between two sections which are radically different, however, care should be taken to insure that the interpolated sections properly represent actual conditions.

RAS also requires that the top of bank stations be known for the main channel in each section. This is done in order to divide the section into three parts: right floodplain, channel, and left floodplain. Computations for conveyance, flow, velocity, and several other hydraulic parameters are carried out for each of the three parts of the section in order to obtain the most accurate results.

Top of bank stations are also used to delineate areas of differing roughness coefficients. The user must enter a minimum of three roughness coefficients. These are normally entered in the form of Manning's n. A roughness value must be provided for each of the three parts of the cross-section as discussed previously. The user may also choose to provide additional roughness values if required. RAS has the optional capability of changing roughness at any point in the cross-section.

Each cross-section must also have loss coefficients for contraction and expansion. The user may provide any value desired ranging from 0.0 for no losses to 1.0 for maximum losses. Default values provided by RAS are 0.1 for contraction losses and 0.3 for expansion losses. These are recommended in RAS documentation for gradual transitions.

See the previous chapter for details concerning the application of the contraction and expansion coefficient.

Ineffective flow areas must also be input in cross-sections within the bridge reach. These may be input as a station and elevation. All area between the input station and the nearest cross-section edge are considered ineffective. This condition is negated if the water surface rises above the user provided elevation. See Figure 11 in the previous chapter for graphical illustration of ineffective flow areas.

The cross-sections must be located with respect to each other as well. The user must give each section a numerical value that RAS uses to arrange the sections in the proper order. The user must also provide the length from each section to the next section immediately downstream. These downstream reach lengths maybe entered separately for the left floodplain, right floodplain, and channel portion of the section. Input lengths are then used during the standard step calculations.

RAS provides graphical views of the river cross-sections and profile. This aids greatly debugging input data prior to running the model. Figure 18 shows a graphical plot of all required cross-section data.

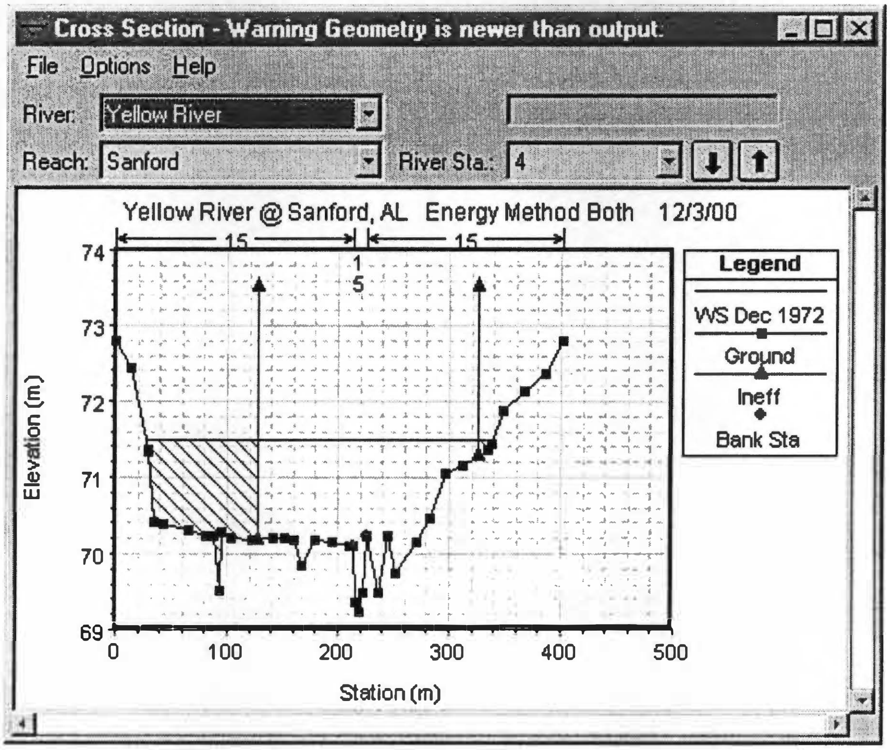


Figure 18: Cross-Section Plot Generated By RAS

RAS also provides several optional capabilities that require additional information. Stream junctions may be modeled if the required data is provided. RAS also models levees if the user provides data indicating the overtopping elevation and its corresponding station on the cross-section. RAS also provides capability to model ice flows, debris jams, and obstructions in the floodplains.

Flow data must also be provided in order to perform a proper bridge model. A study of watershed hydrology provides discharge information and boundary conditions. The user must use his or her own judgement to classify the flow regime as subcritical supercritical or a combination of the two.

Discharge information is generally obtained from peak flow regression equations or some type of hydrologic modeling. Discharge may be input at each cross-section if it flow is spatially varied due to tributary inflow or groundwater seepage, or may be input for the most upstream section if flow is assumed constant through the entire reach. RAS has the capability of computing multiple water surface profiles if required. Multiple discharges may be provided at a single section if the user wishes to simulate several flood events of various recurrence intervals.

In order to begin computations, a boundary condition must be specified. The boundary condition is then used as part of the iterative standard step method as previously discussed. The boundary condition may be in the form of a known water surface elevation, or the user may specify critical or normal depth at the boundary cross-section. If critical or normal depth is specified, RAS automatically computes the water surface elevation based upon the specified depth condition and the user provided stream slope. Normal depth is computed by solving Manning's equation as previously discussed. Critical depth is computed based upon the minimum specific energy as discussed in Chapter III.

Selection of boundary controls is an important consideration in hydraulic modeling of bridges. It is difficult to accurately estimate a boundary water surface elevation for a given discharge without gage data. The various methods RAS provides assist in this process, but care must be taken in selecting the boundary condition, and in locating the boundary cross-section. This is discussed further in a later chapter. Figure 19 shows the RAS boundary condition editor.

The user must also specify which type of flow regime is expected: sub-critical, super-critical, or mixed. This indicates whether RAS should begin computations at the most upstream or downstream cross-section. Specifying a mixed flow regime causes the program to calculate both sub-critical and super-critical portions of the profile.

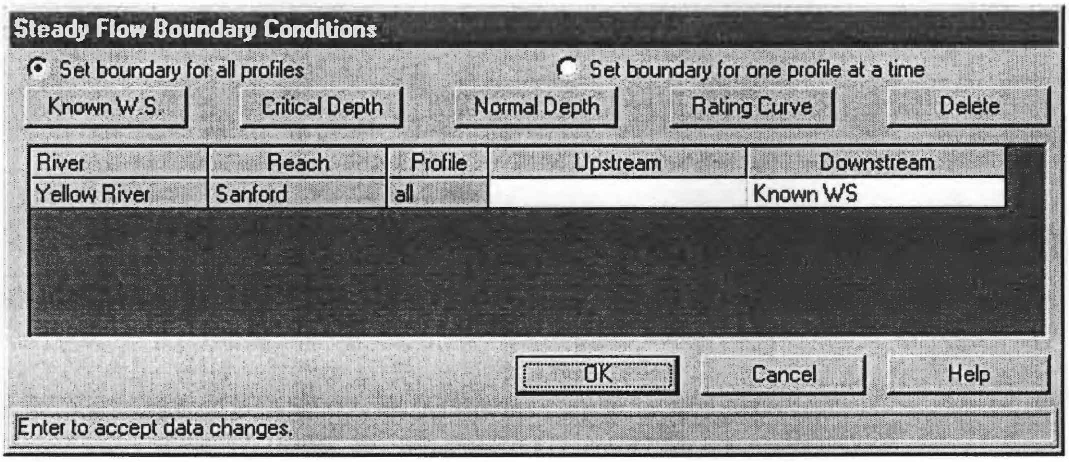


Figure 19: RAS Boundary Condition Editor

Only cross-section and flow data are required for reaches with no hydraulic structures. However, if a bridge or other structure is to be modeled further data is required. Proper modeling of bridges in RAS requires four cross-sections as previously discussed. The user must also provide data to describe the bridge superstructure, substructure, and approach embankment. RAS also has the capability of modeling culverts, levees, or in-stream weirs and spillways.

RAS provides a bridge editor to facilitate data entry (Figure 20). The user must first input bridge width and locate the bridge with respect to other cross-sections. Once the user locates the bridge, RAS chooses the cross-sections that will serve as the BD and BU section and overlays all bridge data on them. The user must then input data to describe encroachment of the approach roadway on the floodplain. As with cross-sections, this is input in the form of stations and corresponding roadway elevations. The stations must be consistent with the stations of the cross-sections near the bridge. RAS then overlays the roadway data onto the cross-section data at sections BD and BU. The user also inputs data to describe the location and thickness of all bridge piers and abutments. Data describing the bridge deck and low chord elevations is also required.

Once all required data is entered, the user may begin calculations. The use of text editors and availability of summary tables and graphical plots makes evaluating the results a simple matter. Changes in the input data are easily made in order to evaluate alternative designs. Standard design procedure requires modeling any existing bridge first, then

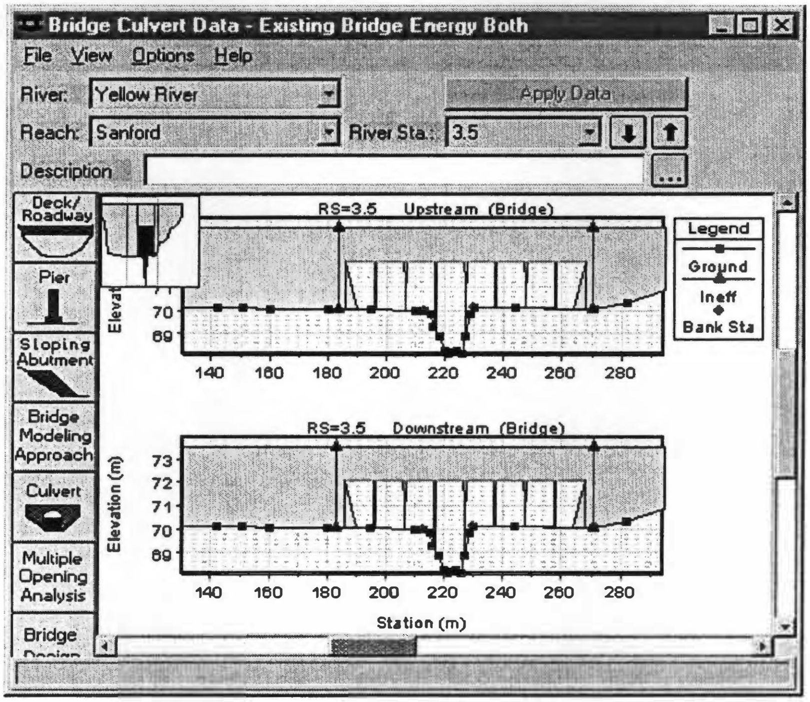


Figure 20: RAS Bridge Data Editor

various alternatives may be modeled so that results may be compared. This helps insure the best possible design.

Study Site Selection

Bridge sites were selected for inclusion in this study based upon available data. Data in addition to the normal requirements of RAS was required. Field measurements for flood flow and peak water surface elevations were required as well. This field data provides a base measurement to which all modeling output can be compared. Individual comparisons between the results of each method and the field collected data help to determine the methods for obtaining the most correct results.

Field data was provided by the USGS (Ming, et. al., 1979). The USGS conducted field investigations at bridges in Alabama, Louisiana, and Mississippi to gather data for the very purpose of computer model verification in the late 1970's. All surveys were conducted using an English unit system of feet and inches. Survey data was converted to metric prior to publication, therefore all results reported here are in metric units. An original goal of including bridges in Tennessee was unattainable due to lack of bridge sites with sufficient information.

In general, all sites were located on wide floodplains with the bridge causing significant contraction of flow. Floodplains were heavily vegetated with timber and other growth requiring large roughness values to be used. Stream slopes were mild in all cases, causing low velocity conditions. Figure 21 shows the typical site topography from a USGS quadrangle map. Note the fairly steep valley sides with a flat floodplain and sinuous channel in the valley bottom. Various sites experienced low flow and pressure flow, however there were no weir flow conditions present.

A field survey was conducted to obtain cross-section data as well as required bridge and embankment data as previously discussed. A minimum of six cross-sections approximately one stream valley width apart were surveyed at each site. An approach

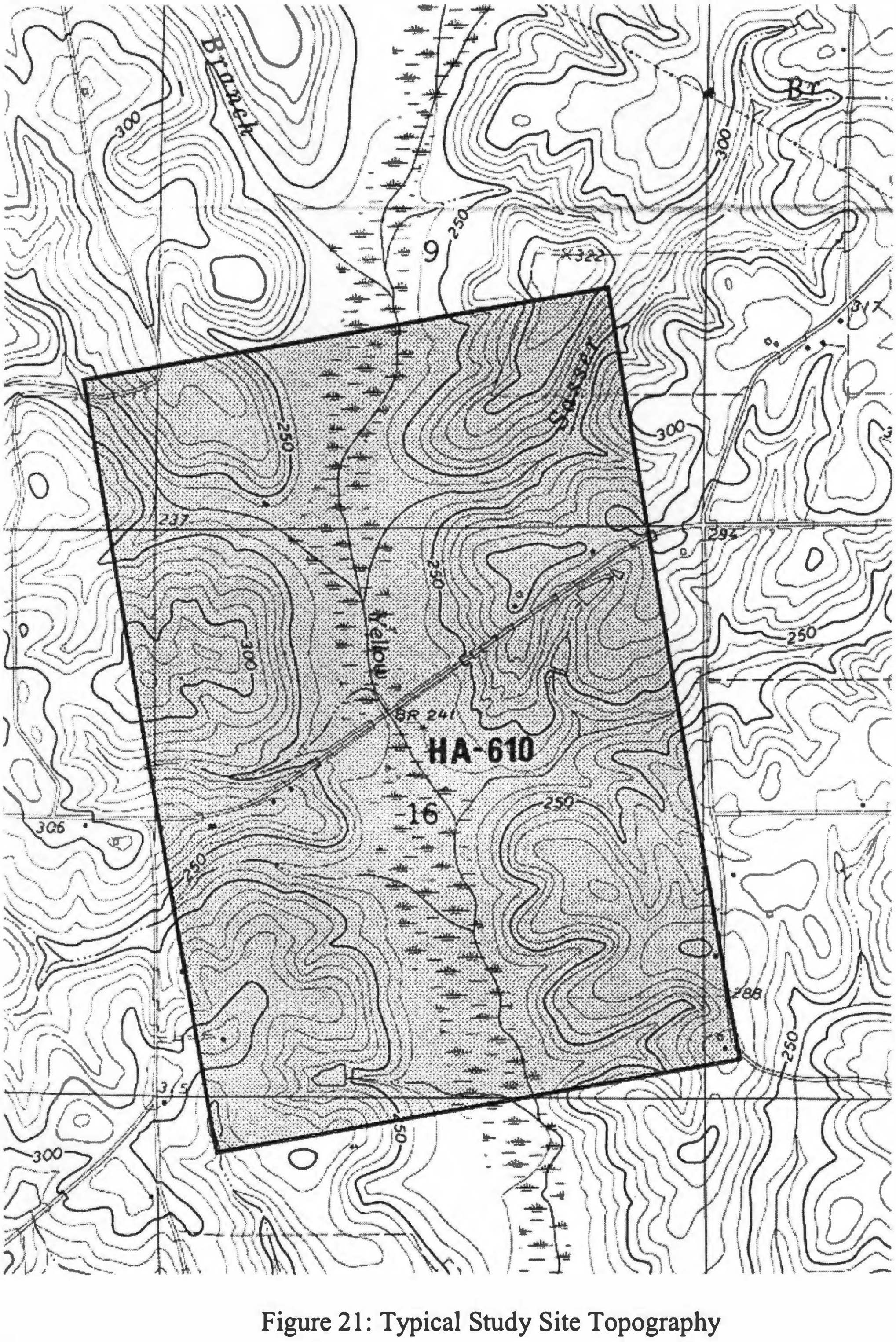


Figure 21: Typical Study Site Topography

cross-section was also surveyed at each site one bridge length upstream of the bridge opening, with additional sections at locations deemed important by field personnel. During model development most sites required additional cross-sections at locations which were not originally surveyed. In those situations the interpolation features of RAS were used to place cross-sections in the proper place. Ground elevations were surveyed to the nearest tenth of a foot and reported to the nearest 0.01 meter. All cross-sections were surveyed along an axis normal to the direction of flow.

Roughness values were reported in the form of composite Manning's n values for left and right overbank and the channel. These were determined based upon the judgement of on-site personnel. The composite values reported vary based upon water depth. The highest values are used at depths below 0.6 meter and roughness decreases linearly up to a maximum depth of one meter.

Flow data was determined based upon on-site measurement with a current meter, or stage-discharge relationships developed by the USGS. Only one flood measurement was available at most sites, however some sites had flow and water surface elevations for multiple flood events. Table 4 shows an overview of bridge and flow data at each location.

Observed water surface elevations were established by high water marks made on-site during the flood event. After recession of the flood these elevations were surveyed at the

Table 4: Description of Bridge Study Locations

Crossing Location	Flood Date	Peak Discharge (m^3/sec)	Rec. Interval (Years)	Bridge Length (m)	Floodplain Width (m)
Alexander Creek near St. Francisville, LA	9/17/71	156	3	73	260
	12/7/71	269	8	73	280
Bogue Chitto near Johnston Station, MS	12/7/71	708	>100	130	1480
	3/25/73	892	>100	130	1580
Cypress Creek near Downsville, LA	2/24/74	42	4	40	265
Flagon Bayou near Libuse, LA	12/7/71	134	4	61	425
Pea Creek near Louisville, AL	12/21/72	50	2	77	300
Poley Creek near Sanford, AL	12/21/72	54	2	62	335
	3/12/73	130	11	62	360
Temmile Creek near Elizabeth, LA	12/7/71	181	4	165	690
Yellow River near Sanford, AL	12/21/72	57	2	82	340
	3/12/73	187	30	82	390

upstream and downstream embankment and at various points along the study reach. Measured elevations at the most downstream cross-section were used for the required boundary elevation when developing the RAS models. The additional observed water surface measurements were used for a base in evaluating output from the various RAS model runs.

Chapter V

Results

As shown in Table 4 of the previous chapter, eight bridges with a total of twelve separate flood events were analyzed using RAS. After all data was input, several important parameters were varied in order to determine an optimum selection for each parameter.

A water surface elevation profile was computed for each flood event presented in Table 4. A profile simply consists of a computed water surface elevation at each cross-section. Computed elevations are then connected by a straight line as depicted in Figure 22. A profile was computed for each flood event using each method or technique discussed below.

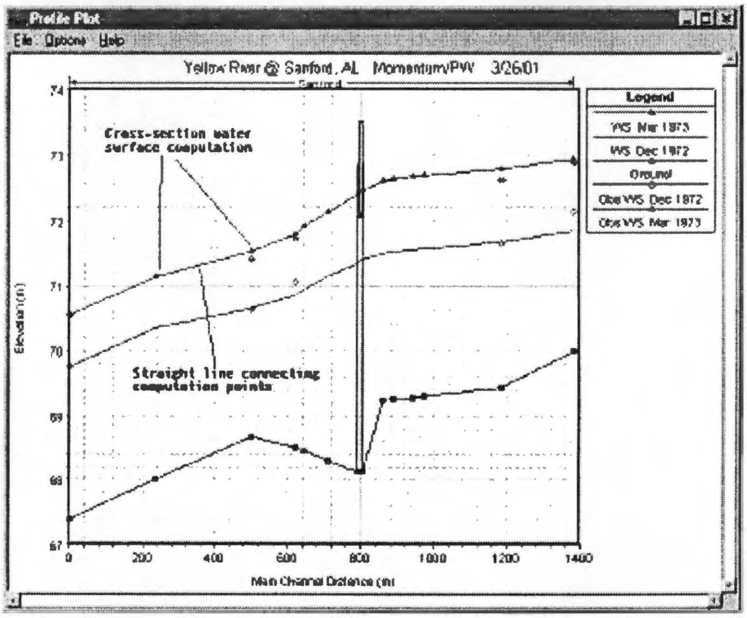


Figure 22: Typical Water Surface Profile

Throughout this document the term error is used. This refers to the difference between computed and observed water surface elevations (WSE). The average error is also referenced many times. In order to determine average error the error of the computed WSE is first determined for several cross-sections. The absolute value of each error is then calculated. The average error is then calculated for all sampled cross-sections. The absolute value operation is applied so that positive and negative errors do not cancel each other. The reported average water surface errors represent all computations at the reference sections for a particular technique. The number and location of reference sections varied depending upon the technique used, and is discussed below.

$$Error = CWSE - OWSE \tag{29}$$

Where: CWSE = Calculated Water Surface Elevation (m).
OWSE = Observed Water Surface Elevation (m).

$$AverageError = \frac{|E_1| + |E_2| + \ldots + |E_i|}{i} \tag{30}$$

Where: E = Error at cross-section (m).
i = Number of cross-sections (varies per each analysis).

As an example, the average error reported for RAS recommendations of transition length is 0.12 meter. In order to compute this average error, water surface profiles were computed for each of the twelve flood events. Errors were then computed for five cross-sections from each profile as shown in Equation 29. Then the absolute value of each error was taken and the average was computed as shown in Equation 30.

Transition Lengths

The first parameters to be analyzed were the contraction and expansion lengths. RAS, HEC-2, and WSPRO all give separate recommendations for these values as previously discussed. Each bridge flood event was modeled using contraction and expansion lengths determined by each recommendation. Contraction and expansion coefficients were 0.1 and 0.3 respectively outside of the bridge reach while 0.3 and 0.5 were used within the bridge transition zones. Ineffective flow areas were placed as previously shown in Figure 11. A base condition with no transition reaches was also modeled. The no transition condition was developed without ineffective flow areas and with contraction and expansion coefficients of 0.1 and 0.3 at all cross-sections. Table 5 and Figure 23 show the contraction and expansion lengths used for the various methods.

Table 5: Expansion and Contraction Lengths

Site	Contraction Length			Expansion Length		
	HEC-2	RAS	WSPRO	HEC-2	RAS	WSPRO
Alex-ander	79	75	71	257	91	71.5
Bogue	607	306	128	1930	786	128
Cypress	175	136	40	700	251	40
Flagon	196.5	112	61	847	146	62
Pea	102.5	140	77	409.5	79.5	76.5
Poley	139	139	61	536	135	61.5
Tenmile	364	197	165	1456	351	165
Yellow	139	169	83	533	147	82

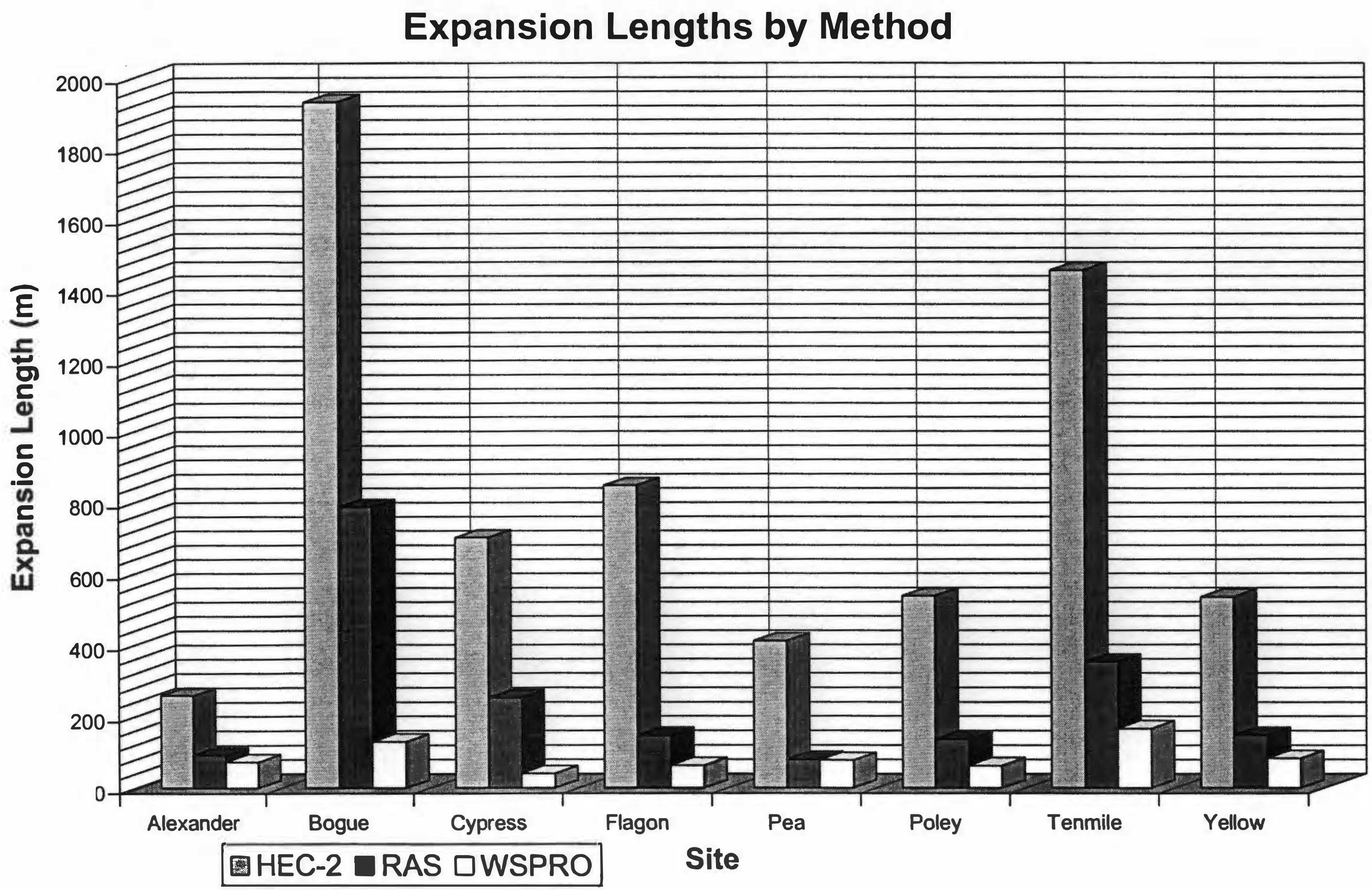


Figure 23: Expansion Lengths

Each low flow event was analyzed using the energy method RAS provides, and high flow events were analyzed with the pressure and weir method. Observed water surface elevations were used to provide downstream boundary values

Errors were computed at each site for each of the methods used at five different cross-sections within the profile. These are as follows:

(1) Downstream of the bridge, within the expansion length.

(2) Just outside the downstream bridge face.

(3) Just inside the upstream bridge face.

(4) Upstream of the bridge, within the contraction length.

(5) Upstream of the bridge, upstream of the contraction.

As previously discussed, the study included twelve separate flow events. Errors were computed at five cross-sections within each event. This means that sixty data points were available for each method. These data points were used to compare the various methods discussed above.

Observed water surface elevations at the upstream and downstream bridge face were somewhat variable. The observed elevation used for computing errors is an average of several observed elevations along the faces of the bridge and approach embankments.

These elevations exhibited some variability depending upon proximity to the main channel.

Tables 6 and 7 present a summary of errors at these data points. Table 6 shows values which represent errors at an individual cross-section averaged over all twelve flood events. Table 7 presents the range of these same errors. Based upon these tables, each method appears to yield similar results. However several things should be pointed out.

HEC-2 consistently gave higher errors and a much larger range of errors with a deviation as high as 1.28 meters. This is due to the fact that HEC-2 recommendations lead to expansion lengths approximately four times longer than the other methods. A longer expansion reach results in an over-calculation of expansion losses. This means that calculated water surface elevations are too high when using this method.

Magnitude of the errors is important when comparing the errors, however it is also important to note whether the error is positive or negative. Positive errors mean the computed water surface elevation is higher than observed, while negative errors indicate that the computed water surface elevation is lower than observed. In order to be conservative, errors should be positive rather than negative. Table 8 shows the number of negative errors out of sixty calculations for each method.

Table 6: Average Errors of Water Surface Elevation Computations At Cross-Sections

Cross-section Location	Average Error (m) by Method			
	No Transitions	WSPRO Transitions	RAS Transitions	HEC-2 Transitions
Upstream of contraction	0.11	0.10	0.10	0.12
Inside contraction reach	0.12	0.11	0.11	0.16
Upstream bridge face	0.12	0.13	0.11	0.15
Downstream bridge face	0.12	0.12	0.14	0.21
Inside expansion reach	0.10	0.10	0.11	0.18
Overall Average Error for Each Method	**0.11**	**0.11**	**0.12**	**0.13**

Table 7: Range of Water Surface Elevation Errors

Cross-section Location	No Transition	WSPRO Transitions	RAS Transitions	HEC-2 Transitions
Upstream of contraction	-0.33 - 0.02	-0.39 - 0.10	-0.36 - 0.16	-0.33 - 0.47
Inside contraction reach	-0.38 - 0.07	-0.33 - 0.10	-0.27 - 0.37	-0.25 - 0.66
Upstream bridge face	-0.37 - 0.11	-0.42 - 0.09	-0.32 - 0.10	-0.23 - 0.37
Downstream bridge face	-0.32 - 0.37	-0.33 - 0.44	-0.26 - 0.48	-0.24 - 1.28
Inside expansion reach	-0.28 - 0.30	-0.28 - 0.30	-0.28 - 0.31	-0.28 - 1.12

Table 8: Negative Water Surface Elevation Errors for Various Transition Length Recommendations

Transition Calculation Method	No. of Negative Calculation Errors	% of Negative Calculation Errors
No Transition	35	58
WSPRO	27	45
RAS	24	40
HEC-2	11	18

As Table 8 indicates, water surface elevation computations performed with no transition reaches were below observed elevations at nearly 60% of cross-sections examined. This is to be expected since this method does not account for expansion and contraction losses. Modeling a bridge without transition reaches will lead to under-design of the bridge and subsequent flooding upstream. This method is unsatisfactory, and some sort of transition reach is required.

The disparity of HEC-2 calculations as previously discussed is obvious when examining Tables 6, 7, & 8. Very few of the calculations made using HEC-2 recommendations were below observed elevations. It is satisfactory to have calculations slightly higher than observed values however, HEC-2 often gives errors that are significantly higher than observed elevations. HEC-2 recommendations are overly conservative and design using these methods would lead to an overly large bridge.

After discarding the idea of using no transition reaches and the HEC-2 recommendations for transitions, the RAS and WSPRO methods are left. Each of these methods appears to give similar results. However, RAS gives slightly fewer negative errors. The RAS recommendations appear to be particularly well suited for computations downstream of the bridge as downstream accuracy was best using transition lengths recommended by RAS.

RAS appears to be the most theoretically sound as it is based upon obstruction length while WSPRO is based upon bridge length. Obstruction length is a direct measurement of the width of contraction and expansion required and, as such, is directly related to the length required for flow to perform this transition. Obstruction length is also dependent upon both bridge length and floodplain width. Bridge length has no definite relationship with floodplain width, and thus, is not directly related to expansion and contraction width. This is supported by the previously discussed work of Hunt and Brunner (1995), which used two-dimensional models to develop the RAS recommendations.

Based upon the theoretical soundness of RAS when compared to WSPRO, as well as the slightly lower errors, the regression equations developed by Hunt and Brunner (1995) and provided as part of the RAS documentation (HEC, 1997) appear to be the most valid method for determining transition lengths.

Low Flow Bridge Analysis Methods

As previously discussed, RAS defines a low flow situation as one in which flow does not contact the low point on the deck of the bridge. RAS includes four methods for computing water surface elevations at bridges during low flow: energy, momentum, Yarnell, and WSPRO. As previously discussed HEC provides some guidelines in RAS documentation concerning which method is best in various situations.

Nine of the twelve flood events used in this study were low flow events. These events are shown in Table 9. Water surface profiles were developed for each of these low flow events using each of the calculation methods provided by RAS. Transition reaches were determined according to guidelines provided by RAS as previously discussed. Observed water surface elevations were used for boundary values.

Pier substructures at all bridges were multiple pile bents. Figure 24 shows a multiple pile bent substructure typical of the field sites used in this thesis. A drag coefficient (C_D) of 2.00 was used for this type of substructure during calculations using the momentum

Table 9: Bridge Locations and Flow Data for Low Flow Events

Crossing Location	**Flood Date**	**Peak Discharge (m^3/sec)**	**Rec. Interval (Years)**	**Bridge Length (m)**	**Flood-plain Width (m)**
Alexander Creek near St. Francisville, LA	9/17/71	156	3	73	260
	12/7/71	269	8	73	280
Cypress Creek near Downsville, LA	2/24/74	42	4	40	265
Flagon Bayou near Libuse, LA	12/7/71	134	4	61	425
Pea Creek near Louisville, AL	12/21/72	50	2	77	300
Poley Creek near Sanford, AL	12/21/72	54	2	62	335
	3/12/73	130	11	62	360
Temmile Creek near Elizabeth, LA	12/7/71	181	4	165	690

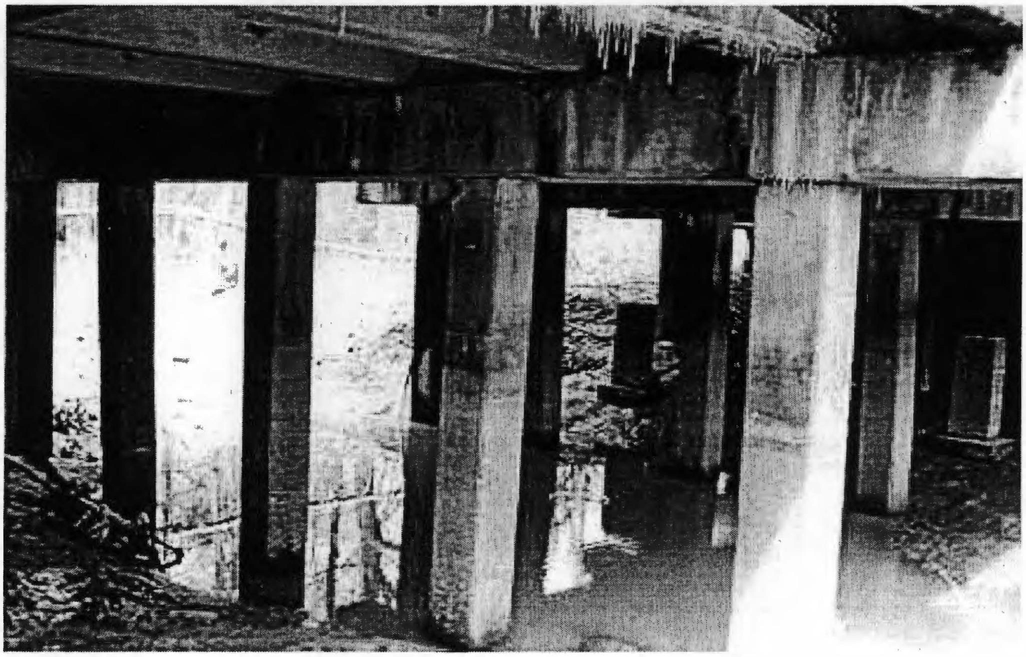


Figure 24: Typical Multiple Pile Bent Installation.

method. RAS does not recommend a C_D for this type of substructure, therefore the value recommended for square nose piers was used. The Yarnell pier coefficient (K) for a ten pile trestle bent (K=2.50) was used for analysis by Yarnell's method.

The procedure for comparing low flow analysis methods was similar to the procedure discussed above for comparison of transition lengths. Each method was used to compute energy losses and water surface elevations through the bridge, and errors were computed at various cross-sections. In the previous discussion five cross-sections were used for comparison, for this analysis only three were used. No comparisons were made at cross-sections located downstream of the bridge. This was because all bridge computation methods discussed here provide the same results downstream of the bridge. The cross-sections compared for this analysis were located upstream of the bridge approach section (1), within the contraction reach (2), and at the upstream bridge face (3).

Errors were computed at each of the three cross-sections for each of the nine low flow events. This provided twenty-seven data points for each computation method. These data points were then used to determine the validity of each method. Table 10 presents the average error of computed water surface elevations while Table 11 presents the range of computed errors, and Table 12 illustrates the number of negative computed errors.

Table 10: Average Water Surface Elevation Errors for Low Flow Computation Methods

Cross-section Location	**Average Error (m) by Method**			
	Energy Method	**Momentum Method**	**Yarnell Method**	**WSPRO Method**
Upstream of contraction	.11	.13	.11	.11
Inside contraction reach	.13	.13	.13	.13
Upstream bridge face	.13	.12	.13	.13
Overall Average Error for Each Method	**0.13**	**0.13**	**0.13**	**0.13**

Table 11: Range of Water Surface Elevation Errors for Low Flow Computation Methods

Cross-section Location	**Energy Method**	**Momentum Method**	**Yarnell Method**	**WSPRO Method**
Upstream of contraction	-0.39 - 0.11	-0.39 - 0.16	-0.39 - 0.11	-0.39 - 0.11
Inside contraction reach	-0.33 - 0.13	-0.22 - .29	-0.33 - 0.13	-0.33 - 0.13
Upstream bridge face	-0.33 - 0.10	-0.23 - 0.23	-0.33 - 0.10	-0.33 - 0.10

Table 12: Negative Water Surface Elevation Errors for Low Flow Computation Methods

Bridge Calculation Method	No. of Negative Calculation Errors	% of Negative Calculation Errors
Energy	15	56
Momentum	8	30
Yarnell	15	56
WSPRO	15	56

Surprisingly, the energy, Yarnell, and WSPRO methods produced the same results. Computed water surface elevations were the same for each of these three methods at the cross-sections examined. The momentum method was the only method to produce significantly different results and this method was different only for certain flood events. The most significant differences between the momentum method calculations and calculations by other methods were for flood events at Alexander Creek, Flagon Bayou, and Tenmile Creek. In an effort to explain these differences, several factors were examined. These are presented below in Table 13.

The differences in computed water surface elevations appears to be primarily due to pier losses. The energy and WSPRO methods compute water surface elevations by an energy based approach. In this method piers simply reduce available area for flow and add wetted perimeter. The Yarnell method does account for piers to some extent, but ignores area of the bridge opening, and the bridge itself.

Table 13: Summary of Low Flow Events

Flood Event Location	Discharge (m^3/sec)	% of floodplain obstructed by embankment	% of bridge opening obstructed by piers
Alexander 1	156	72	2.5
Alexander 2	269	74	3
Cypress	42	85	3
Flagon	134	86	6.8
Pea	50	74	7.1
Poley 1	54	83	3.5
Poley 2	130	83	3.5
Tenmile	180	76	5.8
Yellow 1	57	76	3.7

The momentum method incorporates a pier drag coefficient (C_D) as shown in Equation 31. This concept was introduced in Chapter III, but when applied to bridge analysis Equation 31 has a term added to account for pier loss. Pier losses are represented by the last term on the right side of the equation. Energy loss due to pier drag is a function of C_D, area obstructed by pier (A_P), area of the bridge opening (A_3) and discharge. Essentially, pier loss is a function of velocity through the bridge, and area obstructed by piers.

$$A_3\bar{Y}_3 + \frac{\beta_3 Q_3^2}{gA_3} = A_{BU}\bar{Y}_{BU} - A_{PBU}\bar{Y}_{PBD} + \frac{\beta_{BU} Q_{BU}^2}{gA_{BU}} + F_f - W_X + \frac{1}{2}C_D\frac{A_{PBU}Q_3^2}{gA_3^2} \tag{31}$$

Where: A_3, A_{BU} = Effective flow area at respective sections (m^2).
Q_3, Q_{BU} = Discharge at respective sections (m^3/sec).
A_{PBU} = Area obstructed by bridge pier on upstream side (m^2).
Y_3, Y_{BU} = Vertical distance from water surface to center of gravity of flow area A_2 and A_{BD} (m).

Y_{PD} = Vertical distance from water surface to center of gravity of wetted perimeter area on downstream side (m).

β_3, β_{BU} = Velocity weighing coefficients.

Q_2, Q_{BD} = Discharge at respective sections.

g = Gravitational acceleration constant (9.81 m/sec^2).

F_f = External friction force, per unit weight of water (kg per kg/m^3).

W_x = Force due to weight of water in direction of flow, per unit weight of water (kg per kg/m^3).

C_D = Drag coefficient for flow at piers.

The Alexander 1, Alexander 2, Tenmile, and Flagon flood events had significant differences between elevations computed with the momentum method and elevations computed with the three other methods. The contributing factor appears to be velocity of flow through the bridge. The four events mentioned had the highest flow rates. This, in combination with small bridge opening, resulted in high velocities through these bridges. As shown in Equation 31, the momentum equation computes pier losses as a function of flow and area. Since velocity is also a function of flow and area, then pier loss is a function of velocity.

RAS documentation (HEC, 1997) makes the following recommendation for selection of low flow computation methods:

- "In cases where the bridge piers are a small obstruction to the flow, and friction losses are the predominate consideration, the energy based method, the momentum method, and the WSPRO method should give the best results."

This recommendation holds true for six of the low flow events analyzed. The majority of energy losses for these events was due to friction, contraction, and expansion. Each of the computation methods computes these in a similar manner therefore the resulting water surface elevations are the same, or very similar.

RAS documentation (HEC, 1997) also makes this recommendation:

- "In cases where pier losses and friction losses are both predominant, the momentum method should be the most applicable, but any of the methods can be used."

This recommendation applies to the Alexander 1, Alexander 2, Flagon, and Tenmile flood events. Friction losses were significant for these events, however pier losses were also large.

RAS does not provide guidance for determining whether pier losses are significant. Based upon this analysis pier losses are significant in situations where flow is relatively large, and bridge opening area is relatively small. This essentially means that pier losses become more significant as velocity through the bridge increases.

High Flow Bridge Analysis Methods

RAS considers high flow to be any situation where flow has risen above the maximum low chord. Two methods are provided for computations, the energy method and the pressure/weir method. The energy method performs an energy balance through the bridge reach, while the pressure/weir method makes computations using separate hydraulic formulae for pressure and weir flow through the bridge opening. Chapter III provides further discussion of these methods.

One of the twelve flood events included in this study was determined to be a high flow event. A water surface profile was developed for this event as previously discussed for low flow events. This event was analyzed using both the energy and the pressure/weir method. The water surface rose above the maximum low chord causing pressure flow, but did not flow over the roadway. Errors were computed at three cross-section locations as previously discussed for low flow events. Table 14 presents these errors. The errors presented below are based upon only one flood event.

Table 14: Average Water Surface Elevation Errors For High Flow Computation Methods

Cross-section Location	**Water Surface Elevation Error (m) by Method**	
	Energy Method	**Pressure/Weir Method**
Upstream of contraction	0.41	0.13
Inside contraction reach	0.48	0.18
Upstream bridge face	0.46	0.12
Overall Average Error for Each Method	**0.45**	**0.14**

As Table 14 shows, selection of a high flow computation method can make a significant difference. Both methods over-predict the water surface profile, however the energy method gives errors three times those computed for the pressure/weir method. Indicating that the pressure/weir method is the best choice for pressure flow situations and the energy method is extremely inaccurate.

RAS documentation (HEC, 1997) recommends that the pressure/weir method be used in high flow situations where the bridge is acting as a pressurized orifice, and if the bridge is not acting as a pressurized orifice, the energy method should be used. The user must decide if the bridge is acting in this manner.

Pressure flow occurs when water has risen above the maximum low chord elevation. As water continues to rise above the max low chord elevation a large pressure head develops due to the increasing depth of water upstream of the bridge when compared to depth downstream. When water rises to the point where it flows over the roadway this pressure head is somewhat relieved. As flow over the roadway increases tailwater depths increase. In extreme situations headwater and tailwater depths begin to equalize and pressure is relieved. Therefore in situations with high tailwater, the energy method is recommended.

The one high flow event discussed here experienced only pressure flow. In this situation the energy method does not appear to give a valid solution. Therefore, the pressure/weir method is recommended for situations with pressure flow only. In situations where weir flow is experienced in addition to pressure flow this analysis may not be valid.

Perched Bridge Analysis

A perched bridge is a more complex situation than previously discussed. A perched bridge is one in which the bridge is significantly higher than the approach roadways. Flood events flow over the roadway before rising to the bridge low chord elevation. In this situation low flow is experienced in combination with weir flow. Figure 25 (HEC, 1995) illustrates the situation of a perched bridge.

Two flood events at Bogue Chitto experienced this situation. These events were analyzed as previously discussed for low flow events. Field personnel reported flow across the approach roadway approximately 0.1 meter deep during both flood events.

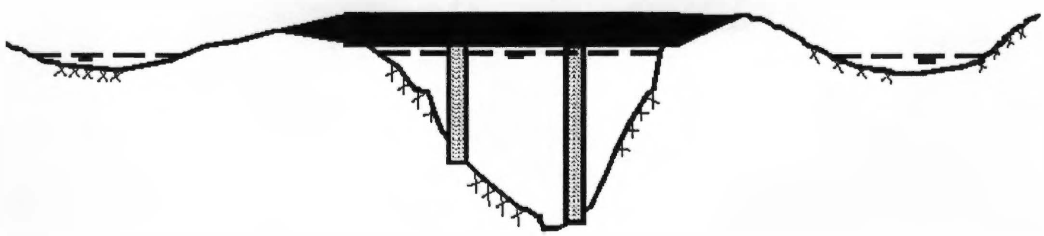


Figure 25: Typical Perched Bridge Section

The momentum and WSPRO methods become invalid in a situation with weir or pressure flow, and RAS automatically disregards them. RAS performs an iterative procedure to balance energy of the low flow method and the weir flow to determine the portion of flow for each.

RAS recommends the energy method for use with perched bridges. This appears to be warranted based upon the two flood events analyzed herein. The energy method gives an average error of 0.03 meter while the Yarnell method gives an average error of 0.14 meter.

Effects of Cross-Section Interpolation

As discussed in Chapter II, RAS monitors changes in the hydraulic parameters between sections. Large changes in conveyance, velocity head, and flow depth cause RAS to give a warning to the user. This provision is part of RAS in order to insure that the assumption of gradually varied flow is valid. If RAS computes large changes in the hydraulic parameters discussed above, additional cross-sections are recommended.

For those situations when additional cross-sections are required the engineer has two options: (1) gather additional field data, or (2) interpolate additional sections with the interpolation feature built into RAS. The engineer must make a decision based upon cost factors, and the anticipated influence of additional sections. If the river reach to be analyzed is fairly consistent and unchanging, sections may be interpolated with some confidence. However, does this affect the results?

In order to determine the effects of cross-section interpolation, three of the previously discussed low flow events and one high flow event were re-analyzed with interpolated cross-sections at various spacings. The momentum method was used for low flow analysis and the pressure/weir method was used for high flow analysis. Each flood event was analyzed with interpolated cross-sections at intervals of twenty meters, fifty meters, and one hundred meters. The results were then compared to the results of an analysis with no interpolated cross-sections.

The Yellow River flood of March, 1973 was analyzed to determine the effects of cross-section interpolation on a high flow event. Fourteen surveyed cross-sections at irregular intervals were used. When cross-sections were interpolated at fifty meter intervals sixty-five cross-sections were used. When the resulting water surface profiles were compared, the maximum difference between water surface elevations was 0.05 meter. Interpolation of cross-sections did not appear to significantly affect this flood event.

The results were similar for the four low flow events analyzed. Interpolation of cross-sections did not affect the water surface elevations at the original cross-sections. However, differences were observed within the reaches between the original cross-sections.

When constructing water surface profiles, RAS simply draws the water surface as a straight line between cross-sections. Addition of interpolated sections revealed slight dips and rises in the water surface elevations not previously apparent. Figure 26 illustrates this phenomenon. The importance of this phenomenon is dependent upon the spacing of surveyed cross-sections which have been provided. If numerous, closely spaced surveyed sections are available the interpolation feature would be required less. The increased detail obtained from interpolated cross-sections appears to make the most difference within the bridge reach and near other features which may cause rapidly varied flow.

The results of the profile with a twenty meter interpolation interval were nearly identical to the results of the one hundred meter interpolation interval. Drastic reduction of spacing appeared to have no affect upon the results. There appears to be a point of diminishing returns at a spacing of one hundred meters.

The slight changes revealed with added sections did not affect elevations previously calculated without interpolated sections. Due to this fact, addition of sections does not appear to be necessary except in areas of particular interest, or areas that experience rapidly varied flow.

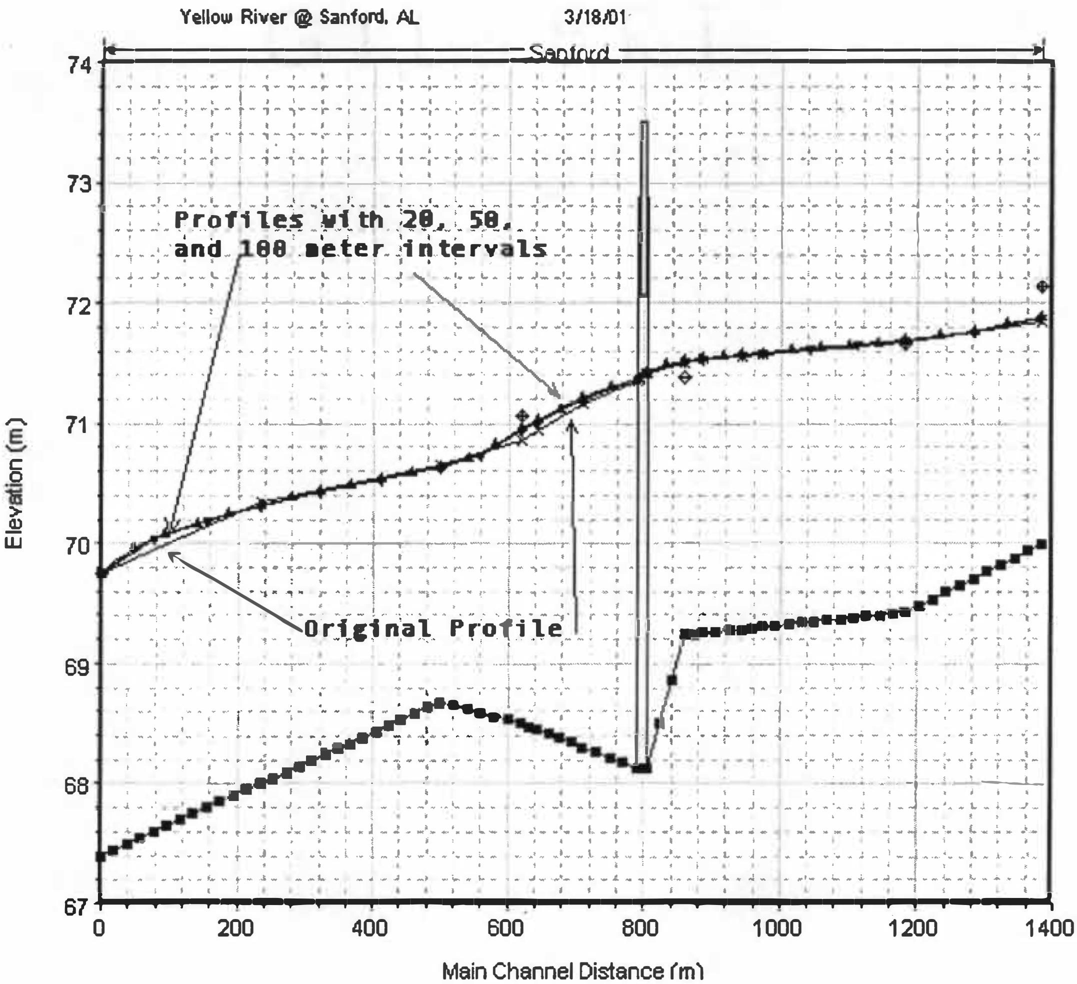


Figure 26: Changes in water surface profiles due to interpolated sections.

Based upon these observations, it appears that use of interpolated cross-sections should be based upon three factors shown below. Interpolated sections do not appear to be required in most situations, except where rapidly varied flow is present. The engineer should always ensure interpolated sections accurately represent actual channel conditions.

(1) Availability and spacing of field surveyed cross-sections.

(2) Locations where specific data is required on water surface elevations (i.e. structures on or near floodplain).

(3) Location of channel features such as bridges or drop-offs which cause rapidly varied flow.

Influence of Boundary Conditions

As previously discussed, all hydraulic computations require a beginning boundary value. This value is then used to begin progression of the standard step method along the river reach. Calculations begin at the downstream most cross-section and proceed upstream for subcritical flow and begin at the upstream most cross-section and proceed downstream for supercritical flow.

Three of the previously developed flood events were analyzed to determine the effects of boundary values upon computational accuracy. Each event was analyzed using normal depth, critical depth, and observed water surface elevation for boundary values as discussed in Chapter III. Figure 27 shows the resulting profiles for a typical flood event.

The iterative nature of the standard step method causes profiles computed with each boundary value to converge as computations move upstream. Movement of computations along the profile slowly eliminates errors as each new cross-section computation is closer to the actual. At some point profiles computed with various boundary values converge and computations are no longer dependent upon boundary conditions. This is illustrated in Figure 27.

This does not mean that care is not required in choosing of the boundary value. Error is gradually reduced, but only over several calculations. Specifying a grossly inaccurate boundary value may cause calculations through the entire reach to be inaccurate. As Figure 28 shows below, a large inaccuracy at the boundary may require many require many step computations before it is eliminated. In Figure 28 the profile computed with an observed water surface elevation as boundary condition is presumed to be the most accurate. The profile computations made with a normal depth boundary do not converge with the more accurate computations until the profile has moved a large distance upstream. This means that all cross-sections downstream of the convergence are inaccurate, and better accuracy could be obtained through more accurate boundary conditions.

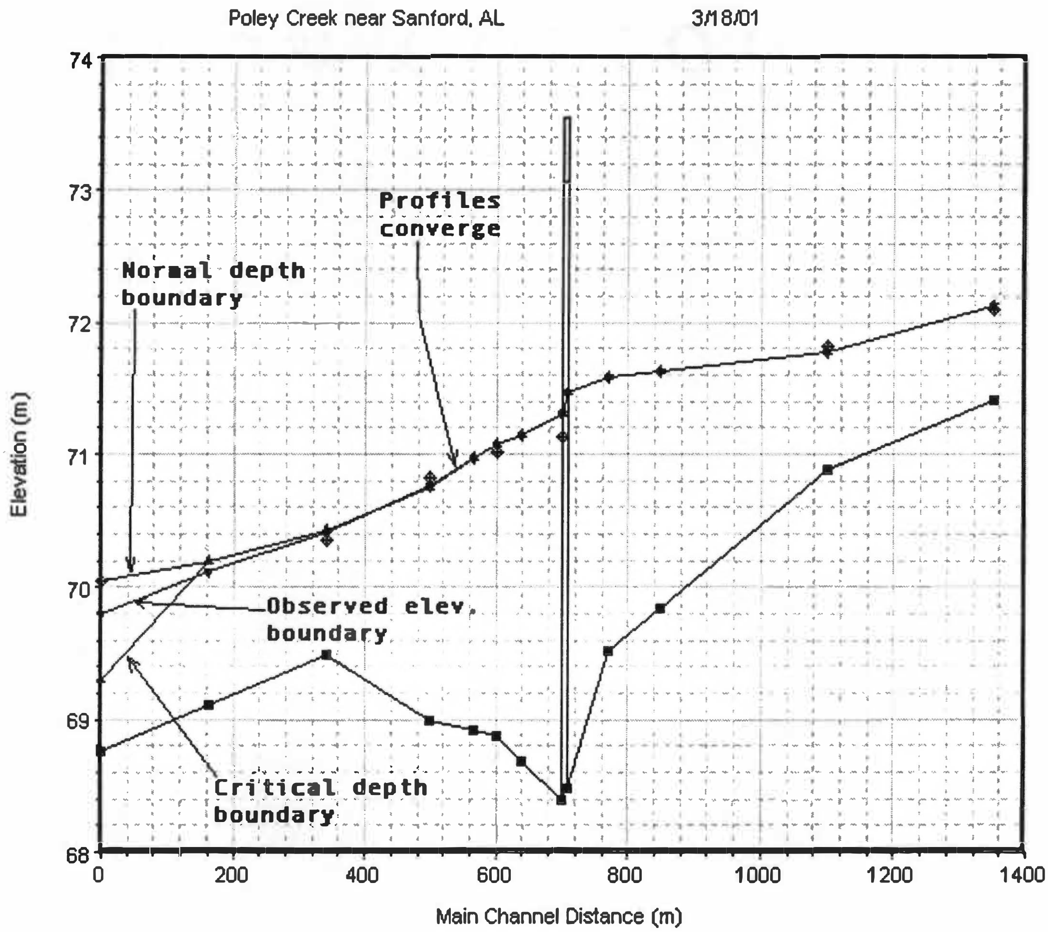


Figure 27: Water surface profiles for various boundary conditions.

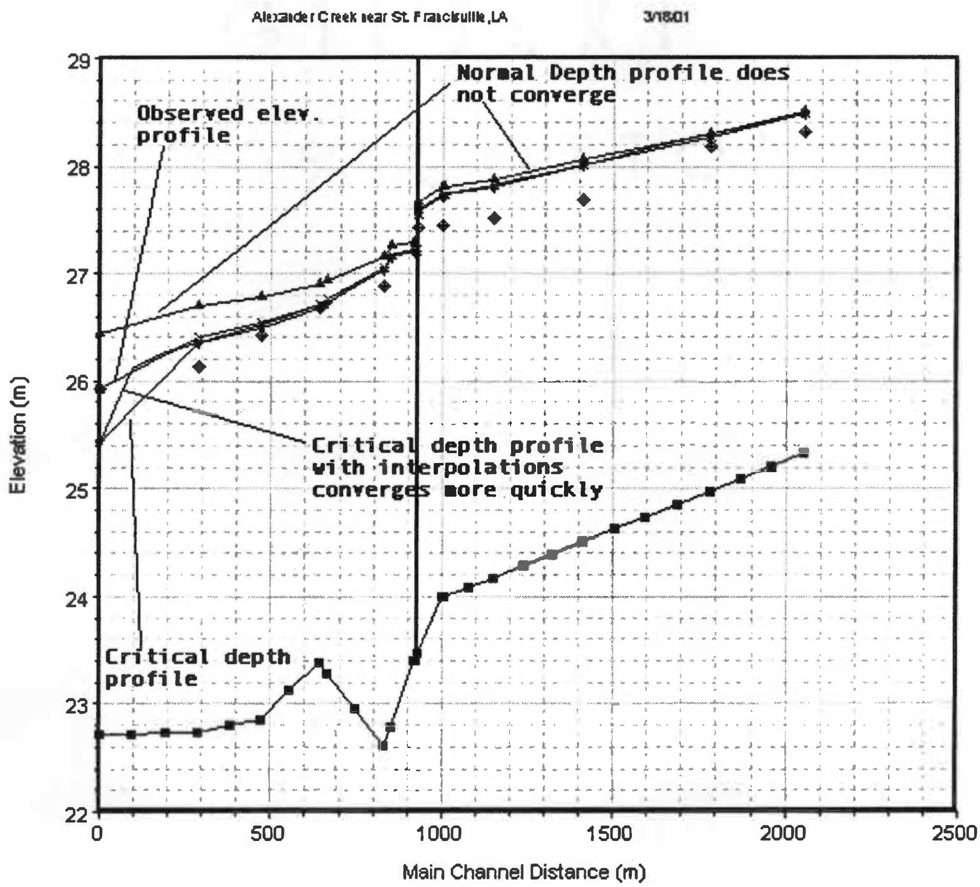


Figure 28: Effect of interpolated cross-sections on computations.

Figure 28 also shows the effect of extra cross-sections upon the errors induced by inaccurate boundary conditions. The third profile shown in Figure 28 was computed with interpolated sections near the downstream boundary. Additional computation steps caused by the interpolated cross-sections cause the profile to converge more quickly than without the additional sections.

Chapter VI

Summary & Conclusions

Flooding causes over five billion dollars of damage and 99 fatalities per year in the United States alone. These ever increasing numbers indicate a need for rigorous design procedures for any structures which exacerbate existing flooding. RAS is the most recent software program developed to aid in design of hydraulic structures.

RAS is currently the best method for hydraulic design of bridges, and documentation available with RAS provides guidance for its use. The author is experienced in hydraulic design of bridges and has developed this thesis in order to address certain generalities and gaps in the RAS documentation. An effort was also made to reconcile practices recommended by RAS for location of the approach and exit section with recommendations made by other hydraulic design programs.

The primary issues examined by this thesis were:

1. Optimum placement of approach and exit cross-sections and transition length requirements. Recommendations from WSPRO, HEC-2, and RAS were examined as well as a situation with no transitions.
2. Evaluation of low flow bridge analysis methods. RAS documentation gives general guidelines for favorable use of the momentum method, but does not discuss specific circumstances which cause a favorable situation.

3. Evaluation of high flow bridge analysis methods. RAS recommends the energy method for situations where "the bridge opening is not acting like a pressurized orifice". No guidance is given as to when this situation may occur.
4. Effects of interpolating cross-sections. The built-in interpolation features of RAS may be used to formulate new cross-sections in areas where field surveyed cross-section data is lacking. The documentation provides no guidance concerning the effects of this practice.
5. Determination of boundary conditions. Any reach analyzed with RAS requires a user-specified condition at the reach boundary. RAS provides three methods: critical depth, normal depth, and observed water surface elevation. The user must judge which of these is best for the situation being analyzed. Some guidance concerning the effects of boundary values upon the modeling computations was developed.

Conclusions

The following comments and recommendations are based on extensive analysis of twelve separate flood events at nine different bridge locations in Louisiana, Mississippi, and Alabama. Conclusions are based upon comparison of results of water surface elevations computed by various methods to observed water surface elevations.

Care should be taken when applying these recommendations. These are best applied to sites similar to those used during this analysis. These recommendations are best at locations with flat sinuous channels, wide & heavily vegetated floodplains, more than 75-80% of the floodplain constricted by the bridge, and piers obstructing more than 3% of the bridge opening. Flows for this analysis varied from 40 to 290 cubic meters per second with recurrence intervals ranging from 2 years to greater than 100 years, but the majority of flows had a recurrence interval less than 10 years. See Chapter IV for additional site data.

The major conclusions of this thesis are:

- Exclusion of bridge transition reaches for downstream expansion and upstream contraction of flow result in calculated water surface elevations which are much lower than actual elevations. This method does not account for energy losses due to expansion and contraction, and calculated elevations are too low. Realistic hydraulic modeling requires including both expansion and contraction reaches.

- HEC-2 recommends that the exit section be placed four times the obstructed length downstream. This recommendation is based upon flume tests. Application of this practice to actual bridge sites results in over estimation of losses and water surface elevations within the bridge reach and this practice should not be followed. See below for further recommendations on this issue.

- Regression equations developed by Hunt and Brunner (1995) based upon two-dimensional models result in the most accurate calculations of water surface elevations. These equations are provided with RAS documentation and should be used when designing bridges with RAS.

- Bridges with large flows and piers which block more than approximately 5% of the bridge opening should be analyzed using the momentum method. These bridges fall into the category of "significant pier losses" discussed by RAS documentation.

- Using the energy method at bridges experiencing pressure flow only can result in extremely large errors. For this situation, the pressure/weir method should be used.

- Interpolating cross-sections gives more detailed water surface profiles in portions of the model reach that did not previously contain cross-sections. However, there is a point of diminishing returns beyond which additional cross-sections do not affect the appearance of the water surface profile. This point depends upon the length of the modeled reach and the spacing of existing cross-sections. In the studied reaches spacing of 100 meters appeared to be the most efficient. Available data was insufficient for providing more detailed guidance on this subject. Further work could result in a dimensionless criteria for determining the optimum spacing of interpolated sections.

- Interpolation of sections does not seem to affect elevations at existing sections except in situations that involve rapidly varied flow, such as within bridge reaches.

- Poorly chosen boundary values will affect computed elevations upstream. Due to the iterative nature of computations by the standard step method, the effects will diminish with each calculation step upstream.

- Additional sections interpolated in the vicinity of the boundary help to alleviate the effects of a poorly chosen boundary value. Uncertain boundary conditions require additional sections near the boundary in order to minimize negative effects of a poorly chosen boundary condition.

Recommendations For Future Work

Further guidance for identifying situations where pier losses are significant would be beneficial as well. RAS documentation gives no advice about factors which cause this situation.

Additional work to determine the limits of effectiveness for the pressure/weir calculation method would be beneficial. RAS has a built in mechanism to disregard pressure/weir calculations if the tailwater depth exceeds 95% of the headwater depth as well as built in adjustments for weir submergence. These factors help the accuracy of the pressure weir method, however, guidance for choosing the energy method instead of the pressure/weir method is lacking.

Additional work might also be beneficial in the area of pier drag. RAS documentation (HEC, 1997) recommends pier drag coefficients for several types of piers. However, piers with multiple posts similar to that shown previously in Figure 24 are not included. The closest approximation is a square nose pier. Using the value provided for a square nose pier does not entirely account for losses caused by a multiple post type support structure. A square nose pier is entirely one piece along the width of the bridge deck, while a multiple post bent has several posts in a line under the bridge deck. Multiple post piers will have additional losses due to turbulence as flow encounters each additional post

The most obvious recommendation for further work is the need for additional types of bridge sites. All sites discussed in this thesis were on wide, heavily vegetated floodplains with sinuous channels. Similar analysis at bridges with a variety of situations would help to clarify design practices. Situations such as bridges with narrow and deep floodplains, or floodplains with multiple bridges, or culverts could all benefit from work similar to this. This appears to be unlikely at this time as the necessary data is unavailable. Brunner and Hunt performed a nationwide search for sites with the necessary data for model verification and had little success. The author searched throughout the state of Tennessee and was unable to find sites which met the necessary data requirements.

Further work in the area of model verification would require extensive field work in order to gather sufficient data.

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