CHAPTER 8

Modeling Gated Spillways, Weirs and Drop Structures

This version of HEC-RAS allows the user to model inline gated spillways, overflow weirs, drop structures, lateral gated spillways, and lateral weirs. HEC-RAS has the ability to model radial gates (often called tainter gates) or vertical lift gates (sluice gates). The spillway crest of the gates can be modeled as either an ogee shape or a broad crested weir shape. In addition to the gate openings, the user can also define a separate uncontrolled overflow weir.

This chapter describes the general modeling guidelines for using the gated spillway and weir capability within HEC-RAS, as well as the hydraulic equations used. Information on modeling drop structures with HEC-RAS is also provided. For information on how to enter gated spillway and weir data, as well as viewing gated spillway and weir results, see Chapter 6 and Chapter 8 of the HEC-RAS User's Manual, respectively.

Contents

General Modeling Guidelines

The gated spillway and weir option within HEC-RAS can be used to model inline (structures across the main stream) or lateral (structures along the side of the stream) weirs, gated spillways, or a combination of both. An example of a dam with a gated spillway and overflow weir is shown in Figure 8.1.

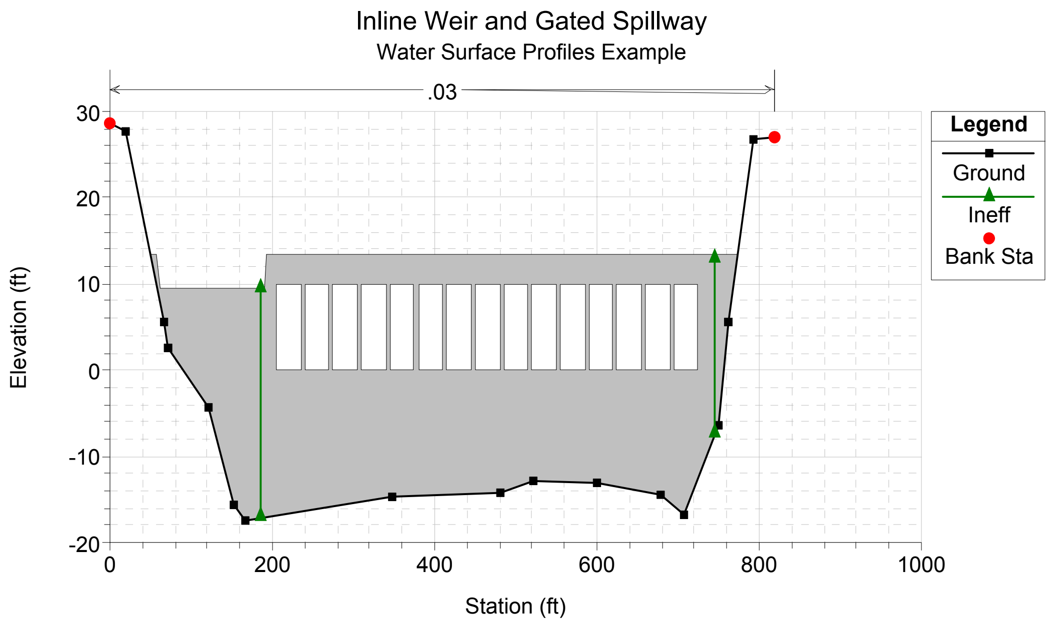

Figure 8.1 Example of Inline Gated Spillway and Weir

In the example shown in Figure 8.1 there are 15 identical gate openings and the entire top of the embankment is specified as an overflow weir.

Gated Spillways within HEC-RAS can be modeled as radial gates (often called tainter gates) or vertical lift gates (sluice gates). The equations used to model the gate openings can handle both submerged and unsubmerged conditions at the inlet and outlet of the gates. If the gates are opened far enough, such that unsubmerged conditions exist at the upstream end, the program automatically switches to a weir flow equation to calculate the hydraulics of the flow. The spillway crest through the gate openings can be specified as either an ogee crest shape or a broad crested weir. The program has the ability to calculate both free flowing and submerged weir flow through the gate openings. Figure 8.2 is a diagram of the two gate types with different spillway crests.

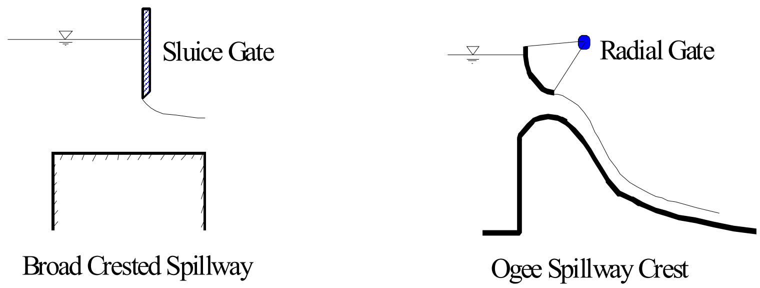

Figure 8.2 Example Sluice and Radial Gates

Up to 10 gate groups can be entered into the program at any one river crossing. Each gate group can have up to 25 identical gate openings. Identical gate openings must be the same gate type; size; elevation; and have identical gate coefficients. If anything about the gates is different, except their physical location across the stream, the gates must be entered as separate gate groups.

The overflow weir capability can be used by itself or in conjunction with the gated spillway option. The overflow weir is entered as a series of station and elevation points across the stream, which allows for complicated weir shapes. The user must specify if the weir is broad crested or an ogee shape. The software has the ability to account for submergence due to the downstream tailwater. Additionally, if the weir has an ogee shaped crest, the program can calculate the appropriate weir coefficient for a given design head. The weir coefficient will automatically be decreased or increased when the actual head is lower or higher than the design head.

Cross Section Locations

The inline weir and gated spillway routines in HEC-RAS require the same cross sections as the bridge and culvert routines. Four cross sections in the vicinity of the hydraulic structure are required for a complete model, two upstream and two downstream. In general, there should always be additional cross sections downstream from any structure (bridge, culvert, weir, etc...), such that the user entered downstream boundary condition does not affect the hydraulics of flow through the structure. In order to simplify the discussion of cross sections around the inline weir and gated spillway structure, only the four cross sections in the vicinity will be discussed. These four cross sections include: one cross section sufficiently downstream such that the flow is fully expanded; one at the downstream end of the structure (representing the tailwater location); one at the upstream end of the structure (representing the headwater location); and one cross section located far enough upstream at the point in which the flow begins to contract. Note, the cross sections that bound the structure represent the channel geometry outside of the embankment. Figure 8.3 illustrates the cross sections required for an inline weir and gated spillway model.

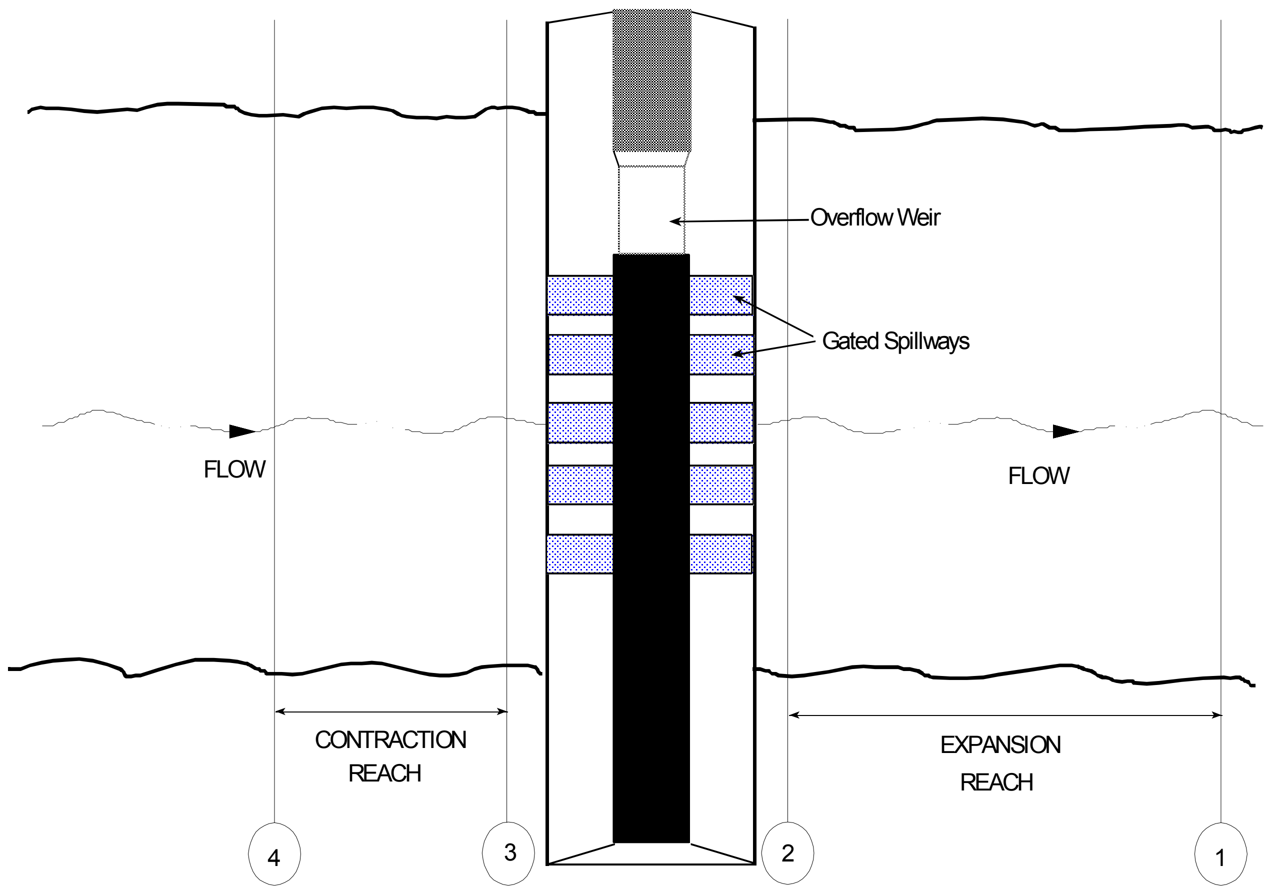

Figure 8.3 Cross Section Layout for Inline Gated Spillways and Weirs

Cross Section 1. Cross Section 1 for a weir and/or gated spillway should be located at a point where flow has fully expanded from its constricted top width caused by the constriction. The entire area of Cross Section 1 is usually considered to be effective in conveying flow.

Cross Section 2. Cross Section 2 is located a short distance downstream from the structure. The computed water surface at this cross section will represent the tailwater elevation of the weir and the gated spillways. This cross section should not include any of the structure or embankment, but represents the physical shape of the channel just downstream of the structure. The shape and location of this cross section is entered separately from the Inline Weir and Gated Spillway data (from the cross section editor).

The HEC-RAS ineffective area option is used to restrict the effective flow area of Cross Section 2 to the flow area around or near the edges of the gated spillways, until flow overtops the overflow weir and/or embankment. The ineffective flow areas are used to represent the correct amount of active flow area just downstream of the structure. Establishing the correct amount of effective flow area is very important in computing an accurate tailwater elevation at Cross Section 2. Because the flow will begin to expand as it exits the gated spillways, the active flow area at Section 2 is generally wider than the width of the gate openings. The width of the active flow area will depend upon how far downstream Cross Section 2 is from the structure. In general, a reasonable assumption would be to assume a 1:1 expansion rate over this short distance. Figure 8.4 illustrates Cross Section 2 of a typical inline weir and gated spillway model. On Figure 8.4, the channel bank locations are indicated by small circles and the stations and elevations of the ineffective flow areas are indicated by triangles.

Cross Sections 1 and 2 are located so as to create a channel reach downstream of the structure in which the HEC-RAS program can accurately compute the friction losses and expansion losses that occur as the flow fully expands.

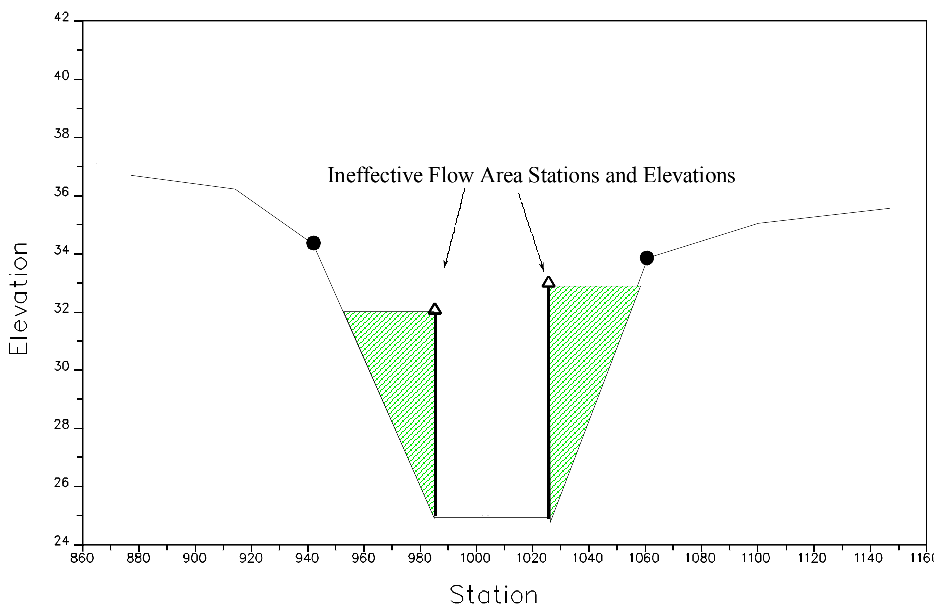

Figure 8.4 Cross Section 2 of Inline Gated Spillway and Weir Model

Cross Section 3. Cross Section 3 of an inline weir and gated spillway model is located a short distance upstream of the embankment, and represents the physical configuration of the upstream channel. The water surface computed at this cross section represents the upstream headwater for the overflow weir and the gated spillways. The software uses a combination of the deck/road embankment data, Cross Section 3, and the gated spillway data, to describe the hydraulic structure and the roadway embankment. The inline weir and gated spillway data is located at a river station between Cross Section 2 and Cross Section 3.

The HEC-RAS ineffective area option is used to restrict the effective flow area of Cross Section 3 until the flow overtops the roadway. The ineffective flow area is used to represent the correct amount of active flow area just upstream of the structure. Because the flow is contracting rapidly as it enters the gate openings, the active flow area at Section 3 is generally wider than the width of the gates. The width of the active flow area will depend upon how far upstream Cross Section 3 is placed from the structure. In general, a reasonable assumption would be to assume a 1:1 contraction rate over this short distance. Figure 8.5 illustrates Cross Section 3 for a typical model, including the embankment profile and the gated spillways. On Figure 8.5, the channel bank locations are indicated by small circles, and the stations and elevations of ineffective areas are indicated by triangles.

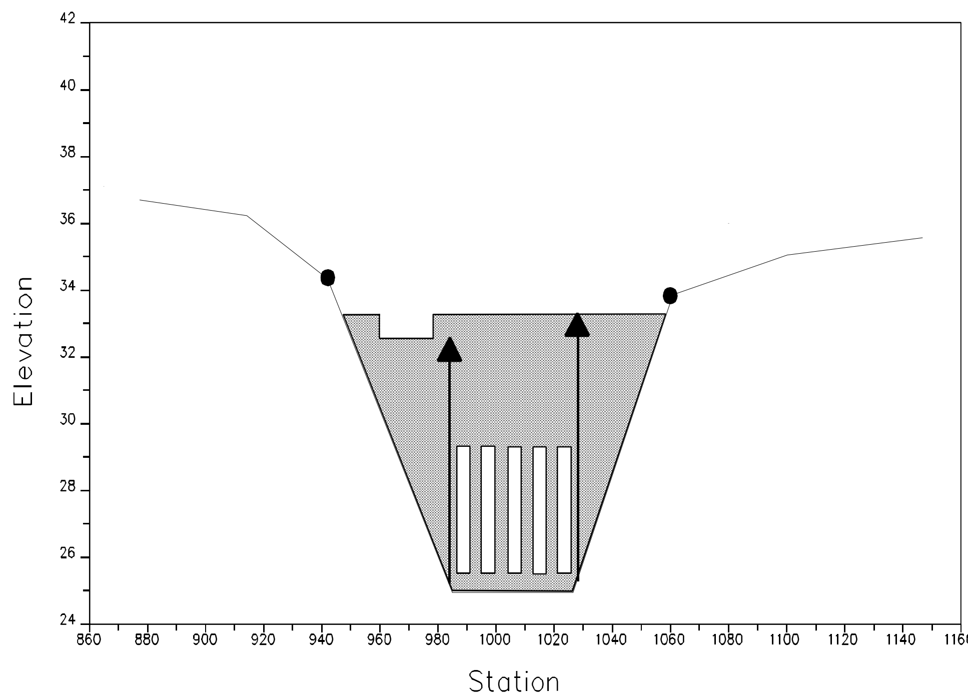

Figure 8.5 Cross Section 3 of Inline Gated Spillway and Weir

Cross Section 4. The final cross section in the inline weir and gated spillway model is located at a point where flow has not yet begun to contract from its unrestrained top width upstream of the structure. This distance is normally determined assuming a one to one contraction of flow. In other words, the average rate at which flow can contract to pass through the gate openings is assumed to be one foot laterally for every one foot traveled in the downstream direction. The entire area of Cross Section 4 is usually considered to be effective in conveying flow.

Expansion and Contraction Coefficients

User-defined coefficients are required to compute head losses due to the contraction and expansion of flows upstream and downstream of an inline weir and gated spillway structure. These losses are computed by multiplying an expansion or contraction coefficient by the absolute difference in velocity head between two cross sections.

If the velocity head increases in the downstream direction, a contraction coefficient is applied. When the velocity head decreases in the downstream direction, an expansion coefficient is used. Recommended values for the expansion and contraction coefficients have been given in Chapter 3 of this manual (table 3.2). As indicated by the tabulated values, the expansion of flow causes more energy loss than the contraction. Also, energy losses increase with the abruptness of the transition.

Hydraulic Computations Through Gated Spillways

As mentioned previously, the program is capable of modeling both radial gates (often called tainter gates) and vertical lift gates (sluice gates). The equations used to model the gate openings can handle both submerged and unsubmerged conditions at the inlet and the outlet of the gates. When the gates are opened to an elevation greater than the upstream water surface elevation, the program automatically switches to modeling the flow through the gates as weir flow. When the upstream water surface is greater than or equal to 1.25 times the height of the gate opening (with respect to the gates spillway crest), the gate flow equations are applied. When the upstream water surface is between 1.0 and 1.25 times the gate opening, the flow is in a zone of transition between weir flow and gate flow. The program computes the upstream head with both equations and then calculates a linear weighted average of the two values (this is an iterative process to obtain the final headwater elevation for a flow in the transition range). When the upstream water surface is equal to or less than 1.0 times the gate opening, then the flow through the gate opening is calculated as weir flow.

Radial Gates

An example radial gate with an ogee spillway crest is shown in Figure 8.6.

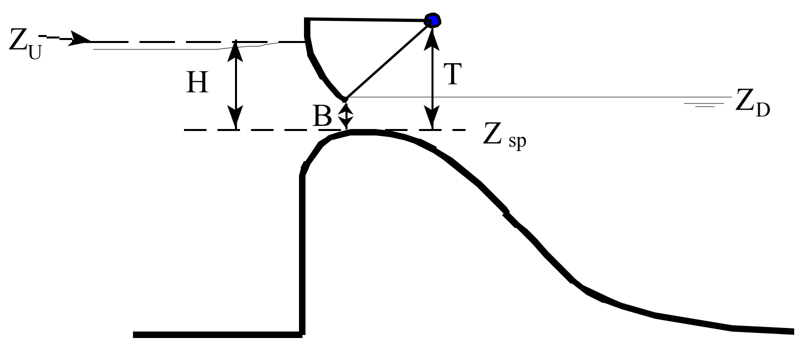

Figure 8.6 Example Radial Gate with an Ogee Spillway Crest

The flow through the gate is considered to be “Free Flow” when the downstream tailwater elevation (Z_D) is not high enough to cause an increase in the upstream headwater elevation for a given flow rate. The equation used for a Radial gate under free flow conditions is as follows:

$$Q = C\sqrt{2g}\,W\,T^{TE}\,B^{BE}\,H^{HE} \tag{8-1}$$

Where: Q = Flow rate in cfs
- C = Discharge coefficient (typically ranges from 0.6 - 0.8)
- W = Width of the gated spillway in feet
- T = Trunnion height (from spillway crest to trunnion pivot point)
- TE = Trunnion height exponent, typically about 0.16 (default 0.0)
- B = Height of gate opening in feet
- BE = Gate opening exponent, typically about 0.72 (default 1.0)
- H = Upstream Energy Head above the spillway crest Z_U - Z_{sp}
- HE = Head exponent, typically about 0.62 (default 0.5)
- Z_U = Elevation of the upstream energy grade line
- Z_D = Elevation of the downstream water surface
- Z_{sp} = Elevation of the spillway crest through the gate

When the downstream tailwater increases to the point at which the gate is no longer flowing freely (downstream submergence is causing a greater upstream headwater for a given flow), the program switches to the following form of the equation:

$$Q = C\sqrt{2g}\,W\,T^{TE}\,B^{BE}\,(3H)^{HE} \tag{8-2}$$

where: H = $Z_U - Z_D$

Submergence begins to occur when the tailwater depth divided by the headwater energy depth above the spillway, is greater than 0.67. Equation 8-2 is used to transition between free flow and fully submerged flow. This transition is set up so the program will gradually change to the fully submerged Orifice equation when the gates reach a submergence of 0.80. The fully submerged Orifice equation is shown below:

$$Q = CA\sqrt{2gH} \tag{8-3}$$

Where: A = Area of the gate opening.

H = $Z_U - Z_D$

Sluice Gate

An example sluice gate with a broad crest is shown in Figure 8.7.

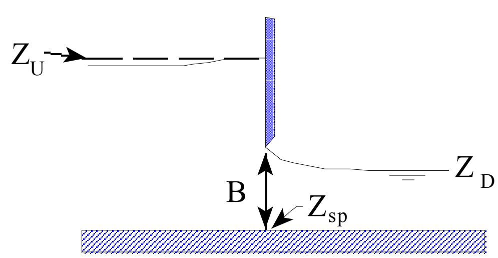

Figure 8.7 Example Sluice Gate with Broad Crested Spillway

The equation for a free flowing sluice gate is as follows:

$$Q = C\,W\,B\sqrt{2gH} \tag{8-4}$$

Where: H = Upstream energy head above the spillway crest ($Z_U - Z_{sp}$)

C = Coefficient of discharge, typically 0.5 to 0.7

When the downstream tailwater increases to the point at which the gate is no longer flowing freely (downstream submergence is causing a greater upstream headwater for a given flow), the program switches to the following form of the equation:

$$Q = C W B \sqrt{2g3H} \tag{8-5}$$

Where: H = Z_U - Z_D

Submergence begins to occur when the tailwater depth above the spillway divided by the headwater energy above the spillway, is greater than 0.67. Equation 8-5 is used to transition between free flow and fully submerged flow. This transition is set up so the program will gradually change to the fully submerged Orifice equation (Equation 8-3) when the gates reach a submergence of 0.80.

Low Flow Through The Gates

When the upstream water surface is equal to or less than the top of the gate opening, the program calculates the flow through the gates as weir flow. An example of low flow through a gated structure is shown in Figure 8.8.

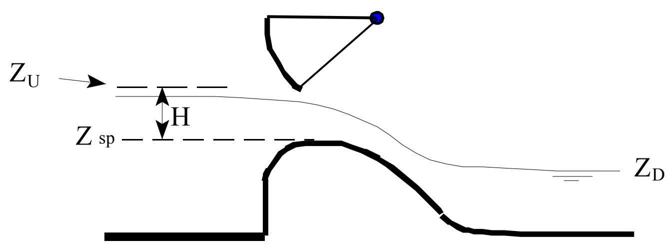

Figure 8.8 Example Radial Gate Under Low Flow Conditions

The standard weir equation used for this calculation is shown below:

$$Q = C L H^{3/2} \tag{8-6}$$

where: C = Weir flow coefficient, typical values will range from 2.6 to 4.0 depending upon the shape of the spillway crest (i.e., broad crested or ogee shaped).

L = Length of the spillway crest.

H = Upstream energy head above the spillway crest.

The user can specify either a broad crested or ogee weir shape for the spillway crest of the gate. If the crest of the spillway is ogee shaped, the weir coefficient will be automatically adjusted when the upstream energy head is higher or lower than a user specified design head. The adjustment is based on the curve shown in Figure 8.9 (Bureau of Reclamation, 1977). The curve provides ratios for the discharge coefficient, based on the ratio of the actual head to the design head of the spillway. In Figure 8.9, H_e is the upstream energy head; H_o is the design head; C_o is the coefficient of discharge at the design head; and C is the coefficient of discharge for an energy head other than the design head.

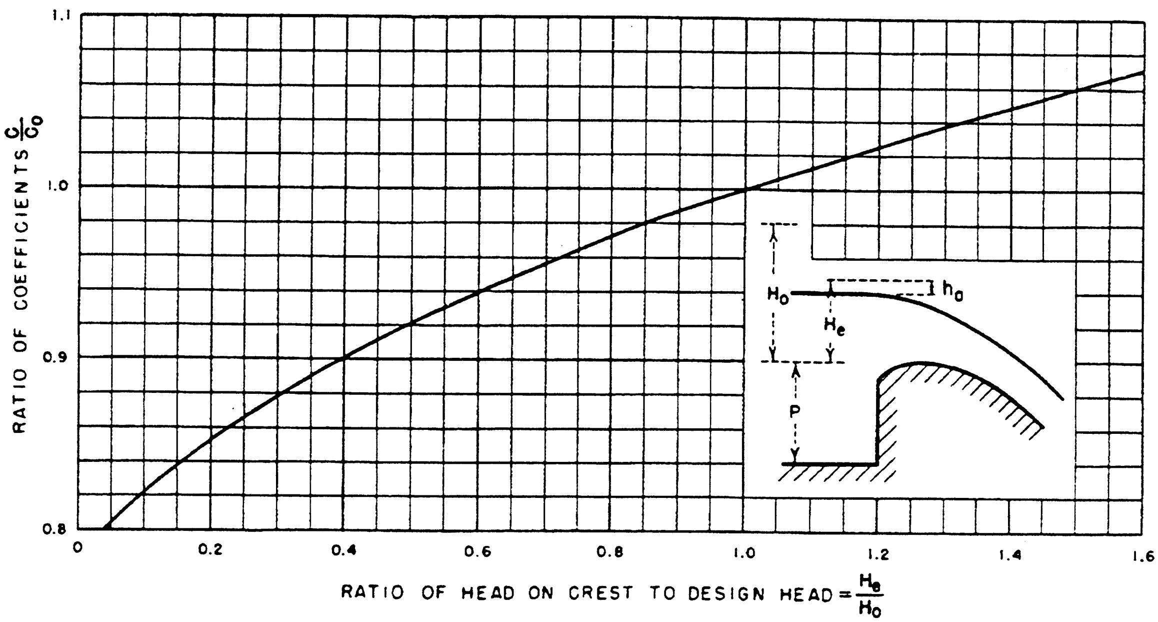

Figure 8.9 Weir Flow Coefficient for Other Than Design Head

The program automatically accounts for submergence on the weir when the tailwater is high enough to slow down the flow. Submergence is defined as the depth of water above the weir on the downstream side divided by the headwater energy depth of water above the weir on the upstream side. As the degree of submergence increases, the program reduces the weir flow coefficient. Submergence corrections are based on a trapezoidal (broad crested) or ogee shaped weir.

Uncontrolled Overflow Weirs

In addition to the gate openings, the user can define an uncontrolled overflow weir at the same river crossing. The weir could represent an emergency spillway or the entire top of the structure and embankment. Weir flow is computed using the standard weir equation (equation 8-6). The uncontrolled overflow weir can be specified as either a broad crested or ogee shaped weir.

If the weir is ogee shaped, the program will allow for fluctuations in the discharge coefficient to account for upstream energy heads that are either higher or lower than the design head (figure 8.9). The program will automatically account for any submergence of the downstream tailwater on the weir, and reduce the flow over the weir. The modeler is referred to Chapter 5 of the Hydraulic Reference Manual for additional discussions concerning uncontrolled overflow weirs, including submergence criteria and selection of weir coefficients.

Modeling Lateral Weirs and Gated Spillways

HEC-RAS has the ability to model lateral weirs and gated spillway structures. The modeler can insert a lateral weir only, or a separate gated spillway structure, or a combination of the two. An example diagram of a lateral weir is shown in Figure 8.10.

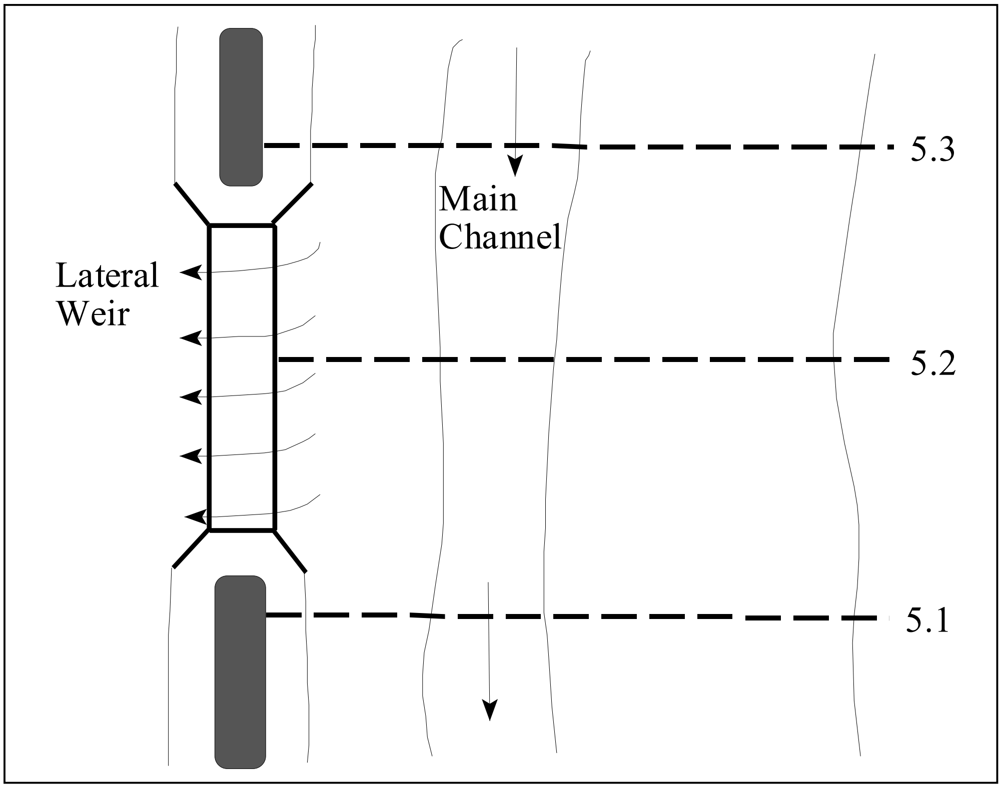

Figure 8.10 Plan View of an Example Lateral Weir

At a minimum there must be a cross section upstream of, and a cross section downstream of the lateral weir/gated spillway. The upstream cross section can either be right at the beginning of the structure, or it can be a short distance upstream. The downstream cross section can be right at the downstream end of the structure or it can be a short distance downstream. The user can have any number of additional cross sections in the middle of the structure.

If there are gated openings in the structure, the hydraulic computations for lateral gated spillways are exactly the same as those described previously for inline gated spillways. The only difference is that the headwater energy is computed separately for each gate, based on its centerline location along the stream. The headwater energy for each gate is interpolated linearly between computed points at each cross section.

An example lateral weir and gated spillway structure is shown in Figure 8.11 as a profile view.

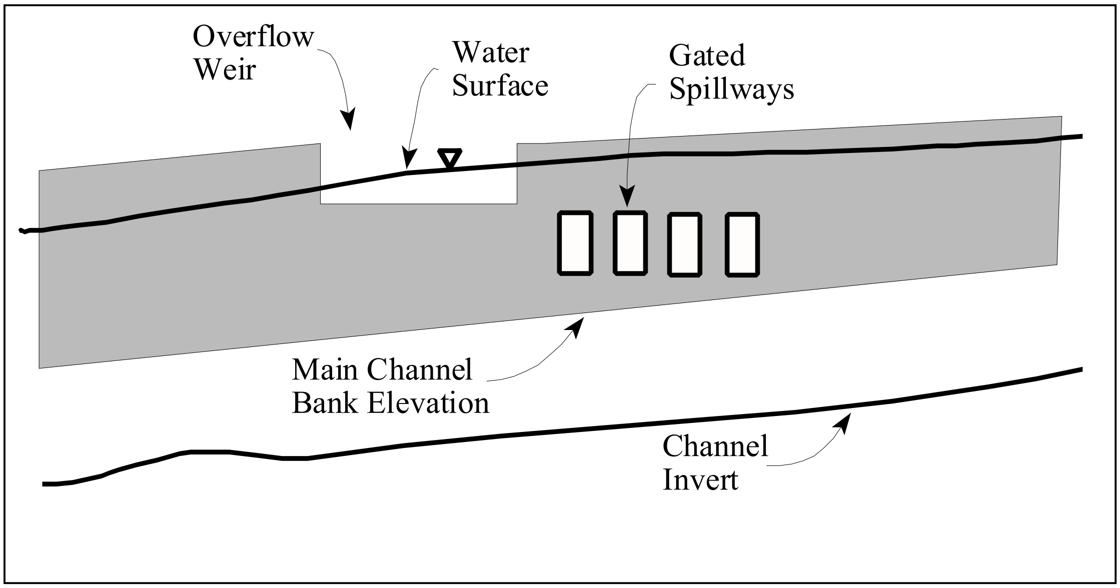

Figure 8.11 Example Lateral Weir and Gated Spillway

As shown in Figure 8.11, the water surface across the weir has a slope to it. Additionally, the weir itself could be on a slope. Because of this, an equation for weir flow with a sloping water surface and weir sill had to be derived. Shown in Figure 8.12 is a sloping weir segment with a sloping water surface. The equation for a sloping line representing the water surface and the weir segment are shown. The constants a_{ws} and a_w represent the slope of the water surface and the weir segment, respectively, while the variable C_{ws} and C_w are constants representing the initial elevations.

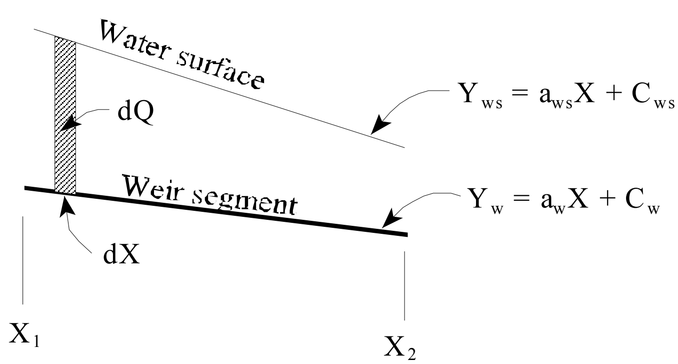

Figure 8.12 Sloping Weir Segment and Water Surface

The standard weir equation (8-6) assumes that the weir is parallel with the water surface (i.e. that the depth of water is constant from one end of the weir segment to the other). The following general equation is derived for a sloping weir and water surface by integrating the standard weir equation:

$$dQ = C(y_{ws} - y_w)^{3/2}\, dx \tag{8-7}$$

$$dQ = C(a_{ws}\, x + C_{ws} - a_w\, x - C_w)^{3/2}\, dx \tag{8-8}$$

$$dQ = C((a_{ws} - a_w)x + C_{ws} - C_w)^{3/2}\, dx \tag{8-9}$$

Assuming: $a_1 = a_{ws} - a_w$ and $C_1 = C_{ws} - C_w$

$$\int_{x_1}^{x_2} dQ = C\int_{x_1}^{x_2} (a_1 x + C_1)^{3/2}\, dx = \frac{2C}{5a_1}(a_1\, x + C_1)^{5/2}\ \Big]_{x_1}^{x_2} \tag{8-10}$$

$$Q_{x_1 - x_2} = \frac{2C}{5a_1}((a_1 x_2 + C_1)^{5/2} - (a_1 x_1 + C_1)^{5/2}) \tag{8-11}$$

The above equation is valid as long as a_1 is not zero. When a_1 is zero, this implies that the water surface and the weir segment are parallel. When this is true, the original weir equation (equation 8-6) is used.

Within HEC-RAS, flow over a lateral weir can be computed from either the energy grade line or the water surface elevation. The standard weir equation is derived with the upstream energy head being based on the distance from the weir sill to the upstream energy gradeline. The energy gradeline is the default for a lateral weir as well. However, the user has the option of instructing the program to use the water surface elevation when computing the head term of the weir equation. This would be most appropriate when the weir is located close to the main channel. In this situation the energy due to the velocity head is in the downstream direction, and not over the top of the lateral weir. Therefore, the computation of the energy head over the lateral weir is best depicted by using the water surface of the flow in the channel.

The predecessor to HEC-RAS (HEC-2 program) used the water surface elevation as the default for lateral weir calculations. This is an important point to remember when comparing results between HEC-RAS and HEC-2. However, both programs allow the user to select either the energy gradeline or the water surface elevation for this calculation.

Drop Structures

Drop structures can be modeled with the inline weir option or as a series of cross sections. If you are just interested in getting the water surface upstream and downstream of the drop structure, then the inline weir option would probably be the most appropriate (as described in a previous section of this chapter). However, if you want to compute a more detailed profile upstream of and through the drop, then you will need to model it as a series of cross sections.

When modeling a drop structure as a series of cross sections, the most important thing is to have enough cross sections at the correct locations. Cross sections need to be closely spaced where the water surface and velocity is changing rapidly (i.e. just upstream and downstream of the drop). An example of a drop structure is shown in Figure 8.13.

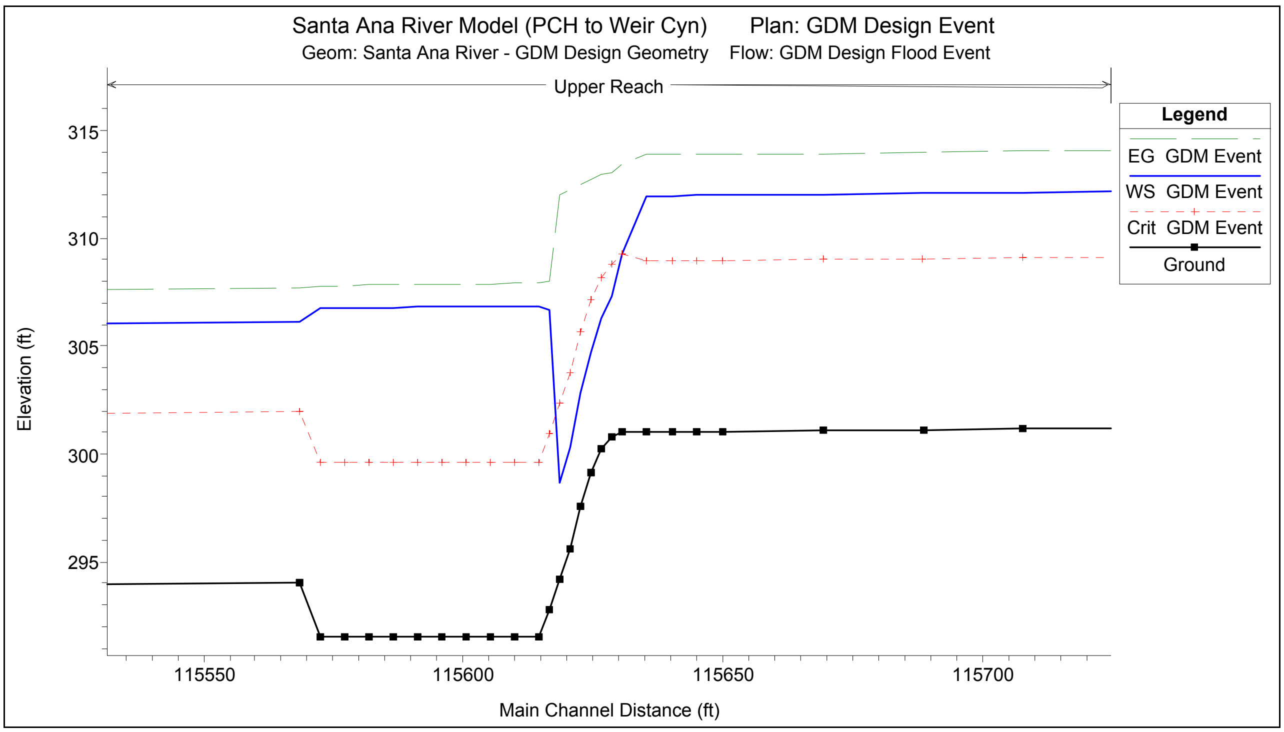

Figure 8.13 Drop Structure Modeled With Cross Sections

As shown in Figure 8.13, the spacing between cross sections should decrease as you get closer to the drop structure (cross sections are located at each square shown on the ground profile). Additionally, if the drop itself is on a slope, then additional cross sections should be placed along the sloping drop in order to model the transition from subcritical to supercritical flow. Several cross sections should also be placed in the stilling basin (location of energy dissipaters) in order to correctly locate where the hydraulic jump will occur (i.e. the hydraulic jump could occur on the slope of the drop, or it may occur inside of the stilling basin). Manning's n values should be increased inside of the stilling basin to represent the increased roughness do to the energy dissipater blocks.

In order to evaluate this method of modeling drop structures, a comparison was made between a physical model study and an HEC-RAS model of the drop structure. During the design phase of improvements to the Santa Ana river, the Waterways Experiment Station (WES) was contracted to study the drop structures and make recommendations. The results of this study were reported in General Design for Replacement of or Modifications to the Lower Santa Ana River Drop Structures, Orange County, California (Technical Report HL-94-4, April 1994, USACE). Over 50 different designs were tested in 1:25 scale flume models and 1:40 scale full width models. The designs evaluated existing structures, modifying original structures and replacing them with entirely new designs. The drop structure design used in the Santa Ana River is similar to one referred to as Type 10 in the report. A HEC-RAS model was developed to model the Type 10 drop structure and the model results were compared to the flume results.

The geometry for the HEC-RAS model was developed from the following design diagram in the WES report.

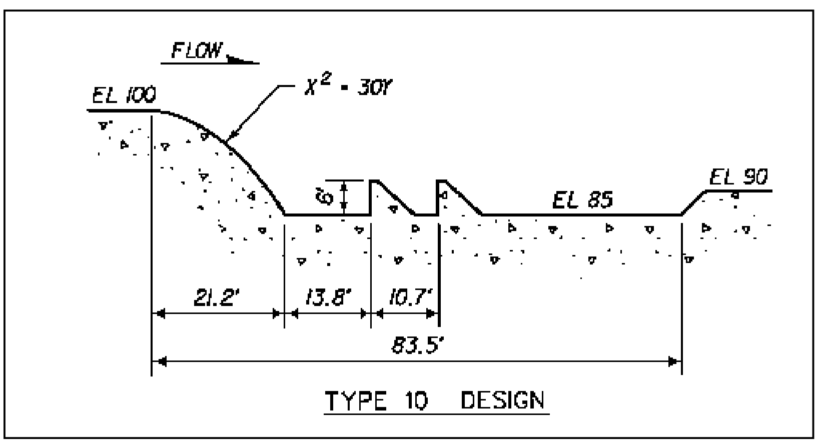

Figure 8.14 WES Report Plate 13.

The total reach in the model was 350 feet, 150 upstream of the crest of the drop structure and 200 feet below the crest. The cross sections were rectangular, with the following spacing used in the HEC-RAS model:

Location	Reach Lengths
Upstream of Drop structure:	10 feet
Over the drop:	2 feet
Inside the stilling basin:	10 feet
Downstream of Structure:	10 feet

The expansion and contraction coefficients were set to 0.3 and 0.1 respectively. Two Manning's n values were used in the HEC-RAS model of the flume. Inside the stilling basin where the bottom elevation was 85 feet, the Manning's n values were set to 0.05. In all other cross sections the Manning's n values were set to 0.03. The higher n value was used in the stilling basin to account for the additional energy loss due to the rows of baffles that exist in the flume but were not added into the cross sections data of HEC-RAS.

The original data from the flume experiments were obtained from the Waterways Experiment Station, and entered in HEC-RAS as observed data. The results of the HEC-RAS model are compared in profile to the observed water surface elevations in the flume study in Figure 8.15. These results show that HEC-RAS was able to adequately model the drop structures, both upstream and downstream of the crest.

Some differences occur right at the crest and through the hydraulic jump. The differences at the crest are due to the fact that the energy equation will always show the flow passing through critical depth at the top of the crest. Whereas, in the field it has been shown that the flow passes through critical depth at a distance upstream of 3-4 times critical depth. However, as shown in Figure 8.15, a short distance upstream of the crest the HEC-RAS program converges to the same depth as the observed data. Correctly obtaining the maximum upstream water surface in the most important part of modeling the drop structure.

Downstream of the drop, the flow is supercritical and then goes through a hydraulic jump. The flume data shows the jump occurring over a distance of 50 to 60 feet with a lot of turbulence. The HEC-RAS model cannot predict how long of a distance it will take for the jump to occur, but it can predict where the jump will begin. The HEC-RAS model will always show the jump occurring between two adjacent cross sections. The HEC-RAS model shows the higher water surface inside of the stilling basin and then going down below the stilling basin. The model shows all of this as a fairly smooth transition, whereas it is actually a turbulent transition with the water surface bouncing up and down. In general, the results from the HEC-RAS model are very good at predicting the stages upstream, inside, and downstream of the drop structure.

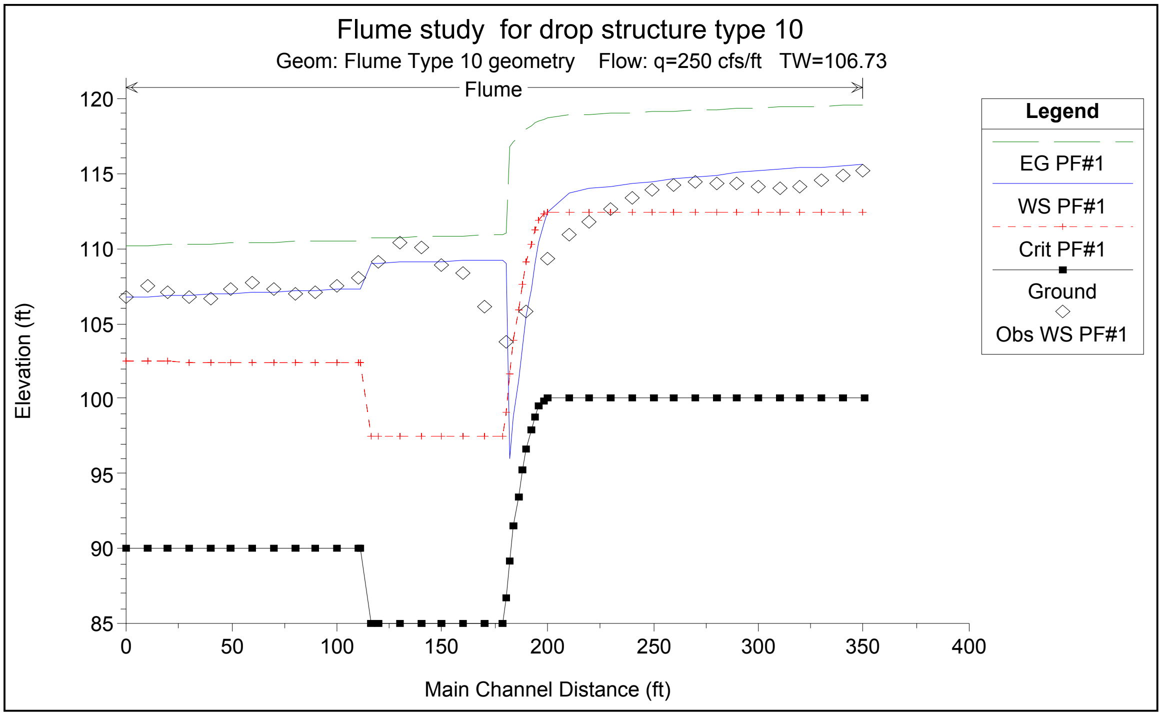

Figure 8.15 Comparison Between Flume Data and HEC-RAS For a Drop Structure