A SunCam online continuing education course

Floodplain Engineering - Part 1

Modeling Flood Profiles

Using HEC-RAS

by

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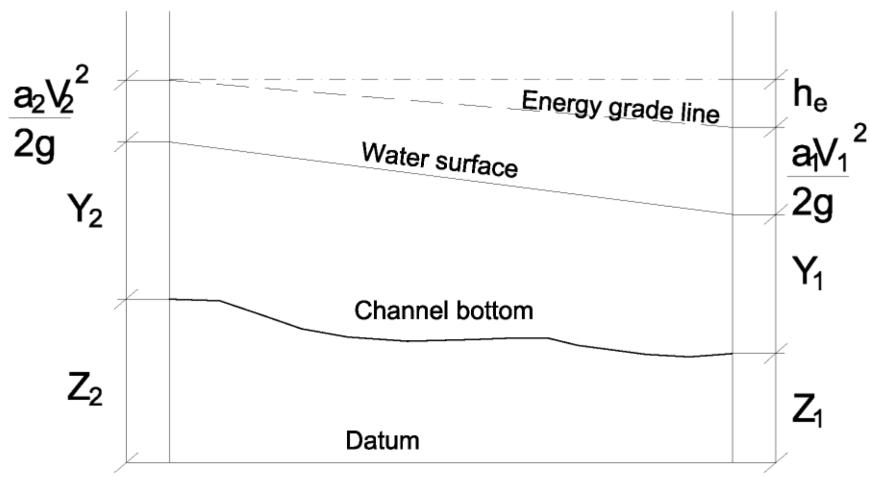
Introduction:
This course presents a discussion of the modeling procedure for flood profiling. The most commonly used software for flood profiling is the U. S. Army Corps of Engineers' River Analysis System (HEC-RAS) and that is the software that is featured in this course. The current version of the HEC-RAS computer software (as of January 2018) is version 5.0.3. HEC-RAS represents a significant advancement over the Army Corps of Engineers earlier flood modeling program, known as HEC-2.

When you complete this course you should be familiar with all of the input parameters involved in a basic flood profiling model and should be able to analyze a river reach using HEC-RAS. This course is not meant as a substitute for the HEC-RAS user's manual but is intended to provide real life examples to guide the user through the river analysis process. Because HEC-RAS is such a sophisticated analysis tool only an overview of its capabilities can be provided in this course. The input parameters for basic stream modeling are included but more advanced HEC-RAS applications are beyond the scope of this course.
Determining the flood profile for various precipitation events is of considerable importance for many reasons, including determining whether a property requires flood insurance, analyzing whether a site is suitable for a particular land use, designing a replacement bridge or culvert and for obtaining governmental approval for a project within a floodplain. In New York State, the agency that has jurisdiction over floodplain issues is the New York Department of Environmental Conservation. In other states, various state agencies have jurisdiction over these areas.

For this course the primary reference is the Army Corps of Engineers technical documentation for the software. The software is available for free download at the US Army Corp of Engineer's website: http://www.hec.usace.army.mil/software/hec-ras/downloads.aspx

Basis of the Calculations:
In order to establish the flood profile along a stream reach it is necessary to balance the energy at each of the cross sections within the reach. This is done by solving the Energy Equation at each cross section. This process is sometimes referred to as a "backwater analysis". The Energy Equation is:
$Y_2 + Z_2 + (a_2 V_2^{\,2} / 2g) = Y_1 + Z_1 + (a_1 v_1^{\,2} / 2g) + h_e$ where:
Y_1 and Y_2 are the depths at cross sections 1 and 2, respectively,
Z_1 and Z_2 are the elevations of the main channel inverts at the two cross sections

a_1 and a_2 are the velocity weighting coefficients at the two cross sections
g is the acceleration due to gravity
h_e is the energy head loss between the two cross sections. This is actually composed of two, parts; the friction losses and the contraction or expansion losses.
The energy head loss can be calculated by the following equation:
$H_e=LSf+C|(a_2V_2^2/2g)-(a1V_1^2/2g)|$ where
L is the average weighted reach length,
Sf represents the friction slope between the two cross sections, and
C is the expansion or contraction loss coefficient.
The remaining terms in the energy equation are shown schematically below:

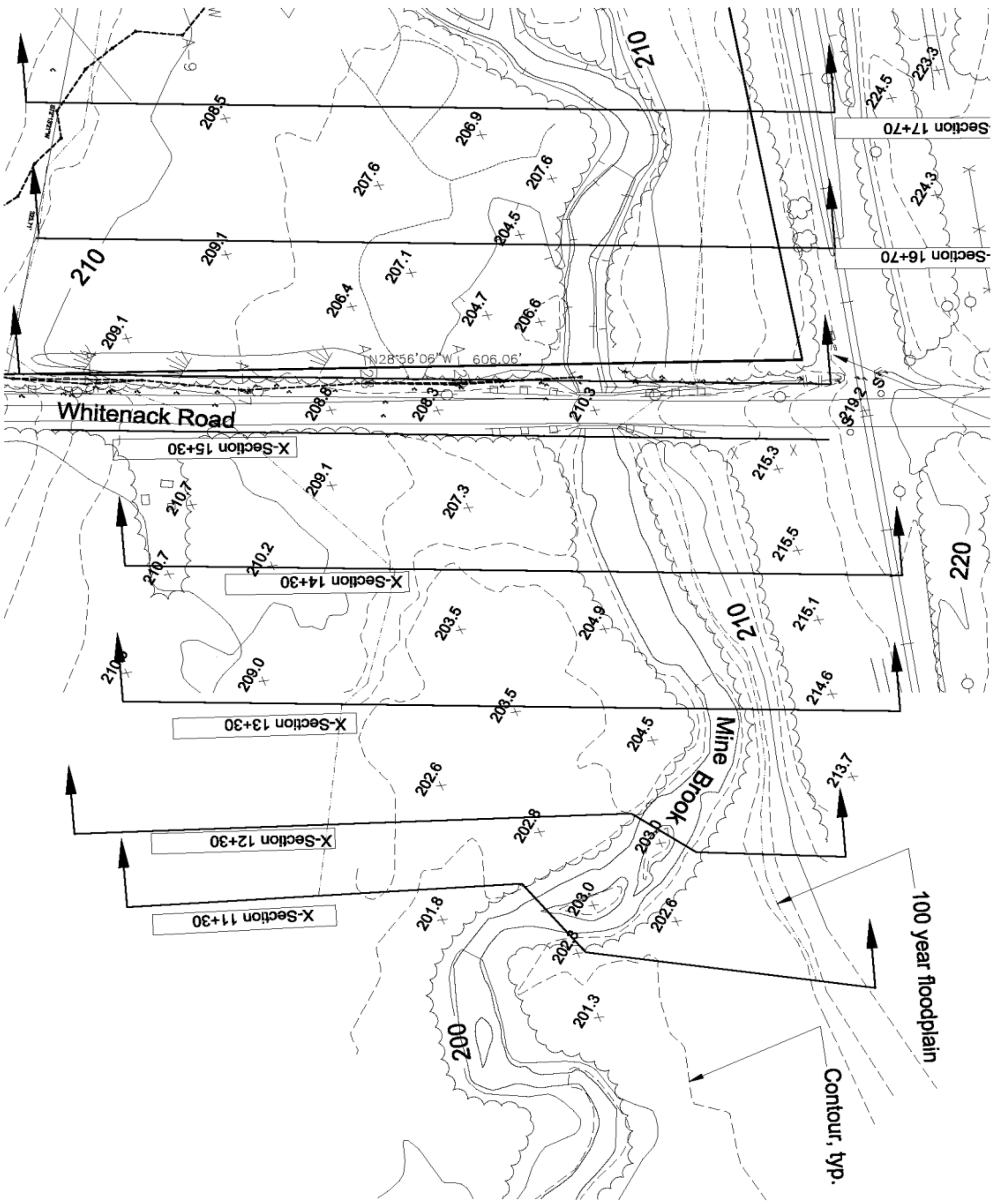


Energy Equation Schematic

Note that the equation above uses the absolute value of the energy multiplied by the expansion or contraction loss coefficient.
HEC-RAS actually has the capability of modeling three different flow regimes: (i) steady flow, (ii) unsteady flow, and (iii) movable boundary – sediment transport flow. However, only steady flow modeling will be considered in this course.

General Modeling Considerations:

In order to model the flood profile through a river reach, it is necessary to first input the stream parameters. HEC-RAS first requires the engineer to model the river reach in plan view. Note that although this seems simple, it is actually one of the key elements in properly modeling the river system. Cross sections should generally be drawn at intervals of approximately 100 feet and they should be drawn perpendicular to the flow direction. This is simple in the channel as the flow is generally parallel with the banks. However, in the overbank sections, the cross sections often have to be bent to properly model the system. Remember that the cross-sections should be drawn so that they are perpendicular to the flow lines both in the channel and in the overbank areas. The schematic on the next page illustrates the placement of cross sections for use in a HEC-RAS model:


Section 17+70
-Section 16+70
X-Section 15+30
X-Section 14+30
X-Section 13+30
X-Section 12+30
X-Section 11+30
Whitenack Road
Mine Brook
100 year floodplain
Contour, typ.
210
210
210
220
200
N28°56'06"W 606.06'
208.5
206.9
207.6
207.6
224.5
223.3
224.3
209.1
207.1
204.5
206.4
204.7
206.6
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208.8
208.3
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219.2
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207.3
215.3
210.7
210.2
215.5
203.5
204.9
215.1
209.0
214.6
203.5
204.5
202.6
213.7
202.8
201.8
203.0
202.6
201.3

The actual flood profile generated by HEC-RAS is a function of the various input parameters. These parameters include the following:

1. Cross section geometry.
2. Manning's roughness coefficients (n values) for the channel and overbank areas.
3. Distance between cross sections.
4. Design flood in cubic feet per second (CFS).
5. Bridges & Culverts.
6. Flow expansion and contraction coefficients.
7. Any other data relative to the stream that will affect the resulting flood profile

Entering Cross Section Data:

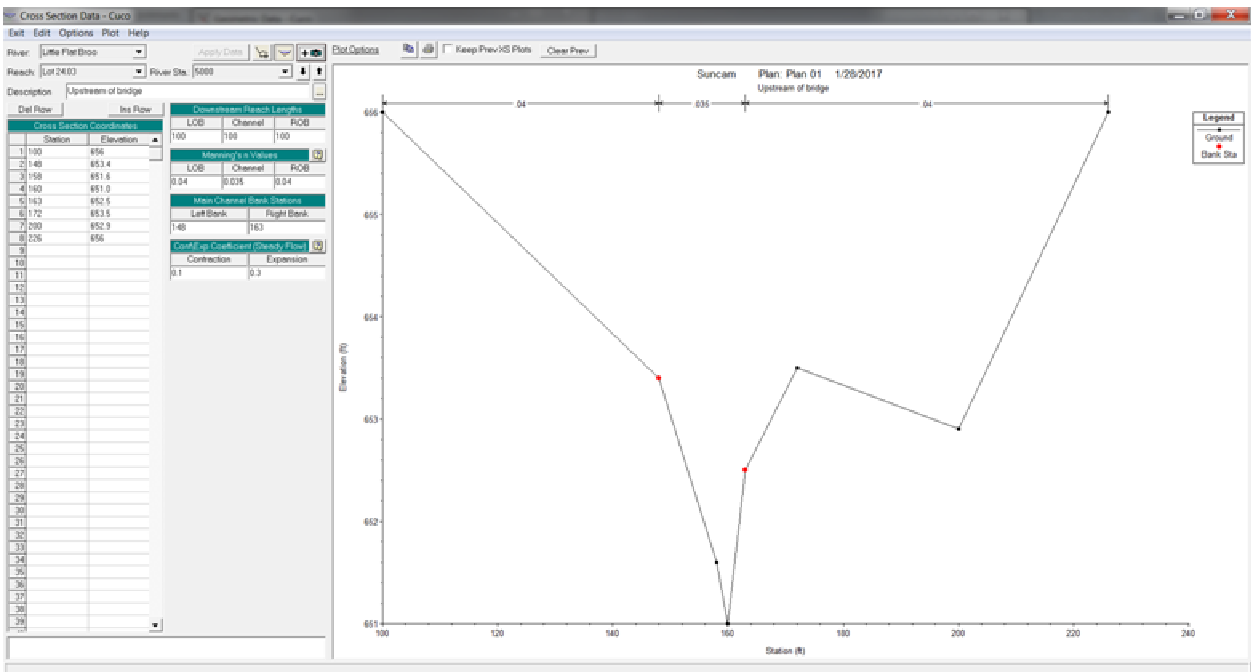
The cross section geometry of the stream reach represents the "bread and butter" of the HEC-RAS input data. The ground points describing the cross section are input using x and y coordinates. After the ground shots have all been entered into the model, the engineer then inputs the following additional data:

1. The reach lengths. Note that a separate reach length is required for the channel and for each of the overbank areas. In many cases, all three of these reach lengths will be the same. However, as shown in the figure above, in some cases they will be different.
2. The Manning's n value (or roughness coefficient) must be input for the channel and for each overbank area. A table of typical n values is presented below (This information is adapted from "Open Channel Hydraulics", by Ven Te Chow and other sources). Published sources are available showing a vary wide variety of channel characteristics. Basically, the more obstructions (in the form of rocks, vegetation, debris, etc.) the more "roughness" in the channel, and, consequently, the higher the resulting 'n' value.

Description of Channel	**Range of n values**
Concrete	0.015 to 0.20
Natural channel on plain, straight, clean & without rifts or deep pools.	0.025 to 0.033
Same as above but with some pools, weeds, & stones.	0.030 to 0.045
Natural channel, with very weedy channel or floodways with heavy stands of timber.	0.075 to 0.15
Mountain streams with a bottom of cobbles.	0.040 to 0.070

3. The stations corresponding to the left and right banks of the channel. This information is used by the program to divide the flow into channel and overbank areas.
4. The expansion and contraction coefficients.

A screen shot of the input data is shown below. On the left is the input data including x and y coordinates of the ground points on the cross section, reach lengths of the overbank and channel areas, Manning's n values, and expansion and contraction coefficients. The cross section is plotted on the right.



One of the limitations of the HEC-RAS program that the reader may have noticed by now is that it divides the cross section neatly into three parts: the channel and the two overbank areas. Many stream reaches do not lend themselves to this type of subdivision. Consider, for example, a stream that has a small parking lot on the right side followed by a heavy woodland and on the left side is bordered by a margin of heavy brush followed by a ball field. In this case, it would be more accurate to divide the cross section into 5 separate parts. However, HEC-RAAS does not allow this. Therefore, the engineer must make a composite of the overbank areas and try to weigh the different contributions that each would play in passing the flood flow. Also, the engineer must sometimes make a judgment as to the extent of the channel. Imagine a situation with a concrete-lined channel with side slopes of 3 vertical to 1 horizontal. Beyond this on each side is a grassy bank with a flatter slope. Beyond the grass banks on both sides are dense stands of woods. Does the channel consist of the concrete flume alone or the concrete in conjunction with the grass banks? This depends on many factors including the expected depth of the flooding. However, it points out one of many situations where engineering judgment must be used in order to have the software properly model the real-life situation.

The photograph below shows a typical wooded stream. Note that the channel is full of boulders and choked by fallen trees. In addition, the overbank areas are densely vegetated. These conditions can be represented by employing a large Manning's roughness coefficient (n). In a case like this one, an n value of 0.05 to 0.06 might be appropriate for the channel and an n value of 0.08 to 0.1 would probably be right for the overbank areas.

The photograph below shows a stream flowing through a wooded meadow. Note that this stream channel is free of boulders, downed timber, and other obstructions. A lower n value of approximately 0.035 would be appropriate for this channel. The overbank meadow areas would not cause nearly as significant a retardance of the flow as the woods would in the previous photograph. Therefore, a lower n value of perhaps 0.05 to 0.06 would be right for modeling the overbank areas in this case.

Yet another stream reach is shown below. In this case, the overbanks travel through a variety of land uses in quick succession, with a ball field to the left and a woods, parking lot, and garage all visible adjacent to the stream on the right. It is advisable to average these land covers to obtain a composite n value for the overbank areas. The somewhat rocky channel n value in this case should probably have an n value of 0.035 to 0.04.

The photograph below shows a winding stream through a meadow. Note that the channel bends around the trees and then sharply back in a sinuous pattern. In this case the water in the channel will travel over a much longer path than floodwaters in the overbank reaches. This fact can be modeled by adjusting the reach lengths in the cross section input data. Based on the relatively clean channel, an n value of approximately 0.035 could be used in this case as well.

The photograph below shows a stream channel enclosed by rock walls. The n value for a channel like this should probably be set at about 0.04 to 0.05. It is a fairly straight, clean channel reach but the bottom has some cobbles and the walls themselves will cause some friction.

Modeling Bridges and Culverts:
Obviously, bridges and culverts are critical features in modeling flood profiles. These structures often constrict the flow during flood events and can, consequently, cause major backwater effects far upstream. Like its predecessor, HEC-2, HECRAS has two main modeling options for structures – bridges and culverts. It is up to the analyzing engineer to determine which method is the most appropriate for a real-world application.

HECRAS has the capacity to model a variety of flow types through bridge structures. These can be divided into the following:

1. Low flow computations. This occurs when the flow through the bridge opening can be represented by open channel flow. Low flow is classified as Class A if it is completely subcritical flow, Class B if it is a mixture of subcritical and supercritical flow and Class C if it is entirely supercritical flow. (A very brief discussion of subcritical vs. critical flow is included below). HEC-RAS includes four separate methods for computing losses under low flow conditions through the bridge. The user can choose between any of the following methods:
   - Energy equation (also known as the standard step method).
   - Momentum balance.
   - Yarnell Equation. This is an empirical equation (based on laboratory experiments) that predicts the change in water surface through the bridge structure. The Yarnell Equation is explained in more detail below.
   - FHWA WSPRO method. This method computes the water surface profile through a bridge by solving the energy equation. It first computes the profile with no bridge in place and then adds incremental losses between the sections making up the bridge.
2. Pressure flow computations. This occurs when the water surface comes into contact with the low chord of the bridge.
3. Weir flow computations. This occurs when the water surface overtops the bridge deck. Flow over the bridge and the roadway approaching the bridge is the calculated using the weir equation: Flow over a sharp crested weir can be approximated by the equation: $Q = cLH^{3/2}$, where Q= the flow over the weir in cubic feet per second (CFS), C is the weir coefficient. This is generally assumed to be approximately equal to 2.6. L is the effective length of the weir (or bridge/roadway combination) in feet, and H is the head in feet.

4. Combined pressure/weir flow computations. If combined flow is occurring, the program will use an iterative approach to determine the amount of each type of flow.

The Yarnell Equation is expressed as:

$H_{3-2} = 2K(K + 10w - 0.6)(a + 15a^4)V_2^2 / 2g$, where:

H_{3-2} is the drop in water surface from cross section 3 to cross section 2
K is the Yarnell's pier shape coefficient.
The table below can be used to determine the pier coefficients:

Pier Shape	**Shape Coefficient**
Semi-circular nose & tail	0.9
Triangular nose & tail	1.05
Square nose & tail	1.25

w is the ratio of velocity head to depth at cross-section 2,
a is the obstructed area of the piers divided by the total unobstructed area, and
V_2 is the velocity downstream at cross-section 2,
and g is the acceleration due to gravity (32 feet per second squared).

The photograph below shows a bridge with piers that need to modelled for use in the Yarnell Equation.

Discussion of Subcritical vs. Supercritical Flow

Flow within a river reach can be classified as subcritical or supercritical flow depending on the energy characteristics of the flow. Ordinarily, flow in most river reaches is subcritical. However, the flow regime can switch to critical or supercritical if the slope is excessive and the velocity increases. The actual flow regime can be calculated by determining the Froude Number, which is the ratio of a wave on the water surface to the speed of the water.

The Froude Number is calculated using the following formula:

$Fr = (V / gD)^{1/2}$, where:

Fr is the Froude Number

V is the velocity in feet per second

g is the acceleration due to gravity (32 feet per second squared).

D is the hydraulic depth (cross sectional area divided by flow top width)

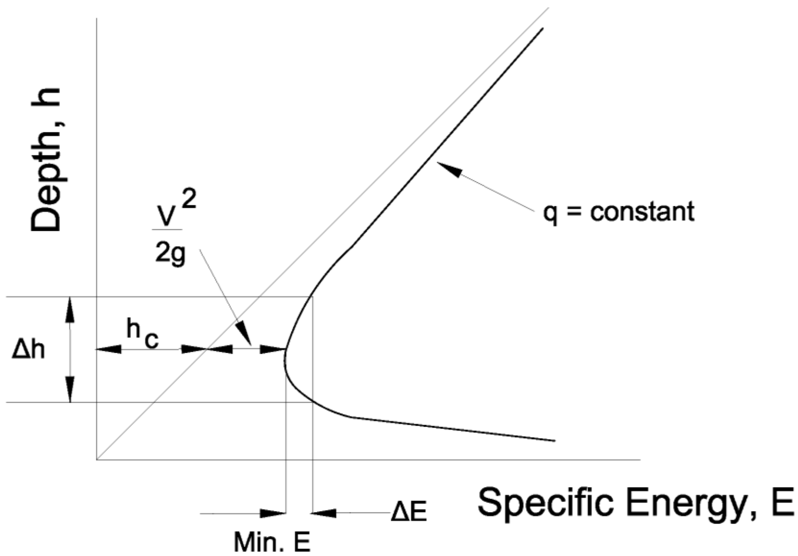
If the Froude Number is greater than 1, the flow is supercritical.
If the Froude Number is equal to 1, the flow is critical.
If the Froude Number is less than 1, the flow is subcritical.

If the Froude Number is 1(i.e. critical; flow) the resulting depth is known as critical depth. Depths near critical depth are inherently unstable because small changes in surface energy can lead to large changes in the local flow depth.
The specific energy is calculated using the following equation:
$E = h + (V^2 / 2g)$, where:
E is the specific energy
v is the velocity
h is the flow depth
g is the acceleration due to gravity (32 feet per second squared).

Example 1: Determine the flow regime of a stream channel that has a velocity of 5 FPS, a cross sectional area of 20 SF and a top width of 8 feet.

Solution: First, determine the hydraulic depth (D), which is the cross sectional flow area divided by the top width. D=20/8=2.5.
Next, plug the values into the equation for the Froude Number: $Fr = (5/(32)(2.5))^{1/2} = 0.25$
The Froude Number is less than 1. Therefore, the flow is subcritical.

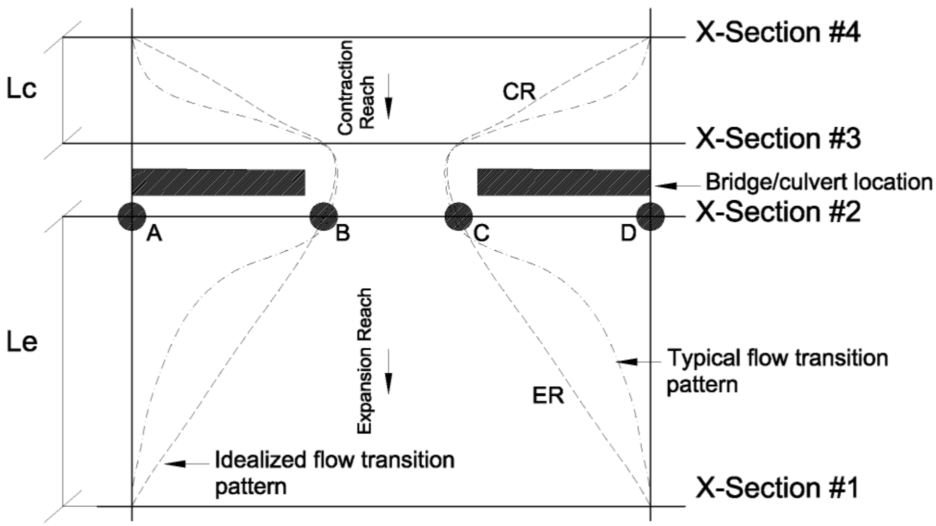
The solution to the energy equation is a parabola as shown in the specific energy curve below.



Specific Energy Curve for Open Channel Flow

Getting back to low flow bridge conditions, the photograph below shows a footbridge over a small stream in a municipal park. The photograph shows a low flow situation (probably Class A because the stream bed is not steeply sloping and subcritical flow would be expected). If the water level reaches the bottom chord of this bridge pressure flow will occur in the stream bed. However, because the bridge deck is higher than the adjacent ground (at least on the right bank), at this point combination flow will result; pressure flow in the channel and weir flow in the overbank. If the flood level increased to a point where it flowed over the bridge deck, pure weir flow might result.

When modeling flow through the bridge it is imperative to locate the cross sections properly. The proper location of the cross sections is shown schematically below.



Bridge & Culvert Cross Section Locations

Cross section #1 must be located far enough downstream of the bridge that the flow is no longer affected by the structure. In other words, the flow at cross section #1 is fully expanded. The distance from the bridge to cross section #1 is most accurately determined by field investigations. However, these are generally not possible during flood conditions. Therefore, the appropriate distance can be approximated by using the expansion ratios listed in the following table. In this table, the ratio b/B is the ratio of the bridge opening width to the floodplain width at the bridge. Even this table shows that this is not an exact science and a range of expansion ratios is given for each of the various combinations of slopes, b/B ratios and n values. These are the expected ranges and the higher values are generally associated with higher discharge rates.

b/B	Slope (feet/mile)	$n_{ob}/n_c=1$	$n_{ob}/n_c=2$	$n_{ob}/n_c=4$
0.10	1	1.4-3.6	1.3-3.0	1.2-2.1
0.10	5	1.0-2.5	0.8-2.0	0.8-2.0
0.10	10	1.0-2.2	0.8-2.0	0.8-2.0
0.25	1	1.6-3.0	1.4-2.5	1.2-2.0

0.25	5	1.5-2.5	1.3-2.0	1.3-2.0
0.25	10	1.5-2.0	1.3-2.0	1.3-2.0
0.50	1	1.4-2.6	1.3-1.9	1.2-1.4
0.50	5	1.3-2.1	1.2-1.6	1.0-1.4
0.50	10	1.3-2.0	1.2-1.5	1.0-1.4

Once an expansion ratio is selected, the distance to the downstream reach (shown as "Le" in the figure above) is calculated by multiplying the expansion ratio by the average obstruction length. This is the average of the distances A to B and C to D in the figure above.

Example 2: Determine the correct placement of cross section #1 in modeling a bridge, given the following parameters:

- The distance from A to B (in the figure above) is 40 feet and the distance from C to D (in the figure above) is 60 feet.
- n value of the channel = 0.035
- n value of the overbank areas =0.06
- Slope = 0.002 feet per foot
- The bridge opening width is 15 feet.
- The floodplain width at the bridge is 50 feet.

Solution: Using the table above, enter the following:
b/B = 15/50 = 0.3. (Use b/B = 0.25 in the table).
slope = 0.002 feet per foot. This translates to a slope of 10.5 feet per mile. (Use S=10 ft/mile in the table).
The ratio of n values from the overbank to the channel is (0.06/0.035) or 1.71. (Use n_{ob}/n_{ch} = 2 in the table).
From the table above, the range of expansion values is 1.3 to 2. We will use the average of these 2 values which is 1.65. This value is then multiplied by the average obstruction width (which is the average of the distances A to B and C to D in the figure). This average is 50 feet.
Therefore, Cross Section #1 should be placed approximately 1.65 X 50 feet = 82.5 feet downstream of the bridge.
This simple example shows just how much engineering judgment is required in this kind of analysis.

Cross section #2 is located just a few feet downstream of the bridge and should represent the natural ground just outside the structure. Ordinarily, this is located at the toe of the downstream embankment.

Cross section #3 is somewhat analogous to cross section #2 and should be located just upstream of the bridge. This cross section is generally located at the toe of the upstream embankment.

Cross section #4 should be located just far enough upstream for the flow lines to be approximately parallel and for the full cross section to be effective. (i.e. This cross section should be just upstream of any flow constricting affect of the bridge). Under most field conditions flow contraction caused by a bridge structure occurs over a shorter distance than flow expansion downstream of the bridge. The distance between cross sections #3 and #4 is best determined by field inspection during high flow conditions. However, as stated above, this is often not possible, and the engineer should use best judgment in placing this cross section.

There are several basic input parameters required for a bridge. First the river reach, station, and bridge description are entered into the program. Then, the following parameters associated with the bridge deck and/or roadway are entered into the model:

1. Distance. This is the distance between the upstream side of the bridge deck and the cross section located immediately upstream of the bridge.
2. Width. The name of this parameter is somewhat confusing as this is the width of the bridge deck along the stream, or in other words, it is the length of the bridge deck along the stream. Therefore, the distance between the bridge deck and the downstream cross section will equal the main channel reach length minus the sum of the bridge "width" and "distance" value entered above.
3. Weir Coefficient. A weir coefficient of approximately 2.6 is generally used for flow over the bridge deck and roadway.
4. Upstream Stationing, High Chord, & Low Chord. This information allows the user to define the geometry of the bridge deck on the upstream side of the bridge. *Note that the stationing of the deck does not have to equal the stations input for the bounding cross sections, but it must be based on the same origin.*
5. Downstream Stationing, High Chord, & Low Chord. This is analogous to the above, except that it represents the downstream geometry of the bridge deck. If this geometry is the same as the upstream side (which is often the case), the user can simply press the "Copy Up To Down" button.
6. Upstream Embankment Side Slope. This should be entered as the vertical to horizontal distance ratio of the embankment.
7. Downstream Embankment Side Slope. This should be entered as the vertical to horizontal distance ratio of the embankment.

8. Maximum Allowable Submergence. This is the maximum allowable submergence ratio that can occur during weir flow calculations. Once this ratio is exceeded, the program will switch to energy based calculations, instead of using pressure and weir flow.
9. Submergence Criteria. The user here determines which method is to be used to determine how much the weir coefficient should be reduced due to submergence. Obviously, once the bridge deck is significantly submerged, the characteristics of the weir flow will be modified. There are two choices: one based on a trapezoidal shaped weir and the other developed for an Ogee spillway shape. (An Ogee spillway is analogous to a sharp crested weir).
10. Minimum Weir Flow Elevation. This specifies the minimum elevation at which weir flow will occur.

When modeling a bridge (or a culvert) the user often can make use of the "ineffective flow area" option on the bounding cross sections. Because the flow is generally restricted in the immediate vicinity of the structure, the flow through the cross sections immediately upstream and downstream can be significantly affected. The use of this feature takes out areas of flow that abut the roadway embankment and are not truly effective in carrying the floodwaters. If this feature is not employed properly, wildly inaccurate flood profiles may result.

In addition to these basic elements, the program allows the user to add a number of other parameters if they are encountered. These include piers, sloping abutments, and several other features that are sometimes encountered in bridges.

Under low flow conditions, the user can specify which of the computational options to use. The first is known as the Bridge Modeling Approach. The second option is known as the WSPRO low flow option. The following will assist in determining which of these options to choose for a particular bridge:

1. In cases where the bridge piers are a small obstruction to the flow, and friction losses predominate, the energy method, the momentum method, and the WSPRO method will yield the most realistic answers.
2. When pier losses and friction losses are both significant, the momentum method is the most applicable. However, under this scenario, any of the methods can be used.
3. If the flow passes through critical depth within the vicinity of the bridge, the WSPRO method cannot be used, as it was developed for subcritical flow only.
4. For supercritical flow, both the energy and momentum methods can be utilized. The momentum method will yield more realistic results for bridges that have a

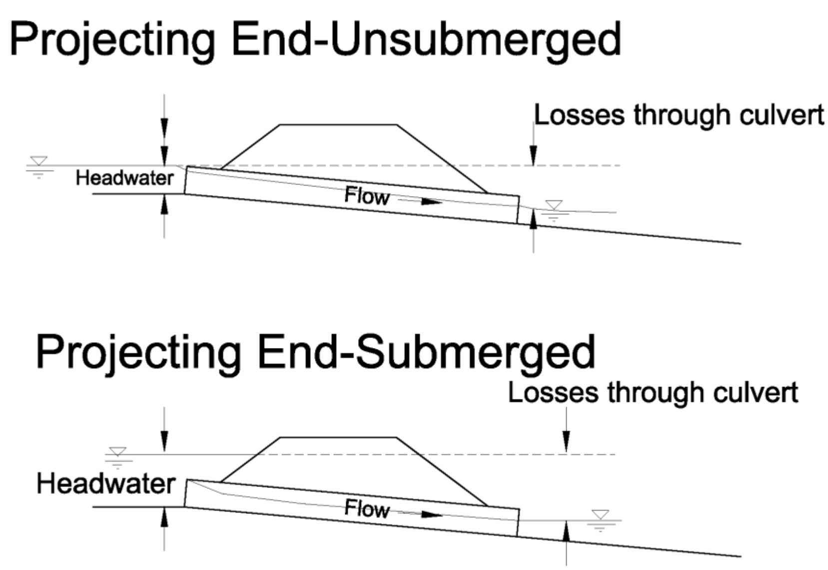
substantial amount of pier impacts. Remember that the Yarnell equation and the WSPRO method are both only applicable to subcritical flow regimes.

5. For bridges in which the piers are the major contributor to energy losses (and subsequent changes in the water surface profile), the user should employ either the Yarnell equation or the momentum method. However, the Yarnell equation is only applicable for Class A low flow.
6. For long culverts under low flow conditions, the energy based, standard step method will yield the most accurate results.

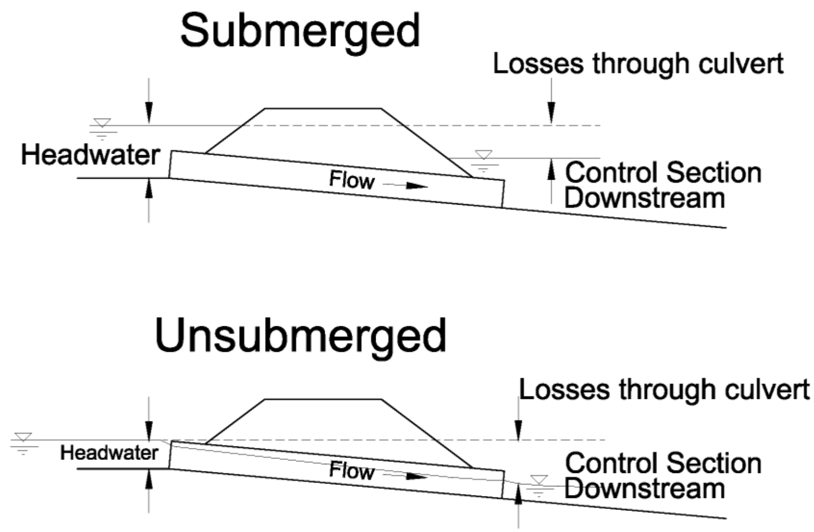
The photograph below shows a structure that is more properly modeled as a culvert.

Culvert hydraulics can be quite involved. For ease in computations, either "inlet control" or "outlet control" is generally assumed. Inlet control flow conditions occur when the flow capacity of the culvert barrel is greater than the flow carrying capacity of the culvert entrance. Outlet control flow conditions occur when either the culvert carrying capacity is limited by downstream conditions or by the flow capacity of the culvert barrel. One of the best references for culvert hydraulics is still the US Department of Transportation's "Hydraulic Charts for the Selection of Highway Culverts".

Culvert hydraulics are shown schematically in the following figures:



Culvert: Inlet Control Flow Conditions



Culvert: Outlet Control Flow Conditions

The location of the cross sections in modeling the culvert are the same as those used in modeling a bridge.

There are several basic input parameters required for a culvert. These are described briefly as follows:

1. Culvert ID#.
2. Solution Criteria. This field allows the user to either specify inlet or outlet control or else to allow the program to choose between the higher of the two methodologies.
3. Culvert Shape. The program recognizes nine different culvert shapes. (These include the following):
    - Circular.
    - Box.
    - Pipe Arch.
    - Ellipse.
    - Semi-circle.

- Arch.
- Low Arch.
- High Arch.
- Conspan Arch.

4. Span. This field is left blank for circular culverts.
5. Rise. (This is the height (or diameter) of the culvert in feet).
6. Chart #. This field is filled out according to the Federal Highway Administration Chart number that corresponds to the shape and type of the culvert in question. A copy of this chart is included below.

Chart #	**Scale #**	**Description**
1	---	**Concrete Pipe Culvert**
	1	Square edge entrance with headwall
	2	Groove end entrance, with headwall
	3	Groove end entrance, pipe projecting from fill
2	---	**Corrugated Metal Pipe Culvert**
	1	Headwall
	2	Mitered to conform to slope
	3	Pipe projecting from fill
3	---	**Concrete Pipe Culvert, Beveled Ring Entrance**
	1(A)	Small bevel; b/D=0.042, a/D=0.063, c/D=0.042, d/D=0.083
	1(B)	Large bevel; b/D=0.083, a/D=0.125, c/D=0.042, d/D=0.125
8	---	**Box Culvert with Flared Wingwall**
	1	Wingwalls flared 30 to 75 degrees
	2	Wingwalls flared 90 or 15 degrees
	3	Wingwalls flared 0 degrees (sides extend straight).
9	---	**Box Culvert with Flared Wingwalls and Inlet Top Edge Bevel**
	1	Wingwall flared 45 degrees, inlet top edge bevel=0.43D
	2	Wingwall flared 18 to 33.7 degrees, inlet top edge bevel=0.83D
10	---	**Box Culvert; 90 degree Headwall; Chamfered or Beveled Inlet Edges**
	1	Inlet edges chamfered ¾ inch
	2	Inlet edges beveled 2 inches per foot at 45 degrees (1:1)
	3	Inlet edges beveled 1 inch per foot at 33.7 degrees (1:1.5)

11	---	**Box Culvert; Skewed Headwall; Chamfered or Beveled Inlet Edges**
	1	Headwall skewed 45 degrees; inlet edges chamfered ¾ inch
	2	Headwall skewed 30 degrees; inlet edges chamfered ¾ inch
	3	Headwall skewed 15 degrees; inlet edges chamfered ¾ inch
	4	Headwall skewed 10 to 45 degrees; inlet edges beveled
12	---	**Box Culvert; Non-Offset Flared Wingwalls, ¾ inch Chamfered at Top of Inlet**
	1	Wingwalls flared 45 degrees (1:1); inlet not skewed
	2	Wingwalls flared 18.4 degrees (3:1); inlet not skewed
	3	Wingwalls flared 18.4 degrees (1:1); inlet skewed 30 degrees
13	---	**Box Culvert; Offset Flared Wingwalls; Beveled Edge at Top of Inlet**
	1	Wingwalls flared 45 degrees (1:1); inlet top edge bevel=0.042D
	2	Wingwalls flared 33.7 degrees (1.5:1); inlet top edge bevel=0.083D
	3	Wingwalls flared 18.4 degrees (3:1); inlet top edge bevel=0.083D
16-19	---	**Corrugated Metal Box Culvert**
	1	90 degree wingwall
	2	Thick wall projecting
	3	Thin wall projecting
29	---	**Horizontal Ellipse: Concrete**
	1	Square edge with headwall
	2	Grooved end with headwall
	3	Grooved end projecting
30	---	**Vertical Ellipse: Concrete**
	1	Square edge with headwall
	2	Grooved end with headwall
	3	Grooved end projecting
34	---	**Pipe Arch: 18” Corner Radius; Corrugated Metal**
	1	90 degree wingwall
	2	Mitered to slope
	3	Projecting
35	---	**Pipe Arch: 18” Corner Radius; Corrugated Metal**

	1	Projecting
	2	No bevels
	3	33.7 degree bevels
36	---	**Pipe Arch: 31" Corner Radius; Corrugated Metal**
	1	Projecting
	2	No bevels
	3	33.7 degree bevels
41-43	---	**Arch: Low-Profile Arch; High-Profile Arch; Semi-Circle: Corrugated Metal**
	1	90 degree wingwall
	2	Mitered to slope
	3	Thin wall projecting
55	---	**Circular Culvert**
	1	Smooth tapered inlet throat
	2	Rough tapered inlet throat
56	---	**Elliptical Inlet Face**
	1	Tapered inlet; beveled edges
	2	Tapered inlet; square edges
	3	Tapered inlet; thin edge projecting
57	---	**Rectangular**
	1	Tapered inlet throat
58	---	**Rectangular Concrete**
	1	Side tapered; less favorable edges
	2	Side tapered; more favorable edges
59	---	**Rectangular Concrete**
	1	Slope tapered; less favorable edges
	2	Slope tapered; more favorable edges
60	---	**ConSpan Span/Rise Approximately 2:1**
	1	0 degree wingwall angle
	2	45 degree wingwall angle
	3	90 degree wingwall angle
61	---	**ConSpan Span/Rise Approximately 4:1**
	1	0 degree wingwall angle
	2	45 degree wingwall angle
	3	90 degree wingwall angle

7. Scale. This denotes the Federal Highway Administration Scale number that corresponds to the type of culvert entrance. It can be taken from the table above.
8. Distance to Upstream Cross Section. This positions the culvert relative to the two bounding cross sections (i.e. cross section #2 and cross section #3).
9. Culvert Length. This is the length along the centerline of the culvert barrel.
10. Entrance Loss Coefficient. The entrance loss coefficient is multiplied by the velocity head inside the upstream end of the culvert. The resulting value represents the energy loss that occurs as the flow transitions from the upstream cross section to inside the culvert barrel. A table of entrance losses is included below.

Pipe Culverts	**----**
Type of Structure & Characteristics of Entrance	**Entrance Loss Coefficient**
Concrete Pipe Projecting from Fill with no Headwall	-----
Socket end of pipe	0.5
Square cut end of pipe	0.2
Concrete Pipe with Headwall or Headwall & Wingwalls	-----
Socket end of pipe	0.2
Square cut end of pipe	0.5
Rounded entrance with rounding radius = 1/12 of pipe diameter	0.2
Concrete Pipe	-----
Mitered to conform to fill slope	0.7
End section conformed to fill slope	0.5
Beveled edges, 33.7 to 45 degree bevels	0.2
Side slope tapered inlet	0.2
Corrugated Metal Pipe or Pipe-Arch	-----
Projected from fill with no headwall	0.9
Headwall or headwall and wingwalls square edge	0.5
Mitered to conform to fill slope	0.7
End section conformed to fill slope	0.5
Beveled edges, 33.7 to 45 degree bevels	0.2
Side slope tapered inlet	0.2

Reinforced Concrete Box Culverts	----
Type of Structure & Characteristics of Entrance	**Entrance Loss Coefficient**
Headwall parallel to embankment with no wingwalls	-----
Square edge on three edges	0.5
Three edges rounded to radius of 1/12 of barrel dimension	0.2
Wingwalls at 30 to 75 degrees to Barrel	-----
Square edge at crown	0.4
Top corner rounded to 1/12 of barrel dimension	0.2
Wingwalls at 10 to 25 degrees to Barrel	-----
Square edge at crown	0.5
Wingwalls parallel (extension of sides)	----
Square edge at crown	0.7
Side or slope tapered inlet	0.2

11. Exit Loss Coefficient. The exit loss coefficient is multiplied by the change in velocity head from inside the culvert to outside the culvert at the downstream end. The resulting value represents the energy loss that occurs as the water exits the culvert.
12. Manning's n for Top. The program allows the user to input different roughness coefficients for the top and bottom sections of the culvert.
13. Manning's n for Bottom.
14. Depth to Use Bottom n. This field is used to specify the depth that divides the culvert from the bottom n value to the top n value. A simplified table of some typical n values for use in culverts is shown below: (The HEC-RAS documentation includes a more detailed table, with additional qualifying parameters).

Type and description of Culvert	**Minimum n**	**Average n**	**Maximum n**
Brass & Smooth steel	0.010	0.013	0.017

Cast Iron	0.010	0.013	0.016
Wrought Iron	0.012	0.015	0.017
Corrugated metal	0.017	0.024	0.030
Lucite, Glass, & Cement	0.008	0.011	0.015
Concrete	0.010	0.013	0.015
Wood	0.010	0.015	0.020
Clay	0.011	0.015	0.018
Brickwork	-----	-----	-----
Glazed	0.011	0.013	0.015
Lined with cement mortar	0.012	0.015	0.017
Rubble masonry, cemented	0.018	0.025	0.030

15. Depth Blocked. This can be used to model a culvert that has the bottom portion silted in or otherwise blocked. Note, however, that if this option is chosen the program assumes that the lower portion of the culvert is blocked throughout the length of the culvert barrel.
16. Upstream Invert Elevation.
17. Downstream Invert Elevation.
18. # of Identical Barrels.
19. Centerline Station. This positions the culvert relative to the bounding cross sections.

The photograph below shows a circular concrete culvert with a headwall and wingwalls. Based on the information provided above, this structure would probably be assigned Federal Highway Chart #1, Scale #2. The entrance loss coefficient would be set at 0.2 and the n value would be set at 0.013. The culvert appears to be clear of debris, so the "depth blocked" option would not be used. Likewise, because the circular culvert is uniform, the top and bottom n values would be the same.

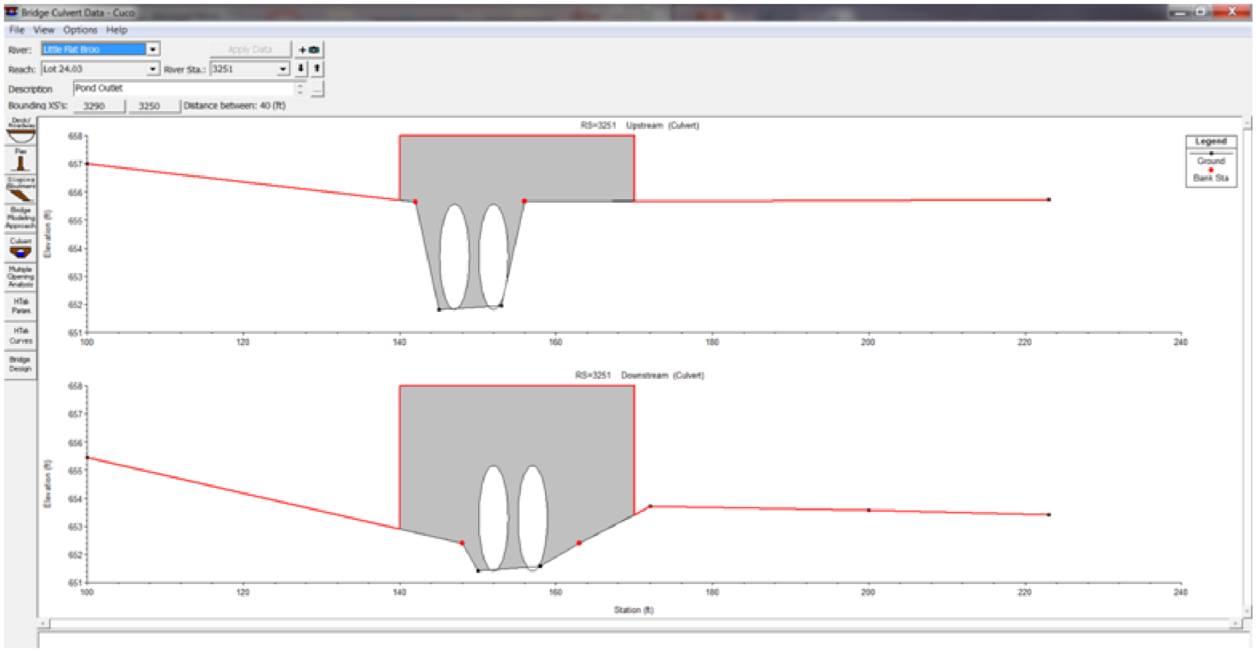
The culvert shown in the photograph below includes a wooden headwall and no wingwalls.

The structure below could be modeled as either a bridge or a culvert. The stone arch would behave like a semi-circular culvert. However, this structure is probably more properly modeled as a bridge.

HEC-RAS also has the capability of modeling multiple bridge and/or culvert openings.

Twin concrete culverts are shown in the photograph below. These culverts carry a small stream through a busy recreational park.

The screen shot below shows a double culvert. The “ineffective flow area” option has been chosen to prevent the overbank areas from contributing to the flow.



Unusual Bridge Situations:
Some relatively common bridge types do not lend themselves to the typical bridge/culvert modeling techniques described above. These are beyond the scope of this introductory course and include the following:

1. Perched Bridges: This is a bridge that has a deck elevation significantly higher than the overbank areas on each side.
2. Low Water Bridges.
3. Bridges on a Skew.
4. Parallel Bridges.
5. Multiple Bridge Opening. (This includes openings of different sizes and shapes).

Entering Steady Flow Data into the Model:
Entering steady flow data into the model is not complicated at all. The user simply inputs the number of profiles to be analyzed, the peak flow data, and any boundary conditions. At least one flow must be provided for each river reach. However, the user can provide additional flow data if, for example, it is desirable to model the flood profile for different storm events. Flow data are

entered from upstream to downstream for a particular river reach. The flow data then can then be changed at any downstream cross section to model tributary flow, the increase in drainage area or any other physical change which will alter the peak flow rate.

Boundary conditions are necessary to establish the starting water surface at the upstream and downstream ends of the river system. If the flow is subcritical, a boundary condition is only required at the downstream end (since the calculations will proceed upstream) whereas if the flow is supercritical, a boundary condition is only required at the upstream end (since the calculations will proceeded downstream). If a mixed flow regime is anticipated, boundary conditions must be provided for both ends of the river. The user has the following options when entering boundary conditions:

1. Known Water Surface Elevations: If the data is available, it is always preferable to enter a known water surface elevation as a starting point. (Note, however, that even if this information is not available, HEC-RAS can provide a very realistic model of the resulting flood. It is generally advisable to start the model at least 5 cross sections beyond the point of interest. This is because the model will work out any instabilities within these 5 cross sections and provide a smoother flood profile beyond them).
2. Critical Depth: If this is specified, the program will calculate the critical depth at the bounding cross section.
3. Normal Depth: If this boundary condition is chosen, the user must specify an actual or assumed slope so that the program can calculate a normal depth using the Manning's Equation. $Q = (1.486/n)AR^{2/3}S^{1/2}$ In this equation, Q is the flow in CFS, n is the Manning's Roughness Coefficient, A is the area, R is hydraulic radius (defined as the cross sectional area divided by the wetted perimeter) and S is the slope in feet per foot.
4. Rating Curve: A rating curve of depth vs. discharge can be provided at the bounding cross sections.

If multiple profiles are run, a different boundary condition can be set for each one.

Example 3: Determine the flow in a river channel with the following characteristics:

- Cross sectional area = 12 SF.
- Wetted Perimeter = 18 FT.
- Manning 'n' value = 0.05
- Slope = 5%

Solution: Use the Manning's Equation: $Q = (1.486/n)AR^{2/3}S^{1/2}$

However, first determine the hydraulic radius. This is equal to the cross sectional area divided by the wetted perimeter. R=12/18=0.67.
Next, convert the slope to feet per foot: 5% = 0.05 feet per foot.
Plug these values into the equation:
$$Q = (1.486/(0.05)(12)(0.67)^{2/3}(0.05)^{1/2} = 61CFS$$

Running the Model:
When running a flood profile there are several decisions that the program allows the engineer to make. These are discussed briefly below:

***Conveyance Calculations*:** The user has the option to sum the wetted perimeter and area between breaks in 'n' values (generally at the edges of the bank) or to sum these values between each of the coordinate points. The conveyance values are then summed to get total left and right bank values. The first option is the default option whereas the second option recreates the old HEC-2 calculations.
Friction Slope Methods: The user can decide between the following four options:

1. Average conveyance (This is the default option).
2. Average friction slope.
3. Geometric mean friction slope.
4. Harmonic mean friction slope.

Setting Tolerances: The program allows the user to modify the default tolerances of the following parameters.

1. Water surface calculation tolerance.
2. Critical depth calculation tolerance.
3. Maximum number of iteration used to try to balance a water surface.
4. Maximum difference tolerance.
5. Flow tolerance factor.
6. Maximum iteration in split flow.
7. Flow tolerance factor in weir split flow.
8. Maximum difference in junction split flow.

Note that increasing any of these tolerances from the default setting could result in computational errors when determining the water surface profile.
Critical Depth Outlet Option (and critical depth computation method): Together these two options allow the user to (i) instruct the program to calculate critical depth and (ii) determine between two different methods of calculating critical depth. These two methods are as follows:

1. The Parabolic Method: This is a comparatively fast method but it is capable of finding only a single minimum value on the energy curve.

2. The Multiple Critical Depth Search is capable of calculating as many as three minimums on the energy curve. This involves a much more cumbersome series of calculations, however, and it can slow down the profile generation. Therefore, this method is generally used only when there is a reason to believe that the program is incorrectly defining critical depth.

HEC-RAS allows the user to specify the types of output data that are presented in the report that is generated. These can include input parameters, basic flood data, bridge and culvert tables, and much more,

Checking the Model:
While the HEC-RAS computer package is a very powerful tool for the analysis of river systems, the final profiles should always be checked for accuracy. Unfortunately, there is often no data against which to check the resulting flood elevations. However, the engineer generally can rely on a few clues when checking the accuracy of a model. Road crossings are often the best place to check for accuracy. For example, if the model shows a county road being inundated by 2 feet during a 100 year storm and the local county engineer's office has no record of this particular road ever being flooded, then the model is most likely not accurately modeling the flood profile. The 100 year flow in the stream reach, the geometry of the bridge or culvert input into the model, or some other input parameters might need to be modified. County and municipal officials, as well as local residents are often very good sources of this type of information.

Additional Features to be Modeled:
As can be seen by the foregoing discussion, HEC-RAS is a powerful analytical tool and can model a wide variety of situation occurring along rivers. In addition to the basic concepts discussed in this course, the program has the capacity to monitor other stream situations including:

1. Modeling flows under river ice conditions.
2. Modeling stream encroachments.
3. Modeling channel modifications.
4. Computing scour at bridges.
5. Modeling tributaries.

All of these situations are beyond the scope of this course.

Some Unusual Situations:
Despite the rather straight-forward presentation of the input data included in the HEC-RAS user's manual, there are a number of real world situations that do not fit very well with the

standard input techniques. These types of situation require that the engineer "think outside the box" somewhat and develop strategies for properly inputting the data. Unusual situations can take almost any form but some that the author has encountered on actual projects include the following:

1. The confluence of two streams occurring within culverts that are several hundred feet upstream and downstream of the inlets or outlets.
2. A dry, historic channel bed running roughly parallel with the stream at a lower elevation than the actual stream channel.
3. An upland area that is inundated with floodwaters from two separate streams during large storm events.

In each of these cases, some engineering judgment was needed to try to properly model the situation being analyzed. In the first instance, a detailed analysis of the culverts led to a "hand calculation" restarting the water surface at the upstream end of the culverts. In the second case, the cross sections were redrawn to prevent the water from "spilling" from the actual to the historic channel. In the third case, separate backwater analyses needed to be run on each stream. The higher elevation was then used as the control flood elevation.

Final Considerations:

HEC-RAS is the basic river analysis system available to engineers who need to determine the flood profile of a river or stream. As can be seen in the discussion in this course, it is a powerful and flexible tool. However, the engineer should never lose site of the fact that it is just a tool and that it is meant to model a real –life situation. All of the input data, underlying assumptions, bridge and culvert methodology, and other features present in the program are meant to be used to approximate a real world stream system. Careful analysis of the input and output are essential to ensure that the results are realistic and accurately portray actual flooding conditions. Sometimes, after reviewing the output against historical flood data, the engineer must make significant changes in the model. Bridges & culverts are key nodes that can have significant impacts on the resulting flood profiles and should be checked carefully. If the HEC-RAS results cannot be made to conform closely to known flood profiles, the user might have to take drastic measures and model a particular structure by hand using the "Hydraulic Charts for Culvert Selection" or other standard references. The more cross checking of the model that is done, the more reliable will be the final output from the program.

