C H A P T E R   16

Advanced Features for Unsteady Flow Routing

HEC-RAS has several advanced features that can be used when modeling complex unsteady flow situations. These features include mixed flow regime capabilities (subcritical, supercritical, hydraulic jumps, and draw downs); the ability to perform a dam break analysis; levee overtopping and breaching; and hinge pool calculations for navigation dams.

Content:

- Mixed Flow Regime
- Dam Break Analysis
- Levee Breaching and Overtopping
- Pump Stations
- Navigation Dams

Mixed Flow Regime

Modeling mixed flow regime (subcritical, supercritical, hydraulic jumps, and draw downs) is quite complicated with an unsteady flow model. In general, most unsteady flow solution algorithms become unstable when the flow passes through critical depth. The solution of the unsteady flow equations is accomplished by calculating derivatives (changes in depth and velocity with respect to time and space) in order to solve the equations. When the flow passes through critical depth, the derivatives become very large and begin to cause oscillations in the solution. These oscillations tend to grow larger until the program goes completely unstable.

In order to solve the stability problem for a mixed flow regime system, Dr. Danny Fread (Fread, 1986) developed a methodology called the "Local Partial Inertia Technique." The LPI method has been adapted to HEC-RAS as an option for solving mixed flow regime problems when using the unsteady flow analysis portion of HEC-RAS. This methodology applies a reduction factor to the two inertia terms in the momentum equation as the Froude number goes towards 1.0. The modified momentum equation is show below:

$$\sigma \left[\frac{\partial Q}{\partial t} + \frac{\partial \left(\frac{\beta Q^2}{A} \right)}{\partial x} \right] + gA \left(\frac{\partial h}{\partial x} + S_f \right) = 0 \tag{16-1}$$

and

$$\sigma = F_T - F_r^m \quad (F_r \le F_T \; ; m \ge 1)$$
$$\sigma = 0 \quad\quad\quad (F_r > F_T)$$

where:
σ = LPI factor to multiply by inertial terms.
F_T = Froude number threshold at which factor is set to zero. This value should range from 1.0 to 2.0 (default is 1.0)
F_r = Froude number.
m = Exponent of equation, which changes shape of the curve. This exponent can range between 1 and 128 (default value is 10).
h = Water surface elevation
S_f = Friction slope
Q = Flow rate (discharge)
A = Active cross sectional area
g = Gravitational force

The default values for the equation are $F_T = 1.0$ and m = 10. When the Froude number is greater than the threshold value, the factor is set to zero. The user can change both the Froude number threshold and the exponent. As you increase the value of both the threshold and the exponent, you decrease stability but increase accuracy. As you decrease the value of the threshold and/or the exponent, you increase stability but decrease accuracy. To change either the threshold or the exponent, select **Mixed Flow Options** from the **Options** menu of the Unsteady Flow Analysis window. When this option is selected, the mixed flow regime options window will appear as shown in Figure 16.1.

As shown in Figure 16.1, the graphic displays what the magnitude of the LPI factor will be for a give Froude number and a given exponent m. Each curve on the graph represents an equation with a threshold of 1.0 (F_T) and a different exponent (m).
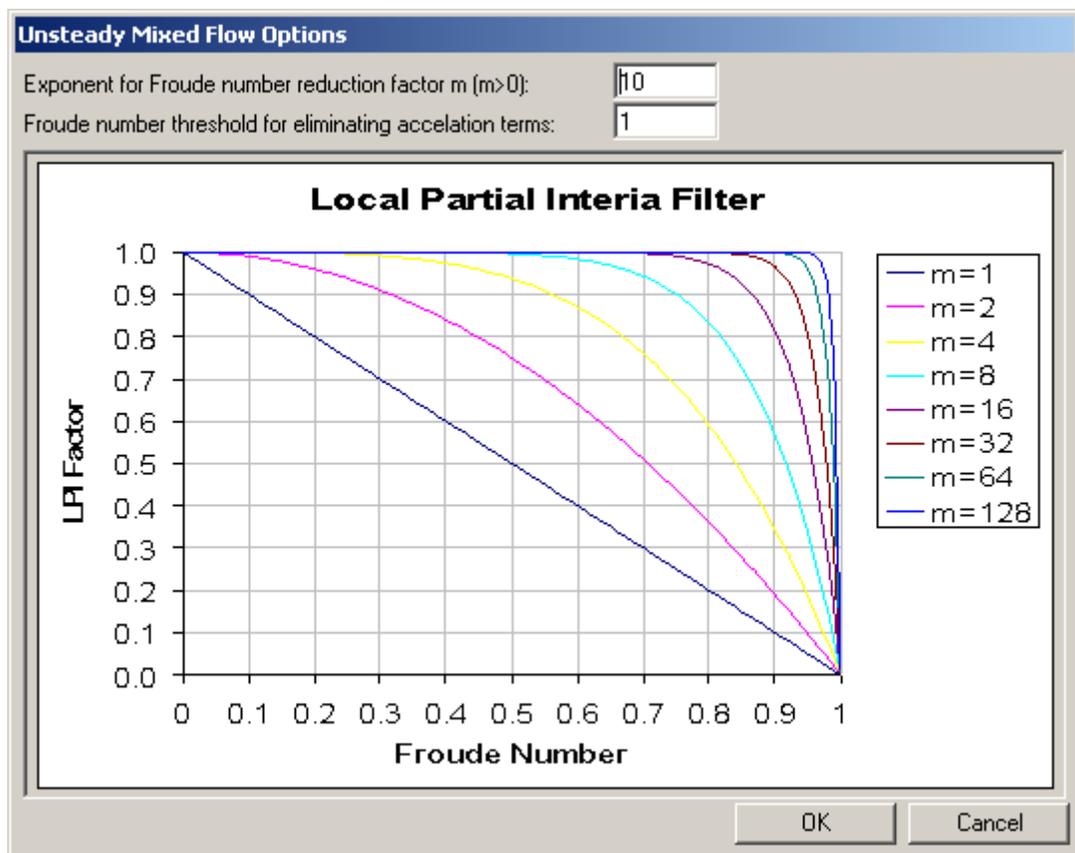
**Figure 16.1 Mixed Flow Options Window**

By default, the mixed flow regime option is not turned on. To turn this option on you simple check the **Mixed Flow Regime** box, which is contained within the computational settings area of the Unsteady Flow Analysis window. This window and option is shown in the Figure below.
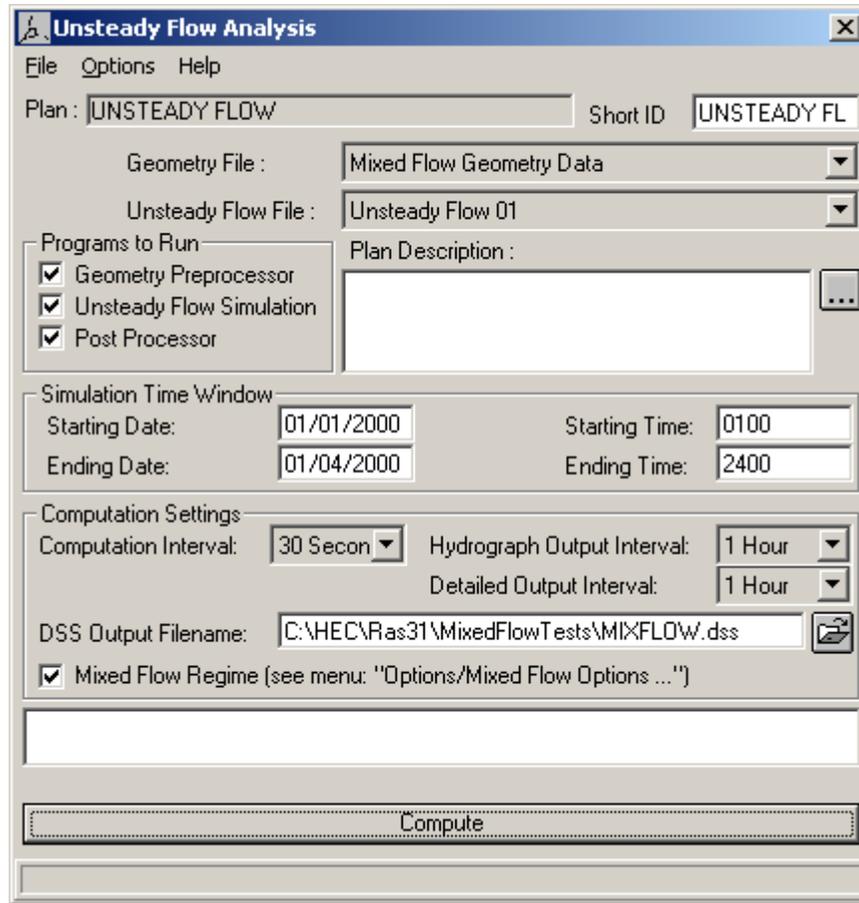
Figure 16.2 Unsteady Flow Analysis Window With Mixed Flow Regime Option Turned On.

In general, if you are working on a river system that is completely subcritical flow, you should not turn on this option. If you have a system that is mostly subcritical flow, with only a few areas that pass through critical depth, then this option can be very useful for solving the stability problems. However, there may be other options for modeling the areas that pass through critical depth. If you have some areas that are drops in the bed, and flow passes through critical depth over the drop, but is subcritical just downstream of the drop. This would be a good location to model the drop as an inline weir within HEC-RAS. By modeling the drop as an inline weir, the program is not modeling the passing through critical depth with the momentum equation, it is simply getting an upstream head water for a given flow from the weir equation. If you are modeling a system that has several areas that pass through critical depth, go supercritical, and go through hydraulic jumps, then the mixed flow methodology will be the only way to get the model to work for unsteady flow.

A profile plot of a mixed flow regime problem is shown in Figure 16.3. This example was run with the unsteady flow simulation capability within HEC-RAS, using the mixed flow regime option. The example shows a steep reach flowing supercritical, which then transitions into a mild reach. A hydraulic jump occurs on the mild reach. The mild reach then transitions back to a steep reach, such that the flow goes from subcritical to supercritical. Because of a high downstream boundary condition (for example backwater from a lake), the flow then goes from supercritical to subcritical though another hydraulic jump.
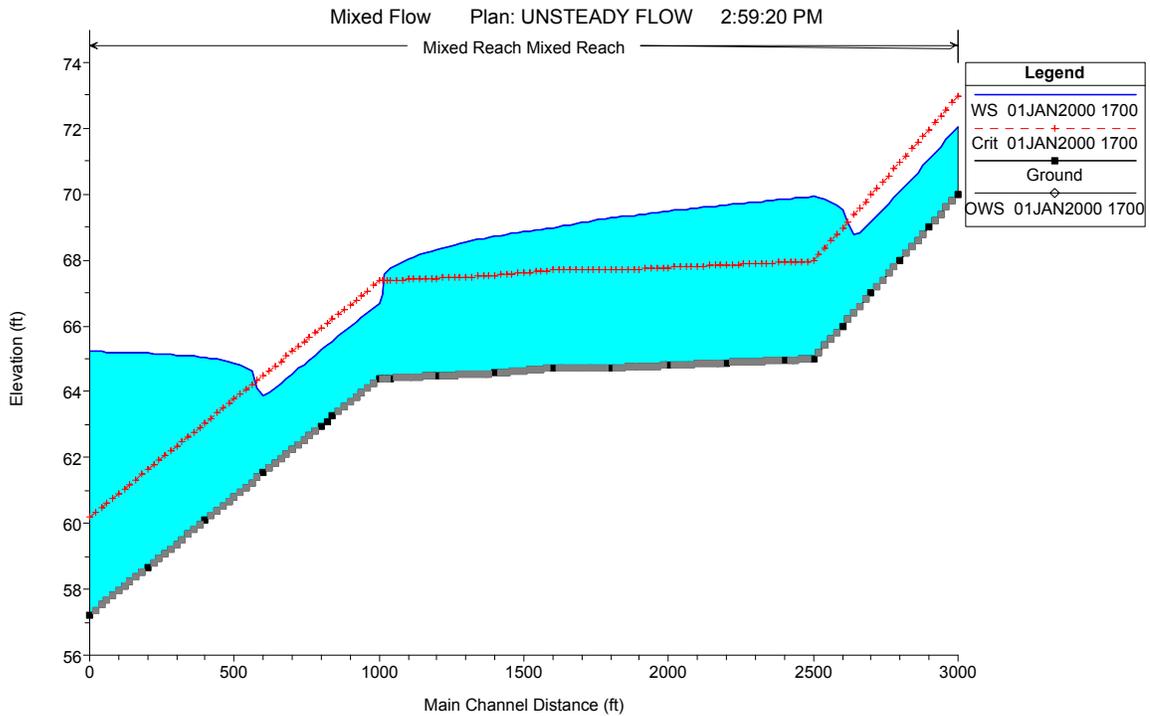
Figure 16.3 Example Mixed Flow Regime Run With Unsteady Flow Routing

Dam Break Analysis

The failure of several dams in this country (Buffalo Creek, West Virginia 1972; Teton dam, Idaho1976; Laural Run Dam and Sandy Run Dam, Pennsylvania 1977; and Kelly Barnes Dam, Georgia 1977), as well as many others, has led our nation to take a strong look at dam safety. One aspect of dam safety is to answer the question, "What will happen if the dam were to fail?" The ability to evaluate the results of a dam failure has been added into the HEC-RAS software.

HEC-RAS can be used to model both overtopping as well as piping failure breaches for earthen dams. Additionally, the more instantaneous type of failures of concrete dams (generally occurring from earthquakes) can also be modeled. The resulting flood wave is routed downstream using the unsteady flow equations. Inundation mapping of the resulting flood can be done with the HEC-GeoRAS program (companion product to HEC-RAS) when GIS data (terrain data) are available.

Dams are modeled within HEC-RAS by using the Inline Structure editor. The Inline Structure editor allows the user to put in an embankment, define overflow spillways and weirs, and gated openings (radial and sluice gates). Gated openings can be controlled with a time series of gate openings or using the elevation control gate operation feature in HEC-RAS. For more information on modeling inline hydraulic structures within HEC-RAS, please review Chapter 6 of this manual (Entering and Editing Geometric Data).

An example of using the Inline Structure feature to model a dam is shown in Figure 16.4. As shown in the Figure, the user enters the embankment and overflow spillway as one piece using the Weir/Embankment editor. The embankment is shown as the gray filled in area above the ground. The overflow spillway is the rectangular notch on the upper left hand side of the embankment. The main outlet works consist of two rectangular gates, which are entered through the gate editor. The gates are shown towards the bottom of the embankment in this example.
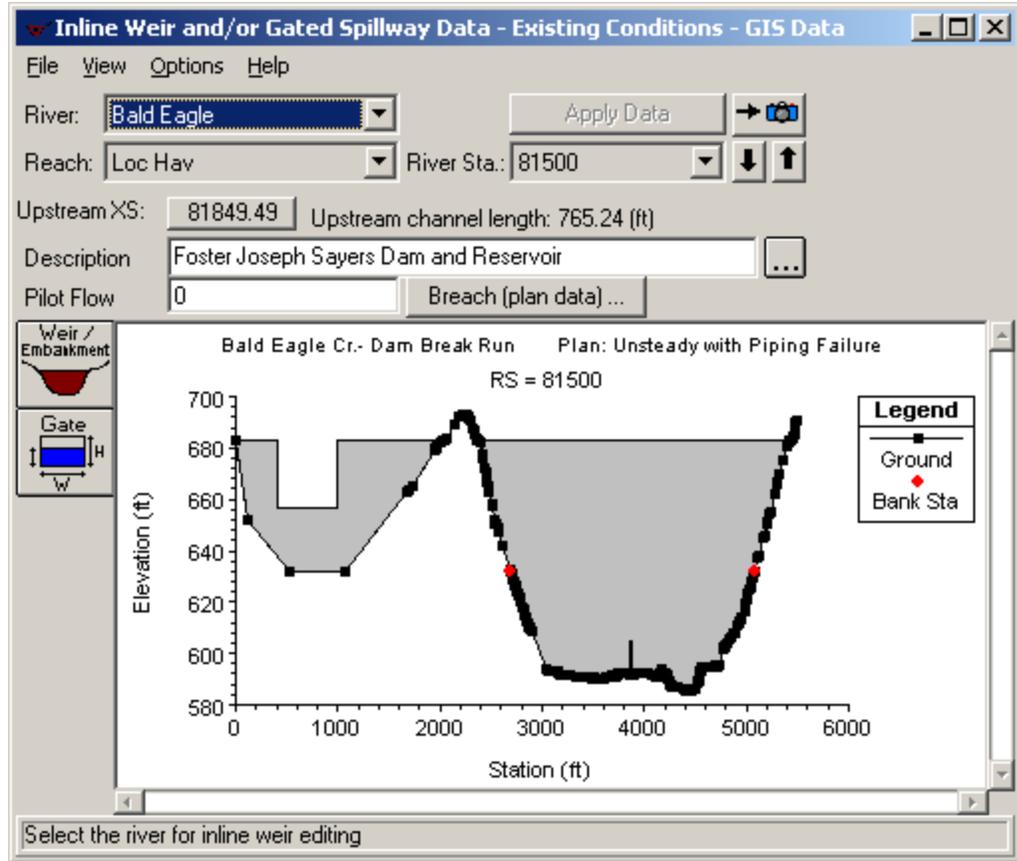
Figure 16.4 Inline Structure Editor With Example Dam Shown

Entering Dam Break Data

Entering dam breach information is accomplished by pressing the button labeled **Breach (Plan Data)**. The breach information is stored as part of the current Plan. This was done to facilitate evaluating dam and levee breaching in a real time river forecasting mode. By putting the breach information in the Plan file, the geometric pre-processor does not have to be run again, thus saving computation time during forecasting. The user can also get to the dam breach information by selecting **Dam Breach (Inline Structure)** from the Options menu of the Unsteady Flow Analysis window. Once the Breach button is pressed, the Dam Breach window will appear as shown in Figure 16.5.
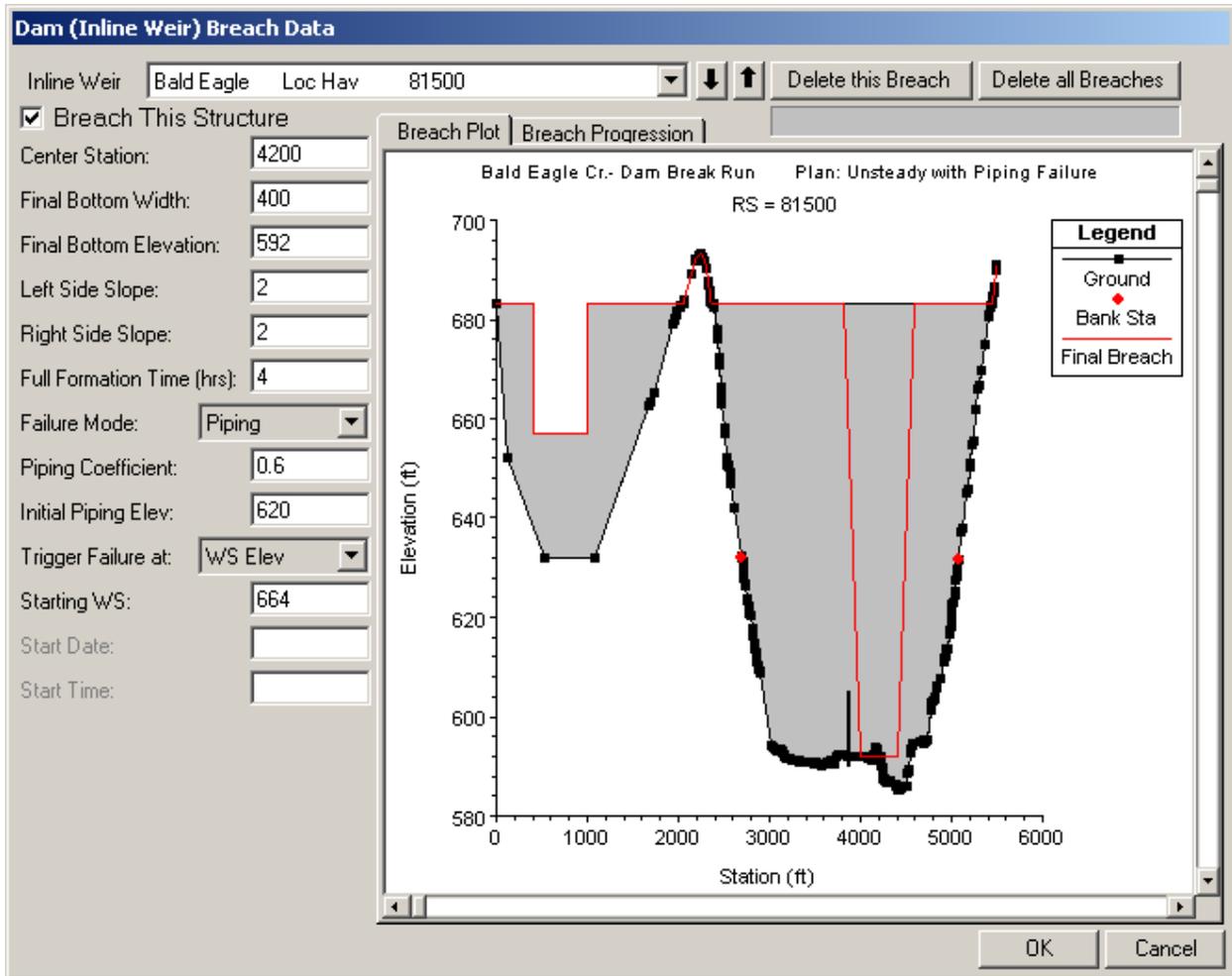
Figure 16.5 Dam Breach Data Editor With Example Dam

The data required to perform a dam breach analysis is as follows:

Inline Structure. This field is used to select the particular inline structure that you want to perform a breach analysis on. The user can enter breach data and perform a breach for more than one dam within the same model.

Delete This Breach. This button is used to clear all of the dam breach information for the currently opened inline structure.

Delete All Breaches. This button is used to delete the dam breach information for all of the inline structures in the model.

Breach This Structure. This check box is used to turn the breaching option on and off without getting rid of the breach data. This box must be checked in order for the software to actually perform the dam breach. When this box is not checked, no breaching will be performed on this structure.

Center Station. This field is used to enter the cross section stationing of the centerline of the breach. The stationing is based on the inline structure that is shown in the graphic.

Final Bottom Width. This field is used to enter the bottom width of the breach when it has reached its maximum size.

Final Bottom Elevation. This field is used to enter the bottom elevation of the breach when it has reached its maximum size.

Left Side Slope. This field is used to enter the left side slope for the trapezoid that will represent the final breach shape. If a zero is entered for both side slopes, the breach will then be rectangular. Side slopes are entered in values representing the horizontal to vertical ratio. For example, a value of 2 represents 2 feet horizontally for every 1 foot vertically.

Right Side Slope. This field is used to enter the right side slope for the trapezoid that will represent the final breach shape. If a zero is entered for both side slopes, the breach will then be rectangular. Side slopes are entered in values representing the horizontal to vertical ratio. For example, a value of 2 represents 2 feet horizontally for every 1 foot vertically.

Full Formation Time (hrs). This field is used to enter the time required for the breach to form. It represents the time from the initiation of the breach until the breach has reached its full size. The user should be very careful in selecting this time. If you are using a linear breach progression rate, you may want to limit this time to when the breach really begins to significantly erode and up to when the major portion of the breach is formed. More information on the breach full formation time will be given later in this chapter.

Failure Mode. This selection box contains two options for the failure mode of the breach, a **Piping** failure or an **Overtopping** failure. The overtopping failure mode should be selected when the water surface overtops the entire dam and erodes its way back through the embankment, or when flow going over the emergency spillway causes erosion that also works its way back through the embankment. The Piping failure mode should be selected when the dam fails due to seepage through the dam, which causes erosion, which in turn causes more flow to go through the dam, which causes even more erosion. A piping failure will grow slowly at first, but tends to pick up speed as the area of the opening begins to enlarge. At some point during the breach, the embankment above the breach will begin to sluff, at which time a large mass wasting of the embankment will occur.

Piping Coefficient. This field is only used if the Piping failure mode has been selected. The user enters an orifice coefficient into this field. The orifice equation is used to calculate the flow through the breach opening while it is acting in a piping flow manner. Once the embankment above the opening sluffs, and the water is open to the atmosphere, the program transitions to a weir equation for computing the breach flow.

Initial Piping Elev. This field is used to enter the elevation of the center of the piping failure when it first begins to occur.

Trigger Failure At. This field is used to enter the mode in which the breach initiation will be triggered. There are three options available within HEC-RAS for initiating the start of the breach: a water surface elevation (**WS Elev**), a specific instance in time (**Set Time**), and a combination of exceeding a water surface elevation for a user specified duration (**WS Elev + Duration**). With the third option (WS Elev + Duration) the user enters a threshold water surface elevation to start monitoring the location. A duration is also entered. If the water surface remains above the threshold value for the user entered duration, then the breach is initiated. Additionally the user can enter a water surface elevation labeled "**Immediate Initiation WS**." If the water surface elevation gets up to or beyond this elevation, the breach is immediately initiated.

Starting WS. This field is only used if the user has selected a trigger failure mode of water surface elevation (**WS Elev**). The user enters a water surface elevation into this field. The water surface represents the elevation at which the breach will begin to occur, once the water behind the dam has reached that elevation.

Start Date. This field is only used if the user has selected the **Set Time** option for the failure trigger mode. The user enters the date at which the breach will begin to occur. The time of the breach initiation is entered into the next field. The date should be entered in a month/day/year format (ex. 05/23/2002).

Start Time. This field is used to enter a starting time to initiate the breach. The time is entered as a military time (ex. 1800 for 6:00 p.m.).

Breach Plot. When this tab is selected, a plot of the inline structure will show up in the graphic window. The plot will show the proposed breach maximum size and location in a red color.

Breach Progression. When this tab is selected a table will appear in the graphic display window. The table is used to enter a user defined progression curve for the formation of the breach. This is an optional feature. If no curve is entered, the program automatically uses a linear breach progression rate. This means that the dimensions of the breach will grow in a linear manner during the time entered as the full breach formation time. Optionally, the user can enter a curve to represent the breach formation as it will occur during the breach development time. The curve is entered as Time Fraction vs. Breach Fraction. The Time Fraction is the decimal percentage of the full breach formation time. The breach fraction is the decimal percentage of the breach size. Both factors are entered as numbers between zero and one. An example of a user entered nonlinear breach progression rate is shown in Figure 16.6.
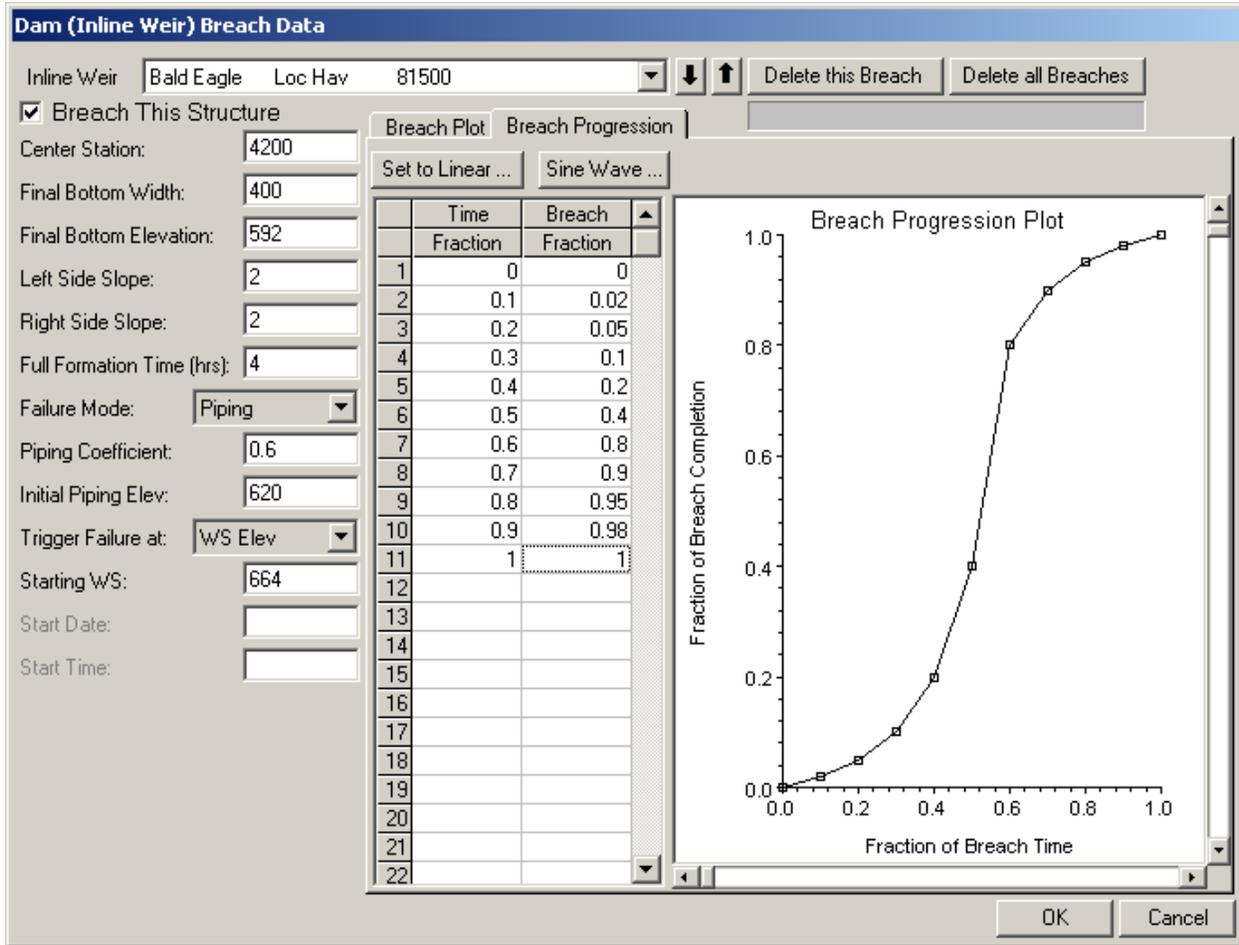
Figure 16.6 Dam Breach Editor With Nonlinear Breach Progression

Once all of the Dam Breach data are entered, press the **OK** button to have the data accepted. However, the data is not saved to the hard disk at this point, you must save the currently opened plan in order for the breach information to be save to the hard disk.

Estimating Dam Break Parameters

The key parameters that must be estimated in any dam breaching analysis are the breach formation time and the maximum size of the breach opening. Several researchers have developed regression equations to estimate breach sizes and times from historical dam breach information. Additionally a few researchers have tried to develop computer models to simulate the physical breach process. The bulk of the research in this area has been summarized in a 1998 publication entitled "Prediction of Embankment Dam Breach Parameters", by Tony L. Wahl of the U.S. Bureau of Reclamation. This publication documents the data from most of the historical dam breaches that

have occurred in the world, as well as describing the equations and modeling approaches developed for predicting the dam breach parameters.

For the HEC-RAS software, the user must estimate the maximum breach dimensions and breach formation time outside of the program. Because the breaching process is complex, it is suggested that the modeler try to come up with several estimates of the breach parameters, and then put together a matrix of potential breach sizes and times. One example would be to use two different sets of regression equations and one of the breach simulation models to estimate the breach parameters. In several studies performed at HEC we have used both the Froelich (1995) and the MacDonald\Langridge-Monopolis (1984) regression equations, as well as the BREACH model by Dr. Danny Fread (Fread, 1988). All three methods give different answers for the breach dimensions, as well as the time for the breach to form. One could simply run each of these estimates as a separate trial within HEC-RAS, or a matrix could be formed by trying all three breach times with each of the breach sizes, thus ending up with 9 different runs. Either way, it is always good to test the sensitivity of the breaching parameters, since they are the most unknown factor in this process.

Each of the breach parameter estimates will yield a different outflow hydrograph from HEC-RAS. However, once these hydrographs are routed downstream, they will tend to converge towards each other. How close they get to each other will depend on the distance they are routed, the steepness of the stream, the roughness of the river and floodplain, and the amount of floodplain storage available for attenuating the hydrograph. If the populated areas below the dam are quite a distance away (say 20 miles or more), then the resulting hydrographs from the various dam breaches may be very similar in magnitude by the time they reach the area of interest. However, if the areas of interest are closer to the dam, then the resulting breach hydrographs could produce quite a range in results. In this situation, the selection of the breach parameters is even more crucial.

HEC-RAS Output For Dam Break Analyses

Several plots and tables are available for evaluating the results of a dam break analysis within HEC-RAS. Graphics include cross section, profile, and 3 dimensional plots, all of which can be animated on a time step by time step basis in order to visualize the propagation of the flood wave. An example cross-section plot of a dam while it is breaching is shown in Figure 16.7. Additionally, the corresponding water surface profile for the same instance in time is shown in Figure 16.8. Hydrographs can be viewed at any location in which the user requested hydrograph output. Shown in Figure 16.9 is a series of hydrographs from the breach shown in the previous figures. These hydrographs represent the flow leaving the dam and then subsequent locations downstream as the flood wave moved through the river system.
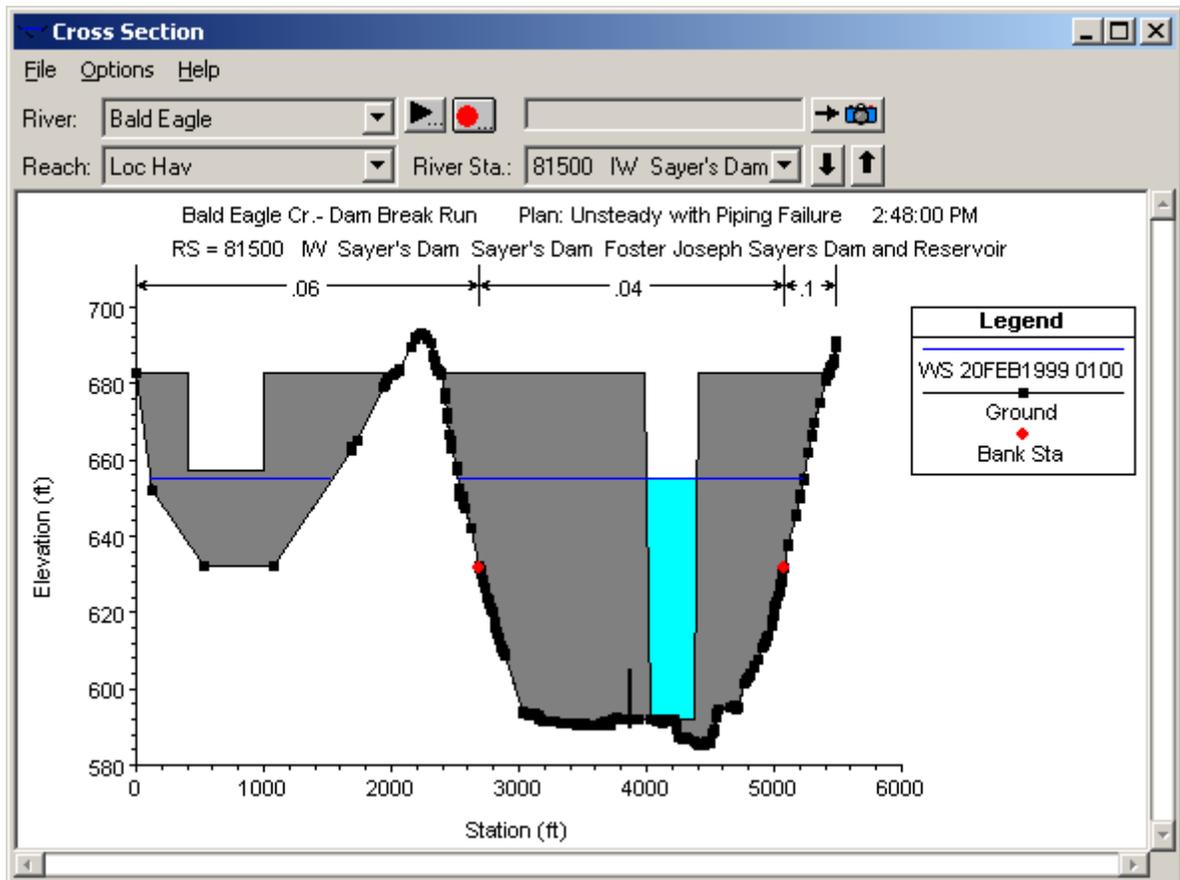
Figure 16.7 Example Plot of Dam While Breaching
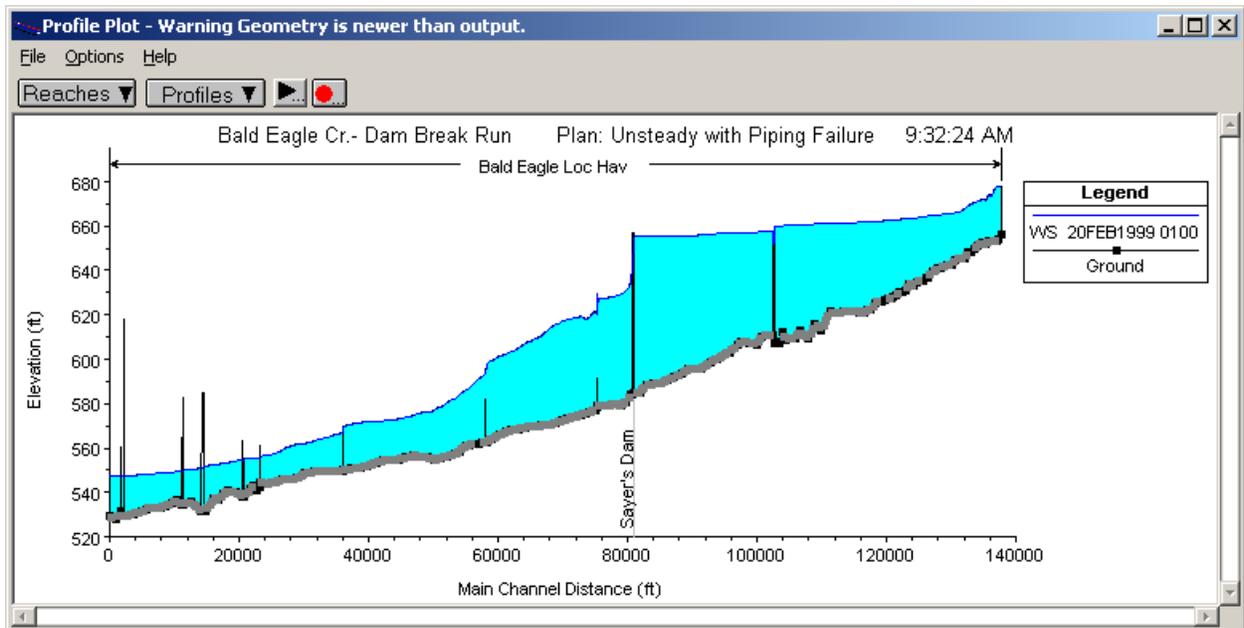
Figure 16.8 Example Profile Plot of Dam Breaching

BALD EAGLE LOC HAV   FLOW

PIPINGFAIL
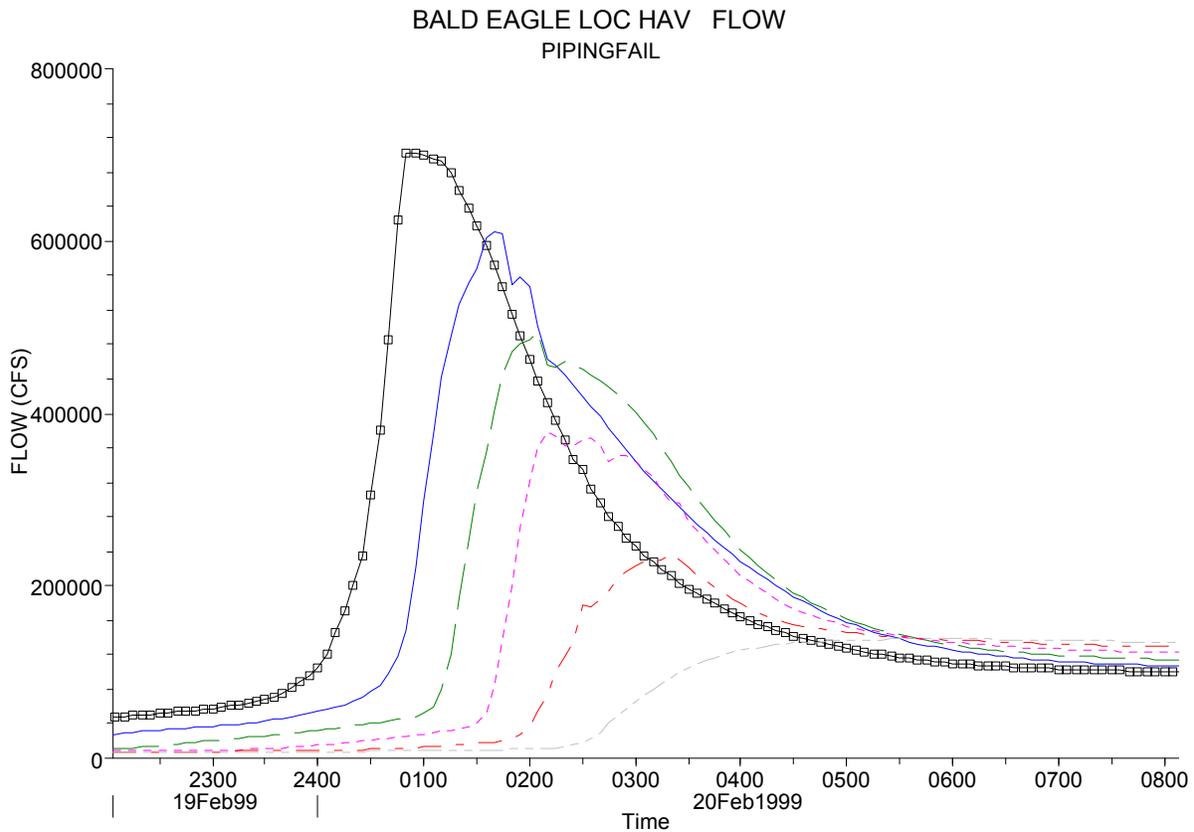
Figure 16.9 Flow Hydrographs From Dam to Downstream Locations.

Levee Overtopping and Breaching

Levee overtopping and breaching can be analyzed within HEC-RAS by modeling the levee as a lateral structure. When modeling a levee with a lateral structure, the area behind the levee should not be included in the cross section data of the main river. The cross sections should stop right at the bottom of the levee. The lateral structure (levee) can be connected to a storage area or another river reach. How you model the area behind the levee will depend upon what will happen to the water if the levee overtops or breaches. If the water going over or through the levee will pond, then a storage area would be more appropriate for modeling the area behind the levee. If the water will continue to flow in the downstream direction, and possibly join back into the main river, then it would be more appropriate to model that area as a separate river reach. Shown in Figure 16.10 is an example schematic with a levee modeled as a lateral structure connected to a storage area to represent the area behind the levee. An example cross section with a lateral structure (levee) on the right hand side is shown in Figure 16.11.
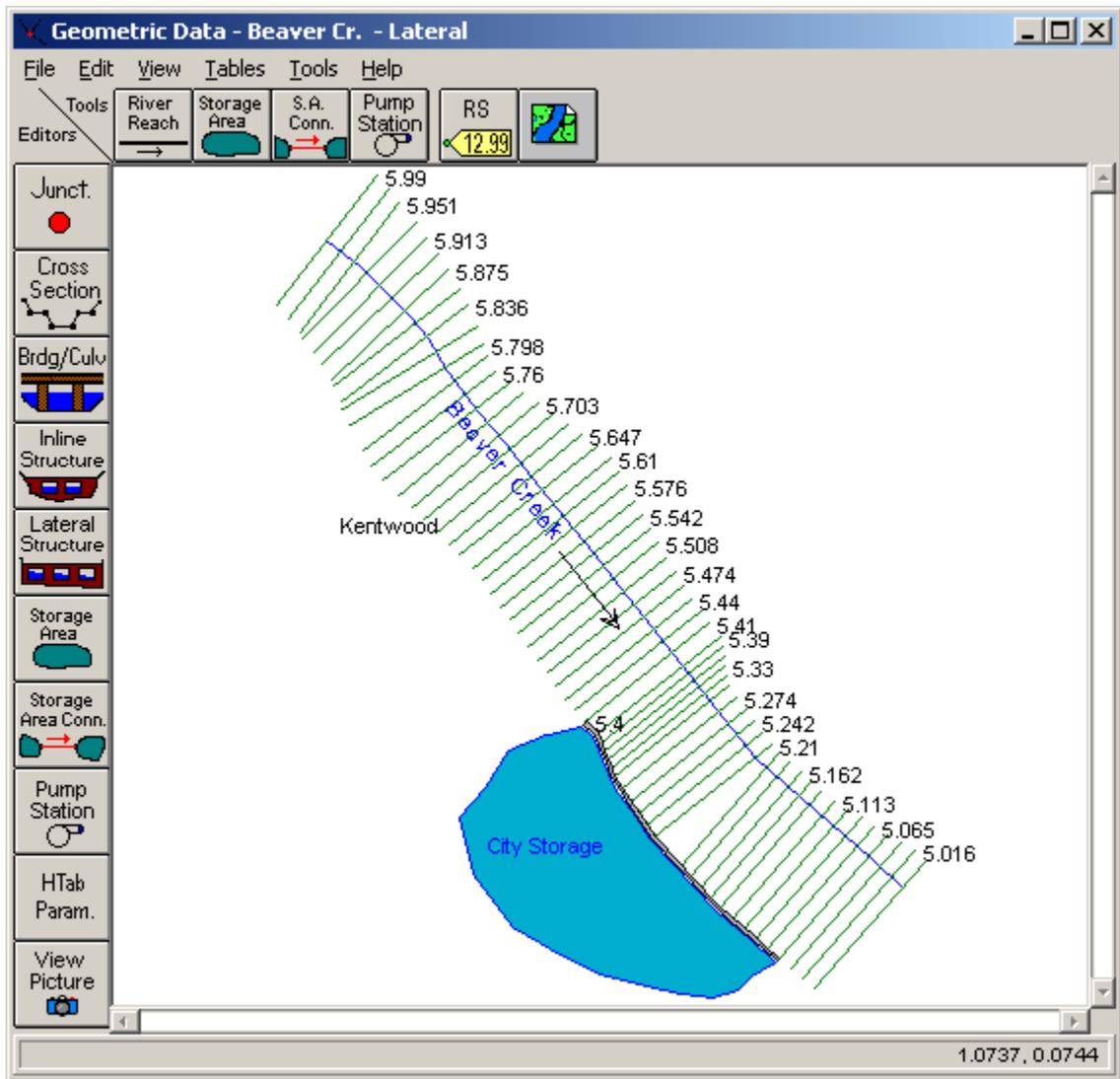
Figure 16.10 Schematic With Example Levee and Storage Area

The user defines the levee by entering a series of station and elevation points that represent the top of levee profile. This station and elevation data is then used as a weir profile for calculating the amount of water going over top of the levee. An example levee entered as a lateral structure is shown in Figure 16.12.
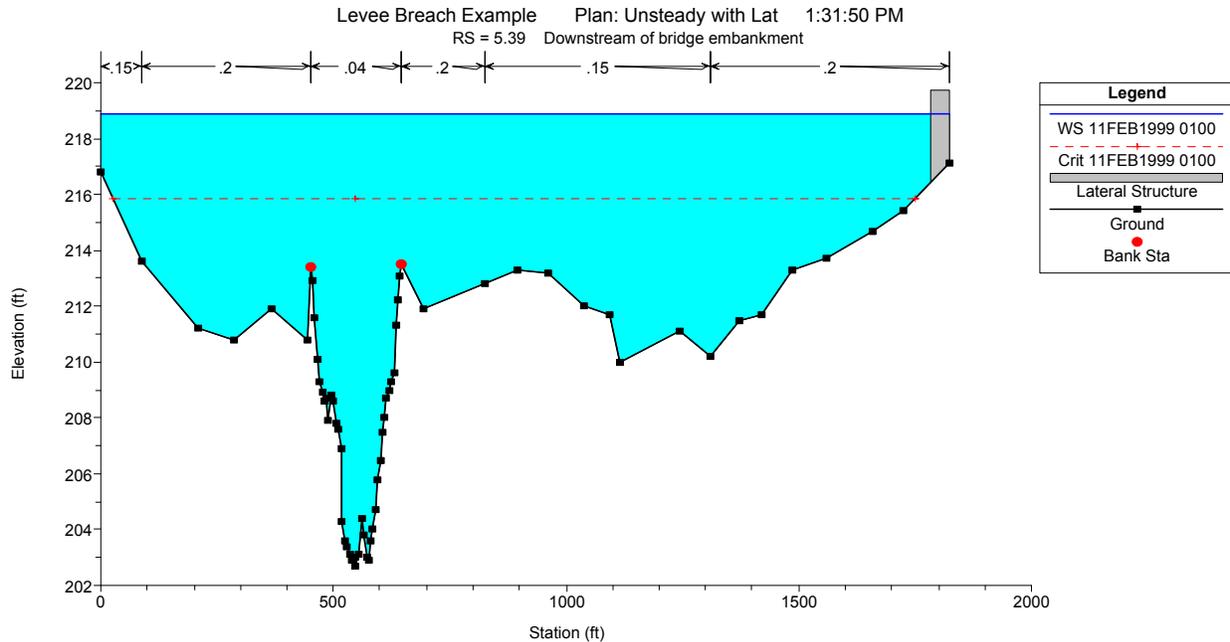
Figure 16.11 Example Cross-Section With Lateral Structure
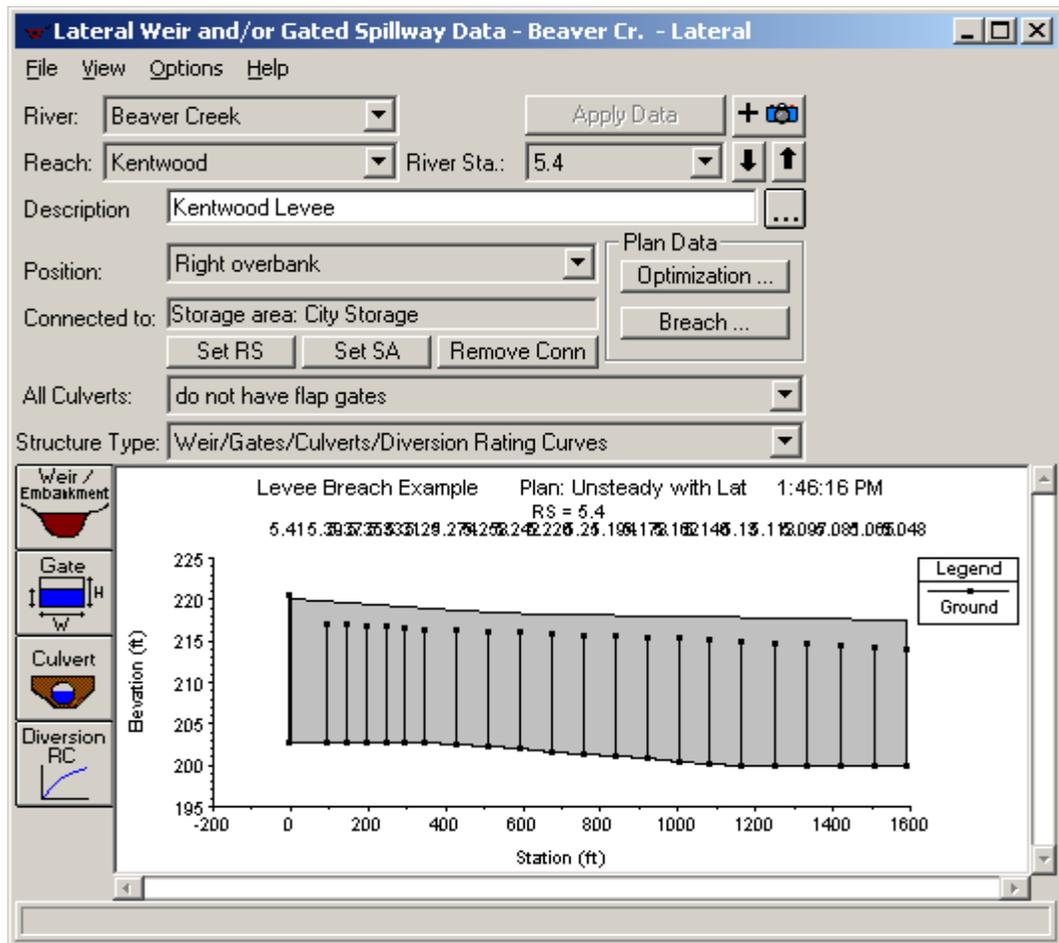
Figure 16.12 Lateral Structure Editor With Levee Modeled as a Weir

In the example shown in Figure 16.12, the levee is connected to a storage area that will be used to represent the area behind the levee. As the levee overtops and/or breaches, the storage area will fill up until it reaches the same elevation as the water in the river. After the flood passes, the water in the storage area can pass back out any breach that may have occurred.

The levee information is entered as station and elevation data in the **Lateral Weir/Embankment** editor shown on the Lateral Structure editor. The station elevation data represents the top of the levee. An example of this editor with levee data is shown in Figure 16.13.
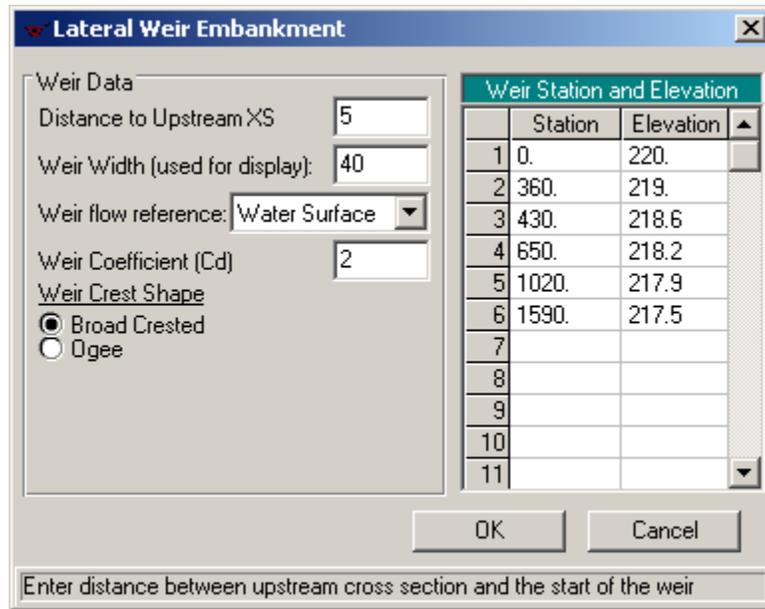
Figure 16.13 Lateral Weir/Embankment Editor With Example Levee Data

As shown in Figure 16.13, the user enters the distance that the upstream end of the levee is from the nearest upstream cross section; the width of the levee (which is only used for drawing purposes); the head reference for weir flow calculations; the lateral weir coefficient; and the station and elevation data representing the top of levee.

Once the physical levee information is entered, the user can press the **Breach** button in order to bring up the levee breach editor. An example of this editor is shown in Figure 16.14.
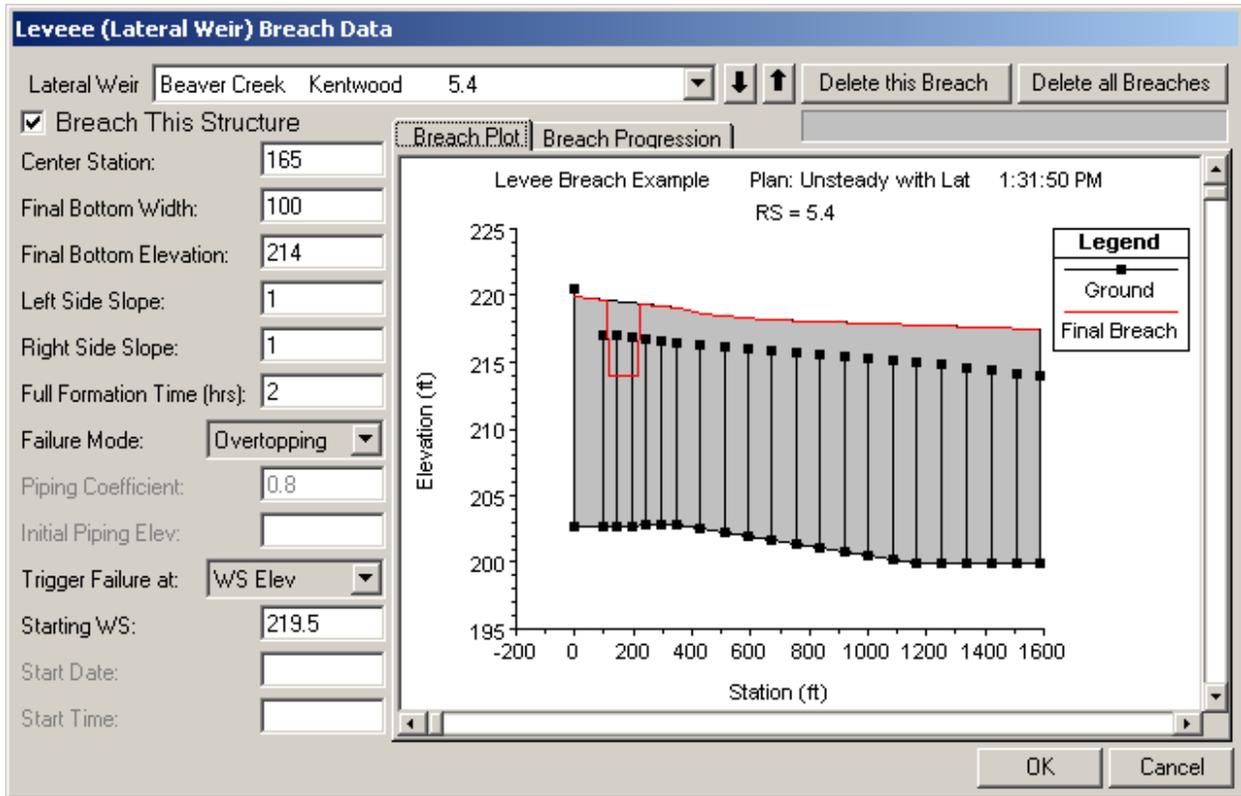
Figure 16.14 Levee Breach Editor With Example Levee and Breach

As shown in Figure 16.14, the information required to perform a levee breach is the same as performing a dam break. To get the details of each data field, please review the information found under the Dam Break section of this chapter.

After all of the data are entered and the computations are performed, the user can begin to look at output for the lateral structure (levee). Plots such as the profile plot, lateral structure hydrographs, and storage area hydrographs, can be very helpful in understanding the output for a levee overtopping and/or breach. Shown in Figure 16.15 is an example profile plot with a levee breach. Shown in Figure 16.16 is a stage and flow hydrograph plot for the lateral structure. In this plot there are three stage lines and three flow lines. The stage lines represent; the stage in the river at the upstream end of the levee (Stage HW US); the stage in the river at the downstream end of the levee (Stage HW DS); and the stage in the storage area (Stage TW). The river is always considered to be the headwater, and the storage area is the tailwater. The flow lines on the plot represent: the flow in the river at the upstream end of the levee (Flow HW US); the flow in the river at the downstream end of the levee (Flow HW DS); and the flow leaving the river over the lateral weir to the storage area (Flow Leaving).
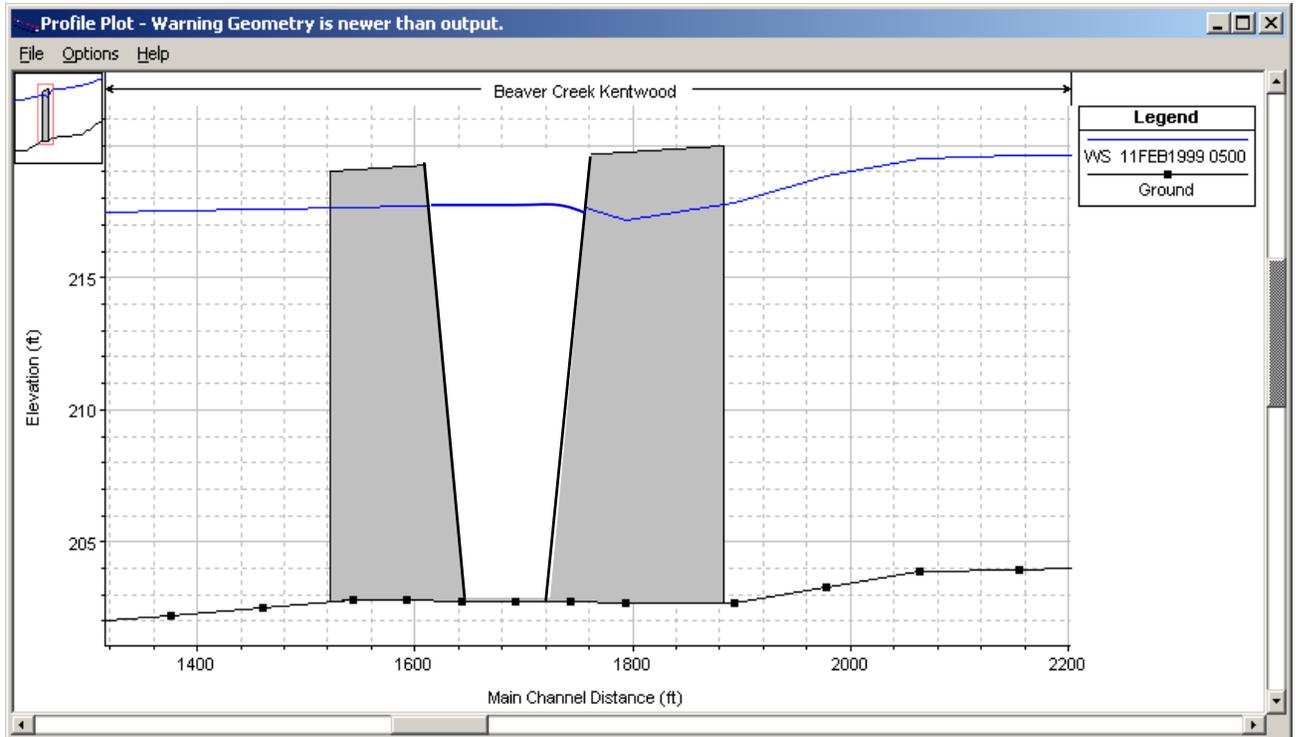
Figure 16.15 Profile Plot With Levee Breach
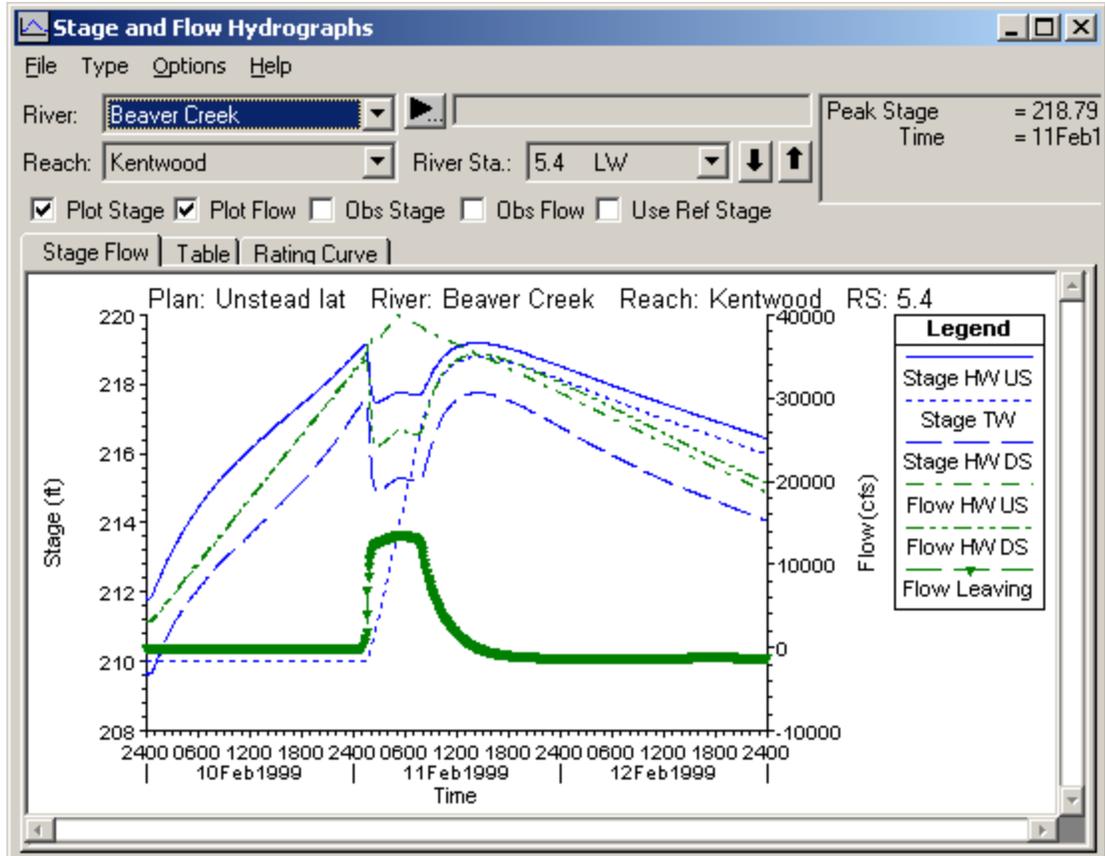
Figure 16.16 Inline Structure Stage and Flow Hydrographs

In addition to the profile plot and the lateral structure hydrographs, it is normally a good idea to plot the stage and flow hydrographs for the storage area. This allows the user to easily see the amount of flow coming into and out of the storage area, and the change in the water surface elevation. Shown in Figure 16.17 is the stage and flow hydrograph for the storage area in this example.
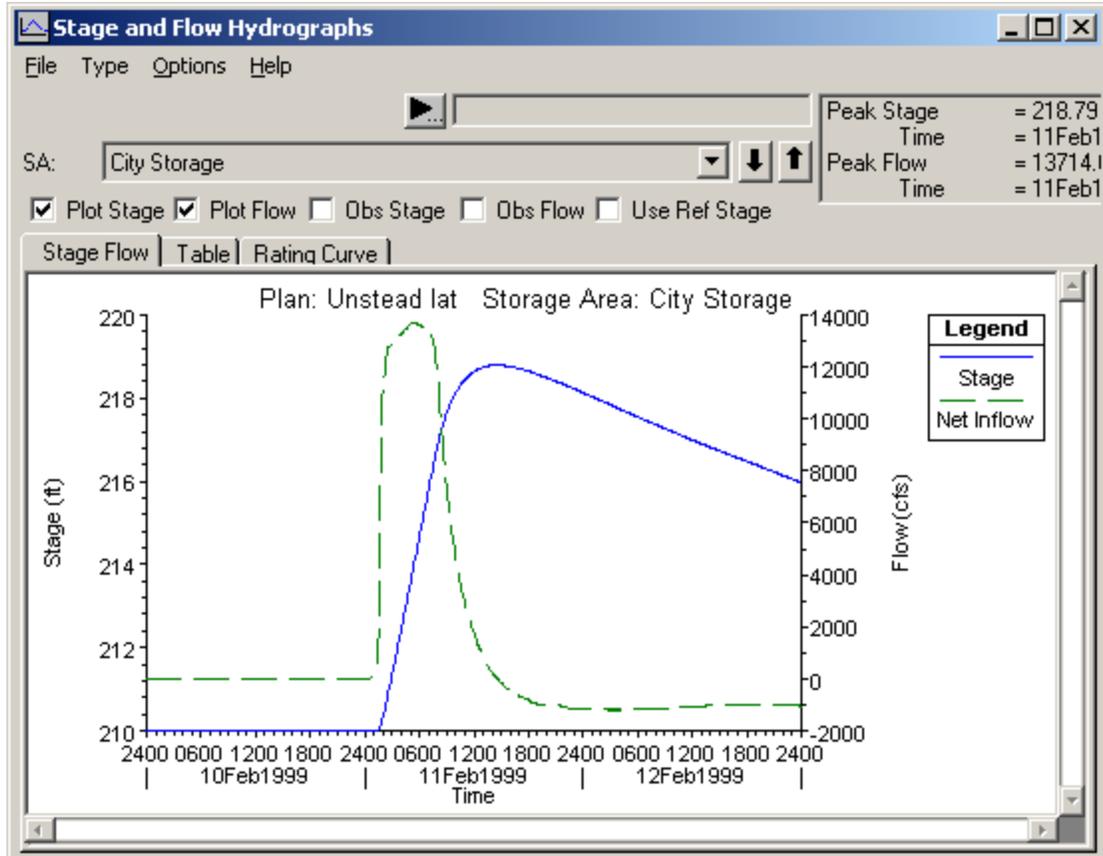
Figure 16.17 Stage and Flow Hydrograph Plot for Storage Area

Referring to Figures 16.16 and 16.17, as the levee breaches, the flow going into the storage area and the stage increase quickly, while the stage and flow in the main river drop. In addition to the graphics in HEC-RAS, tabular results are also available. Shown in Figure 16.18 is a detailed output table for the lateral structure. The user can select a specific time line for viewing the output.
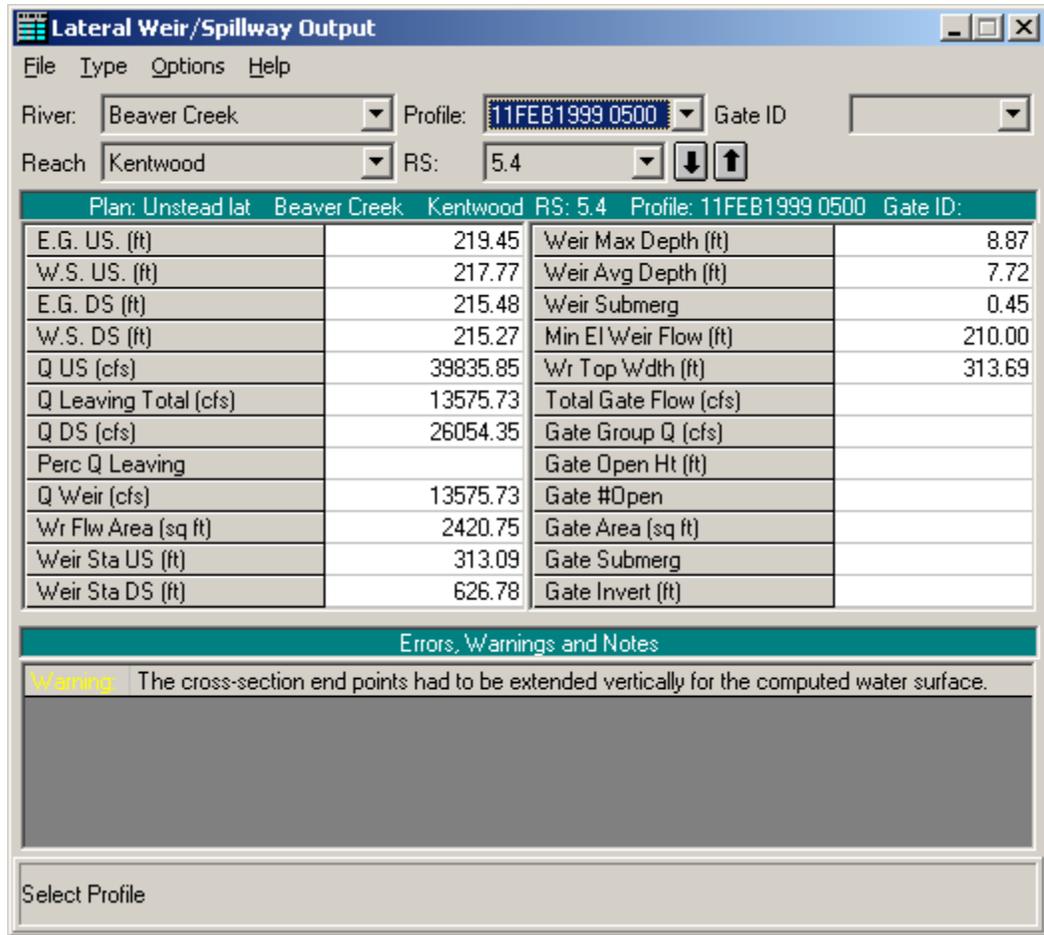
Figure 16.18 Detailed Tabular Output for Lateral Structure

Modeling Pump Stations

Pump stations can be connected between storage areas; a storage area and a river; and between river reaches. The user can have up to ten different pump groups, and each pump group can have up to twenty identical pumps. Each pump can have its own on and off trigger elevation. To learn how to connect a pump and how to enter pump data, please review the section on pumps in Chapter 6 of the user's manual.

Pump stations can be used for many purposes, such as pumping water stored behind a levee (interior sump) into the main river. An example schematic of an interior ponding area behind a levee is shown in Figure 16.19. Note that the pump is connected from the storage area to a river station at the downstream end of the levee.
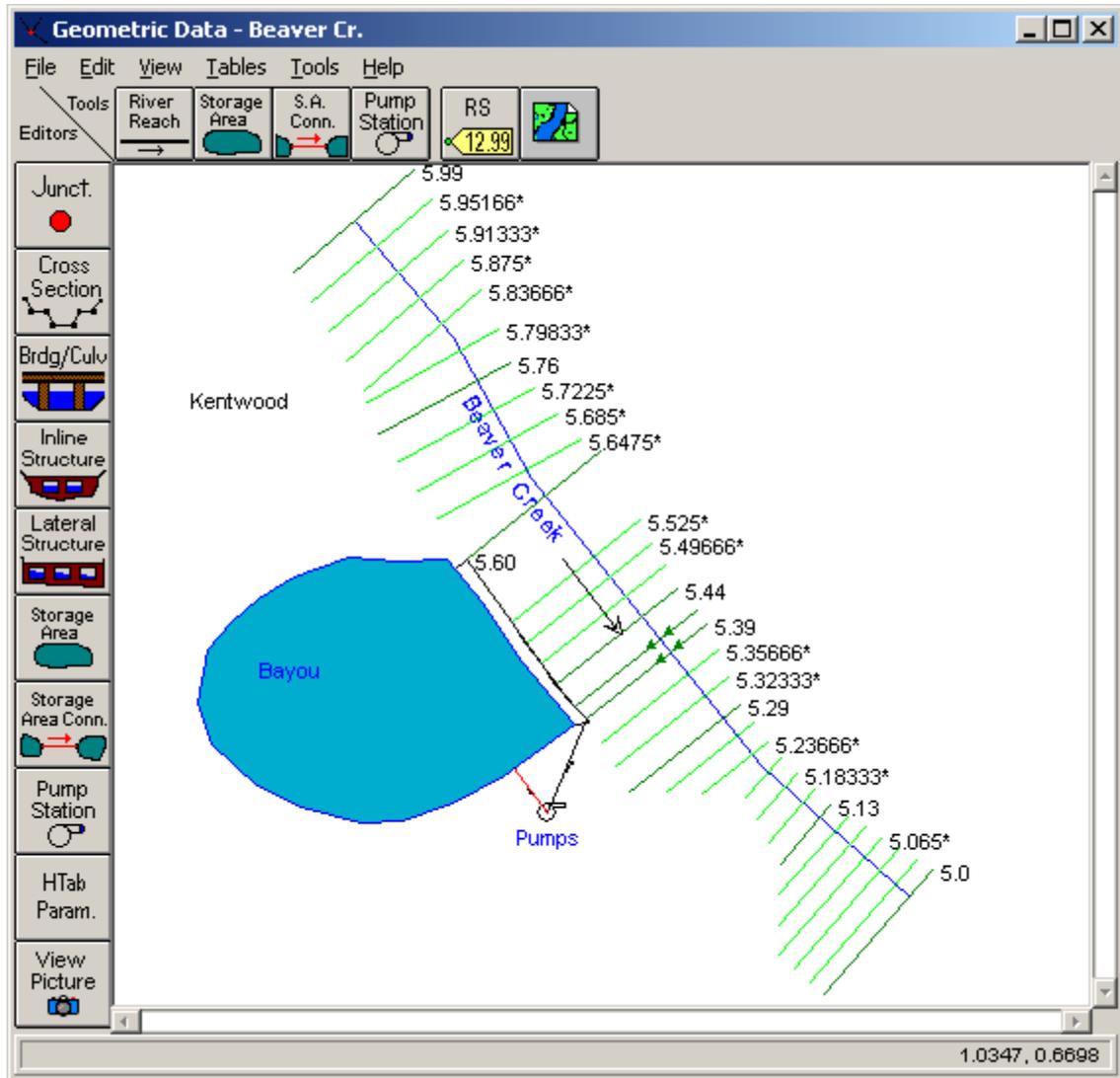
Figure 16.19 Example Pumping Station for Interior Ponding Area

In the example shown in Figure 16.19, a lateral structure was entered to represent the levee. This structure has a gravity draining culvert with a flap gate. The flap gate only allows water to drain from the storage area to the river. Additionally, a pump station is included to pump flows over the levee during a rainfall event. The pump station was drawn by selecting the **Pump Station** tool, then drawing a connection from the storage area to the cross section at river station 5.39.

In this example there is a hydrograph attached to the upstream end of the river reach, which represents the incoming flood wave to this reach. There is also a lateral inflow hydrograph attached to the storage area, which represents the local runoff collecting behind the levee. The pumps are used to pump water from the storage area, over the levee, to the river. The top of the levee is

around elevation 220 feet. Therefore, the pump station is constantly pumping to a head of 220 feet. The data for the pump station is shown in Figure 16.20.
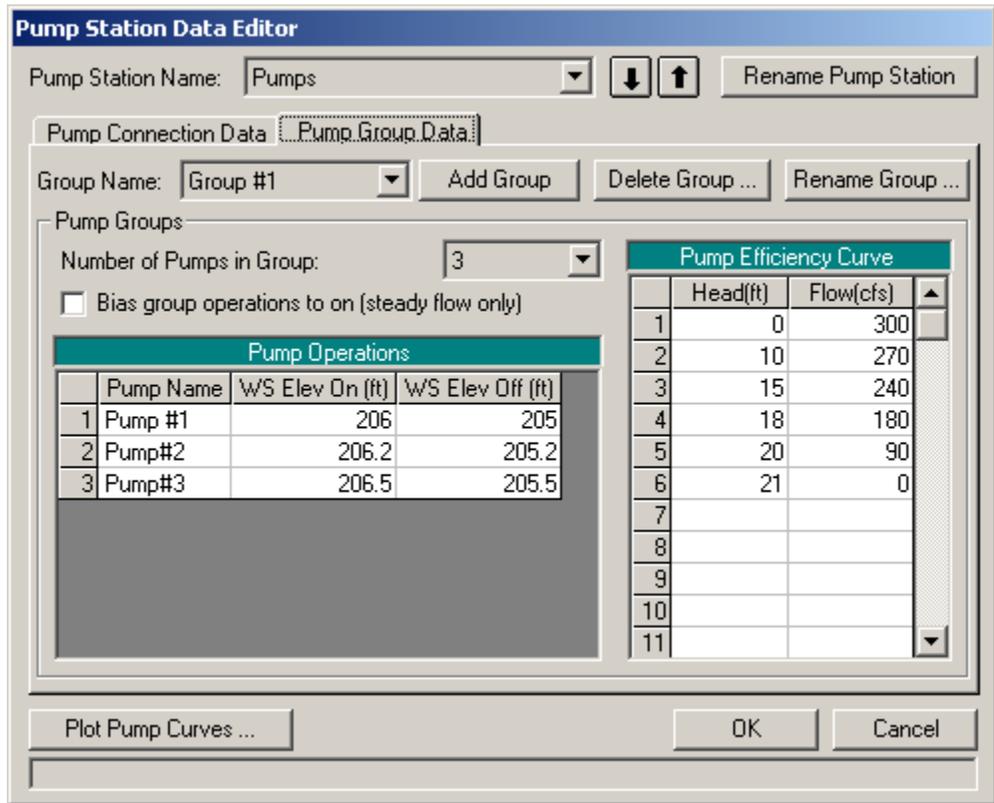
Figure 16.20 Pump Station Editor With Example Data.

As shown is Figure 6.20, there is one pump group with three identical pumps (pumps are the same size and capacity). However, each of the pumps has a different on and off trigger elevation. The pump efficiency curve is used for all three of the pumps.

After the computations are performed the user can view output for the pump station by selecting the stage and flow plotter, then selecting **Pump Stations** from the **Type** menu at the top of the window. An example stage and flow plot for the pump station is shown in Figure 16.21. As shown in the figure, the stage for the tailwater location (Stage TW) is a constant 220 ft. This is due to the fact that the pump is constantly pumping over the levee at elevation 220. The stage at the headwater location (stage HW) is the water surface elevations in the storage area. The storage area elevation starts out at an elevation of 205 ft., goes up to around 206.6, and then back down to around 205.1. The flow through the pumps was zero until an elevation of 206 was reached within the storage area. The second pump turned on when the storage area got to elevation 206.2, and the third at elevation 206.5. On the following side of the hydrograph the pumps began to turn off as the stage went down in the storage area. Shown in Figure 16.22 are the stage and net

inflow to the storage area. The net inflow represents all the inflows minus the outflows at each time step.

Figure 16.21 Stage and Flow Hydrographs for Pump Station
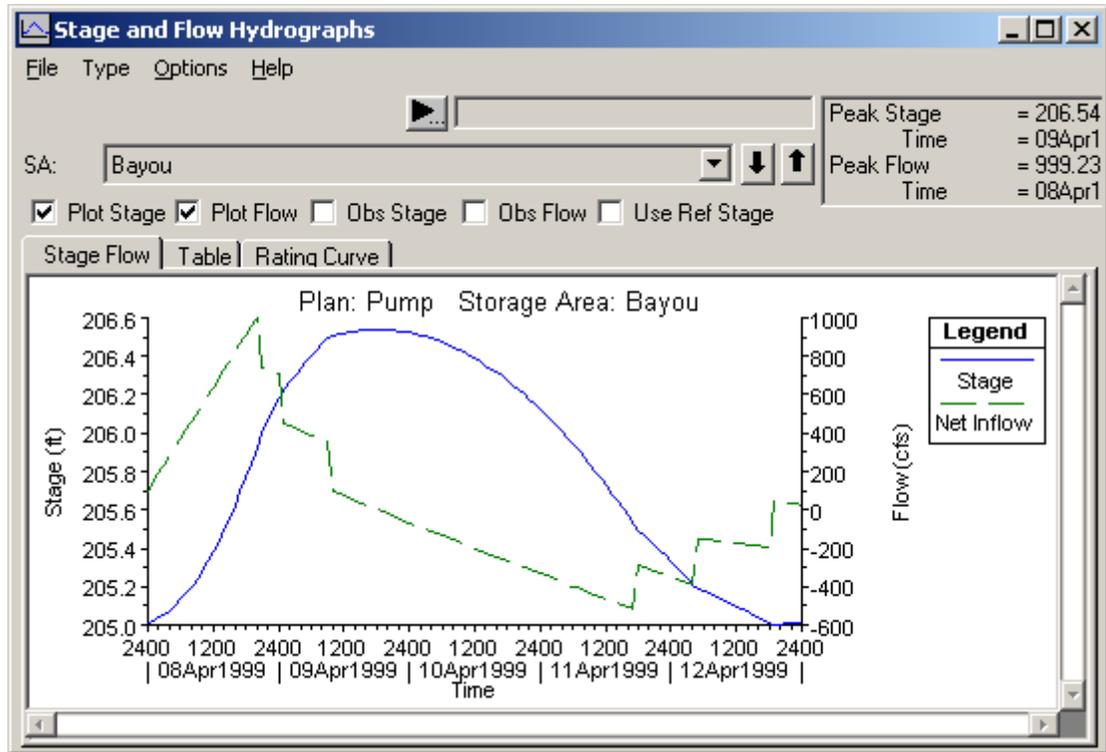
Figure 16.22 Stage and Flow Hydrographs for the Storage Area

Navigation Dams

This section discusses the navigation dam option in HEC-RAS. For a navigation dam, the program will try to maintain both a minimum and maximum water surface in one or more locations along a navigation channel. The program does this by controlling the gate settings on an inline structure. The user enters a target water surface (and various other calibration data) and the program will adjust the gate settings at user specified time intervals in order to meet the target water surface as closely as possible. This section describes the data requirements for a navigation dam and includes a general discussion of how the gate operations are performed.

The first step in modeling a navigation dam is to add the physical data for the navigation dam by selecting the Inline Structure option on the Geometry Data editor and entering the appropriate information. The next step is to add the inline weir as a boundary condition on the Unsteady Flow editor and then click the **Navigation Dams** button. The editor, as shown in Figure 16.23, will appear (note: the fields will be blank when first brought up).

Figure 16.23. Navigation Dam Editor with Flow Monitor

Normal gate change time increment – This field states how often the program will adjust the gate settings. In the example shown in Figure 16.23, the program will only make adjustments to the gates every six hours under normal operations.

Rapidly varying flow change increment – This field represents the minimum length of time between gate setting adjustments. For example, during rapidly changing conditions, the program can adjust the gates up to once an hour in order to maintain the appropriate water surfaces.

Initial gate change time – This field is the military style time for when the first gate change will take place. In this example, it is 10:00am. If the simulation starts after 10:00am then the gates will be first adjusted at 4:00pm, 10:00pm, or 4:00am as appropriate.

Gate minimum opening – This field is the minimum opening for the first gate group (the first gate group as defined on the Geometry editor). The program will keep the gates on this gate group open to at least 0.1 feet. The other gate groups may be closed completely (see discussion of gate opening and closing below).

The final two fields [*Gate opening and Gate closing rate*] are the maximum speed that the gates in any gate group can be opened or closed. Generally this rate is determined by the physical speed with which the gates can be adjusted. Sometimes, however, opening or closing the gates too quickly can cause instability in the unsteady solver. In this case, it may be necessary to reduce the opening or closing rate. But a shorter time step may also help.

Pool Only Control

There are several types of navigation dam operations. The simplest is pool only control (as shown in Figure 16.23). In this case, the program tries to maintain the water surface immediately upstream of the dam within user specified targets. In the other operations (see below), the target water surface is located some distance upstream of the dam and there may or may not be limits on the water surface right at the dam.

In order to keep the water surface at the dam within the user specified limits, while only infrequently changing the gate settings (i.e., every six hours), the program needs to know what the approximate inflow at the dam will be some time into the future. This is done by monitoring the flow at an upstream cross section. The user must enter this location. In the above example (Figure 16.23), the **Flow Monitor** tab has been activated and the flow monitor location has been entered as river station 315.5. The flow monitor location should be chosen so that the river travel time between the monitor location and the navigation dam is on the order of (or somewhat less than) the normal gate increment. In this example (Figure 16.23) the gate time increment is every six hours, so a location a few hours upstream would be appropriate.

The calibration of the navigation dam control data involves some empirical decisions and trial and error experimentation. This is true of the flow monitor location as well as most of the remaining data below.

The flow monitor location must be a normal cross section in the model. This means that cross sections must be extended far enough upstream of the dam to account for this location. At some point, an option may be added for the navigation dam to see into the upstream hydrograph so that there is no minimum requirement of upstream cross section distance. Note also that the monitor point can be located upstream of other hydraulic structures including other navigation dams. As long as another upstream navigation dam does not have a significant storage capacity, it should not affect the results of the flow monitor. The ability of one navigation dam to "talk" to a downstream dam may be added in the future.

After the flow monitor location has been chosen, the **Pool Control** tab can be pressed bringing up the editor shown in Figure 16.24.

Figure 16.24. Navigation Dam Editor with Pool Control

The user enters a range of water surfaces and corresponding *Flow Factors*. In this example, the ideal target water surface has been entered as 459.35. The primary target range is from 459.2 (Target Low) to 459.5 (Target High). In general, if the water surface is between *Target Low* and *Target High* and it is time to change the gate settings, then the program will adjust the gates to get an average of the current flow at the dam and the monitor flow.

For instance, assume that at time 10:00 the current discharge from the navigation dam is 10,000 cfs, 11,000 cfs of flow is observed at the monitor location, and the water surface at the dam is 459.4 feet. Since 459.4 is in the primary target range, the program will compute the average of the flows, 10,500 cfs. By trial and error, the program will change the gates (and compute the corresponding flow) until there is 10,500 cfs (plus or minus the tolerance) of discharge at the dam. The tolerance is 1% of the flow, in this case 105 cfs. So the program will actually stop iterating whenever it first determines a gate setting that results in a flow that is between 10395 cfs and 10605 cfs. After the gates have been changed, they won't (normally) be adjusted for the next six hours. The flow from the dam will, of course, still vary as the water surface at the dam fluctuates.

As the water surface at the dam gets out of the primary target range, then the flow (that is, the discharge from the dam) is adjusted by the Flow Factors. In general, when the stage is between Target High and Maximum, then the flow is multiplied by Flow Factor Target High (in this case 1.03). Between Maximum and Maximum High, it is multiplied by at least 1.07. Between Maximum High and water surface Open River, the flow is rapidly increased up to at least Flow Open River (listed as 50,000). Flow Open River does not represent a cap. If the flow at the monitor location gets high enough, the discharge at the dam can go above Flow Open River based on the Flow Factors. Above water surface Open River, all the gates are opened all of the way.

The operations below the target zone work the same way. Flow Factor Target Low and Flow Factor Minimum are applied in the same way. Between Minimum Low and water surface Close Gates, the flow will be rapidly decreased to Flow Minimum, but again, this is not an absolute minimum. If the water surface remains low enough, the program will continue to close the gates and reduce flow. The only absolute minimum is that the program will not close the first gate group below the gate minimum opening.

The water surface targets are basically calibration knobs and no particular water surface targets have to exactly match the operationally prescribed limits on the pool surface. However, the best response will probably be obtained if the Maximum and Minimum are close to the prescribed limits.

Hinge Point Only Control

The next type of navigation dam operation is hinge point control. This is similar to pool control. The main difference is that instead of the water surface targets being located right at the face of the dam, the water surface targets are located some distance upstream. Figure 16.25 shows the Hinge Point Only editor. (Hinge point control is selected by clicking on the drop down box near the top right of the editor.)

In this example, the navigation dam is located at river station 714.35, and the hinge point is located at 728.28. The program will adjust the gates at the dam in order to maintain an approximate water surface of 645.55 feet (the target water surface) at river station 728.28. The target water surfaces and Flow Factors behave the same as for pool control. A flow monitor location is still needed. It should be located an appropriate distance upstream of the hinge point. For this dam, it is located a few hours upstream at river station 750.1 (see Figure 16.25).

The Steady Profile Limits Table is an optional feature (see Figure 16.25). It can make the navigation dam operations more robust for rapidly changing flow. It addresses the situation where the water surface for a give flow at the dam diverges significantly from the water surface that would be expected at the dam for a steady state, uniform flow condition.

A typical example is the trailing end of a high flow hydrograph. For instance, the flows at the hinge point and monitor location may have fallen considerably below the Open River condition, but the water surface at the dam is still a little high (compared to the flow). When the program computes a desired flow at the dam, e.g. 10,000 cfs, it adjusts the gates to get this flow. Over the next six hours, however, as the water surface at the dam continues to fall toward a lower equilibrium, the discharge can drop significantly below 10,000. This means that the navigation dam response is either sluggish in returning to the target water surface at the hinge point and/or the gates have to be changed more frequently. This is where the table becomes useful.

The data in this table give the water surfaces at the dam that will produce the target water surfaces at the hinge point for steady state conditions. For this navigation dam, it is desired to keep the water surface at the hinge point between 645.3 and 645.7 feet. If, for instance, there is a long term (steady state) flow of 10,000 cfs between the hinge point and the dam, then maintaining a water surface at the dam of 645.19 feet will result in a water surface of 645.3 feet at the hinge point. Similarly, a water surface of 645.59 at the dam will result in a water surface of 645.7 feet at the hinge point, for the same 10,000 cfs flow. The table can be semi-automatically generated by HEC-RAS. After entering the hinge points limits (inside the Generate WS Limits box) and entering a range of flows (left-most column of the table), the user can click on the **Compute Limits** button and the program will use the steady state run model to fill in the rest of the table.
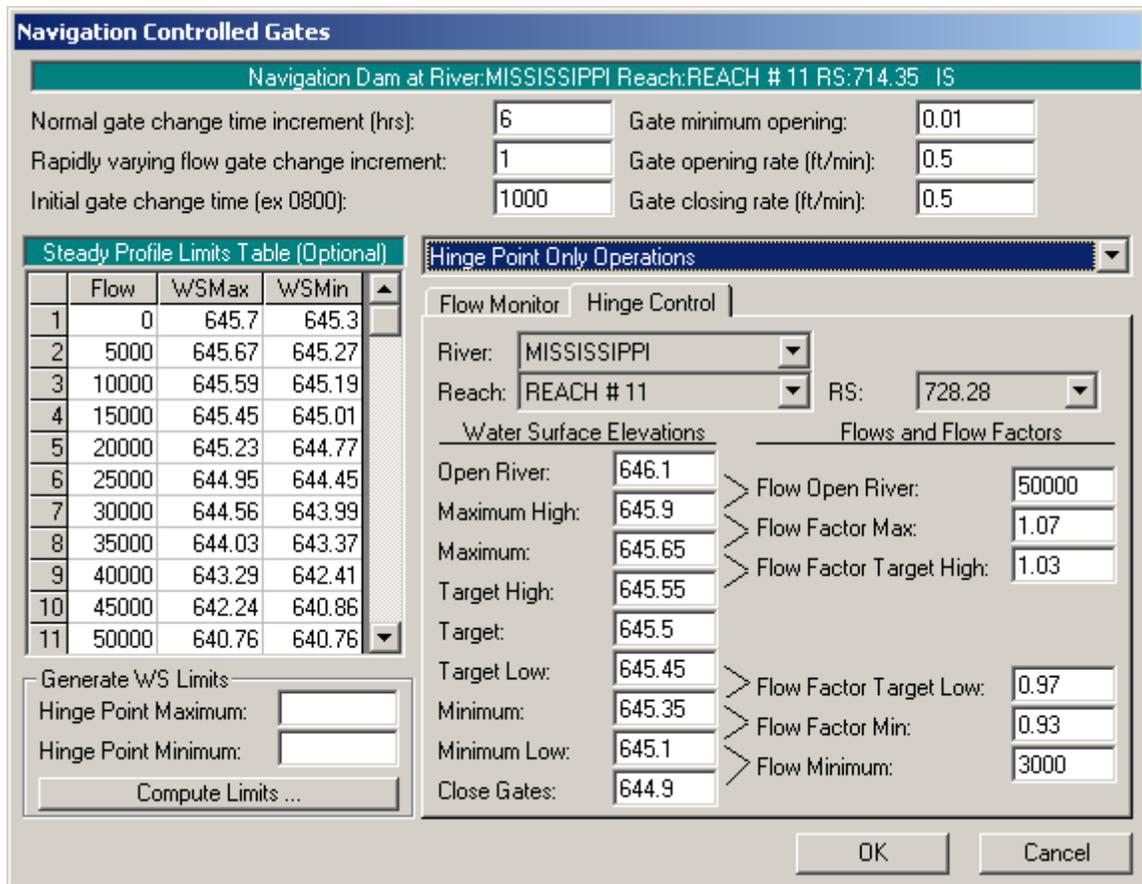
Figure 16.25. Navigation Dam Editor with Hinge Control

Continuing on with the 10,000 cfs flow example, before the program starts to iterate, it checks the current water surface at the dam against the table. If the current water surface is between the limits (in this case 645.19 and 645.59), the program continues normally as it would if the table was not being used (that is, the user had left it blank). However, lets assume that the water surface at the dam is 646.0 feet. This would mean that the water surface at the dam is above the limits. In this case the program will temporarily assume a headwater of 645.59 feet at the dam and determine the gate settings that will result in a discharge of 10,000 cfs for this lower, assumed, headwater. After this has been done, the program will use this gate setting and continue on normally. This will result in the flow at the dam initially being above the 10,000 cfs target. However, as the water surface at the dam drops, the flow should also drop down towards the 10,000 cfs range. This will, hopefully, produce a faster response without over shooting the target water surface at the dam.

If the water surface is on the low side, it works the same way except the lower limit is used. If the water surface at the dam were 645.0 then the gate setting would be based on an assumed water surface of 645.19. The profile table is optional. It can be ignored, by leaving it blank. However, it can produce a better response, at least for some data sets. That being said, it should also be

noted that the table will not perform as well when the flow at the dam is being heavily influenced by tailwater conditions.

Figure 16.26. Navigation Dam Editor with Steady Profile Limits Table

Hinge Point and Minimum Pool Operations

The hinge point navigation dam operation can also be combined with limits on the water surface at the dam. Hinge point and minimum pool operation will try to maintain the water surface within targets at the hinge point, but only when the water surface at the dam is above certain limits. When the water surface at the dam drops too low, the program will adjust the gates based on the water surface at the dam, essentially reverting to pool only control.

The hinge point and the minimum pool operation are each treated as separate control points. In addition to the water surfaces and Flow Factors for the Hinge control, the pool minimum has its own full set of water surfaces and Flow Factors as shown in Figure 16.27 (these are accessed by clicking on the **Min Pool Control** button). Even though the minimum pool control is only trying to maintain a minimum water surface at the dam, a full range of water surfaces and Flow Factors are needed. These include the "too high" numbers

such as Maximum High and Flow Open River. This allows the program to smoothly transition between hinge control and pool control. It also allows the pool control response to be fully calibrated between sluggish and overly sensitive transitions.

For hinge and pool minimum navigation dams, the program independently determines a desired flow for each control point (that is, the hinge and the pool minimum). It will then take the lower of the two flows and use that for determining the gate settings.

For instance, assume the flows at the monitor location and the hinge point are 40,000cfs and that the water surfaces at the hinge point and the dam are 645.6 and 644.9 respectively. Based on the hinge point conditions (water surface at hinge point, Targets and Flow Factors for the hinge point), the program might compute a desired flow of 41,000cfs. Next the program will look at the conditions, targets, and Flow Factors at the dam and compute a desired flow of, perhaps, 42,000 cfs. Since the desired flow for the hinge point targets is lower than the desired flow for the navigation dam targets, the pool minimum is not a limiting factor. The program will adjust the gate settings to get 41,000 cfs and the navigation dam is operating under hinge control.
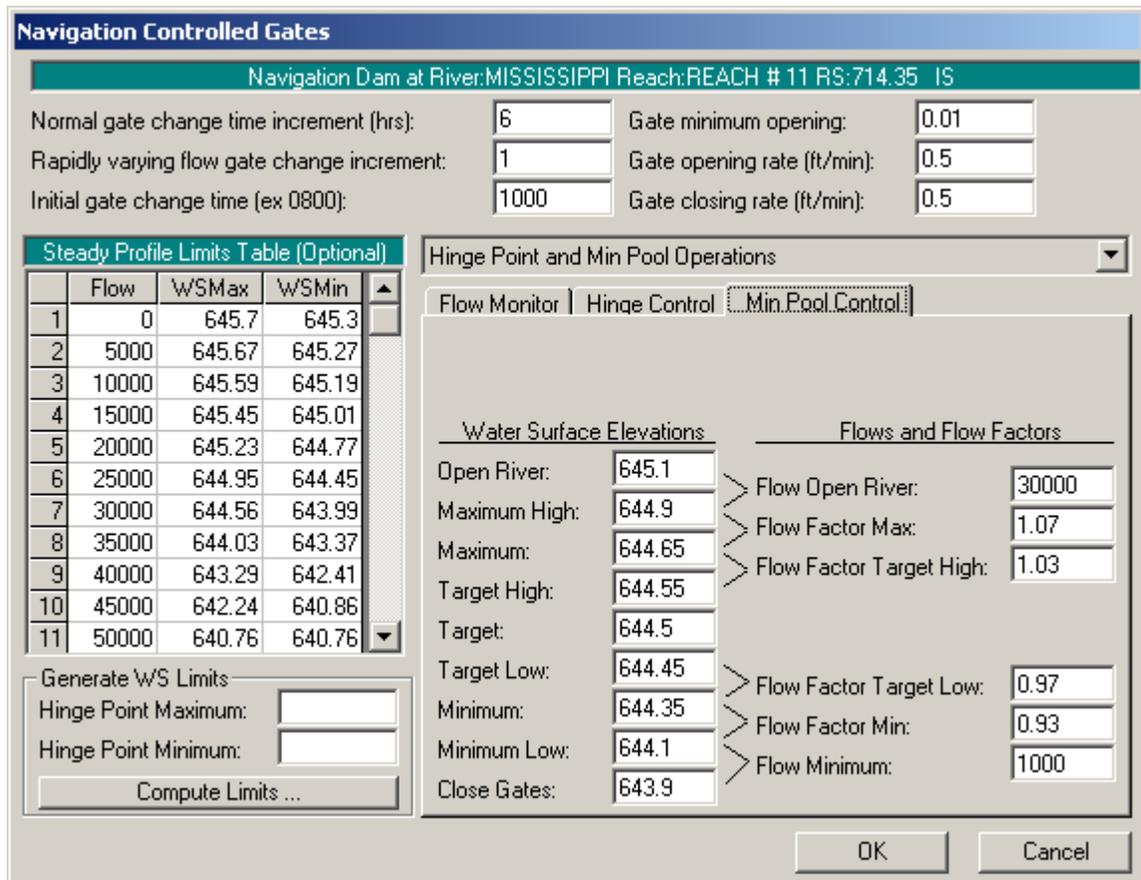
Figure 16.27. Navigation Editor with Hinge Point and Minimum Pool Operations and Control

The next time the gates are adjusted, assume the flow at the monitor and hinge point are still basically 40,000 cfs, but that the water surfaces have dropped to 645.5 feet at the hinge and 644.4 feet at the dam. The new computed flows might be 40,000 cfs at the hinge and 39,000 at the dam. In this case the program would use the 39,000 cfs figure and the dam would be under pool minimum control. In other words, the water surface at the dam has dropped to the point that the program has to operate the gates to maintain a minimum water surface at the dam regardless of what is happening at the hinge point.

The hinge and pool minimum operation is usually under hinge control for low and normal flows. At high flows the water surface at the dam must be lowered in order to keep the hinge point within the target range. At even higher flows, the water surface at the dam cannot be lowered far enough to keep the hinge point in range, thus the dam reverts to pool minimum control. Ideally, the pool would be kept at the specified absolute minimum (perhaps 644.1 feet in the above example) until the hinge point dropped back down into the target range. This is not possible without continuous adjustments of the gates, which is not practicable.

Instead, the water surface at the dam will fluctuate slightly even when it is operating under pool minimum control (just like it would fluctuate for pool only control). This is reflected in the range of target water surfaces for pool minimum control. The spacing of the target water surfaces have to be determined by trial and error. For instance, if the water surface Target, Target High, and so on, are set to relatively high elevations (compared to the desired value), then the water surface at the dam might stay significantly above the minimum of 644.1. This is not desired when the water surface at the hinge point is above the targeted range. Moving the dam target water surfaces closer together (closer to 644.1) will cause the program to increase the flows more quickly in order to drive the water surface back down. However, this can also cause the program to overshoot the desired target leading to frequent gate changes and/or bouncing water surfaces.

If the pool minimum is a hard minimum (a hard minimum might be, the pool should not be allowed to drop below 644.1 feet), then this minimum should be coded as one of the lower target water surfaces. For instance, if 644.1 is the operationally prescribed absolute minimum and the user coded the primary water surface Target as 644.1, then the pool would fluctuate around the value of 644.1 during pool control. It would be better, in this case, to code it to the Minimum Low, for example. On the other hand, if the minimum is a "soft" minimum (a soft minimum might be 644.45 +/- .25 feet) then setting Target Low or even perhaps the primary Target to 644.45 might give better results. As already mentioned, the user should be prepared to take a trial and error approach in order to get the best results.

For hinge point and minimum pool operation, the Steady Profile Limits table can still be optionally used. This table is only used when the dam is operating under hinge control. The water surface values in the table can be lower

elevations than the actual limits on the pool. These values are still used, but the pool control minimum will still apply. For instance, the values in the table go below the 644.1 desired minimum at the pool. During rapidly changing conditions, when the water surface for a given flow diverges from the steady state water surface (for that flow), these lower values can still be used and will (in some cases) give a faster response. However, if the water surface actually drops down to around the 644 to 645 level, the flow based on pool control will eventually be lower than that based on the Hinge/Steady Profile table and the dam will revert to pool control (which, again, does not use the tables).

Hinge Point and Minimum and Maximum Pool Control

The final type of navigation dam operations is combining hinge point control with both a minimum and maximum limit on the water surface at the dam. This editor has a third button as shown in Figure 16.28 and Figure 16.29.

The minimum and maximum pool controls are treated as separate control points even though they are both located immediately upstream of the dam. They each have a full set of target water surfaces and Flow Factors. The program will compute a desired flow for each control point. So there will be a flow based on the hinge point targets, a flow based on the minimum pool elevation, and a flow based on the maximum pool elevation. During normal operations, the flow will be based on the hinge point target. However, the desired flow will not be allowed to go below the minimum pool control flow and it will not be allowed to go above the maximum pool control flow.

Having separate control points for the minimum and maximum control allows a smooth transition between pool control (either high or low) and hinge control for a full range of flows. It also provides the greatest control and sensitivity for allowing the water surface at the pool to be maintained within the tightest tolerances.

The optional steady profile limits table may still be used. As before, it only applies to hinge control.

Navigation Controlled Gates

Navigation Dam at River:MISSISSIPPI Reach:REACH # 17 RS:273.47   IS

Normal gate change time increment (hrs):	6	Gate minimum opening:	0.1
Rapidly varying flow gate change increment:	1	Gate opening rate (ft/min):	0.5
Initial gate change time (ex 0800):	1000	Gate closing rate (ft/min):	0.5

Steady Profile Limits Table (Optional)

	Flow	WSMax	WSMin
1			
2			
3			
4			
5			
6			
7			
8			
9			
10			
11			

Hinge Point and Min and Max Pool Operations

Flow Monitor | Hinge Control | Min Pool Control | Max Pool Control

Water Surface Elevations — Flows and Flow Factors

Open River:	446.1
Maximum High:	446
Maximum:	445.9
Target High:	445.8
Target:	445.7
Target Low:	445.6
Minimum:	445.5
Minimum Low:	445.4
Close Gates:	445.3

Flow Open River:	150000
Flow Factor Max:	1.07
Flow Factor Target High:	1.03
Flow Factor Target Low:	0.97
Flow Factor Min:	0.93
Flow Minimum:	3000

Generate WS Limits

Hinge Point Maximum:	
Hinge Point Minimum:	

Compute Limits ...

OK   Cancel

Figure 16.28. Navigation Editor with Hinge and Maximum and Minimum Control (Min Pool Control Shown)

Figure 16.29. Navigation Editor with Hinge and Minimum and Maximum Control (Max Pool Control Shown)