Diagnosing and Fixing Common Stability Problems Solution

Introduction

This workshop was intended to help students learn how to use find, diagnose, and fix common model stability problems when using HEC-RAS to perform a Dam Break analysis.

NOTE: While this data is from an actual river system, the model and results of this workshop do not represent current or future conditions of the river. The United States Army Corps of Engineers has granted access to the information in this model for instructional purposes only. Do not copy, forward, or release the information without United States Army Corps of Engineers approval.

Problem Description

Students were given an HEC-RAS project with a Title of "**Bald Eagle Cr. Dambreak Stability Wrkshp"**, and a file name of "**BaldEagleStabilityWorkshop.prj**".

This data set is the same project that was used in the Dam Break workshop. However, the data set contained **six significant problems**. Students were asked to run the model, find the six problems, and fix them in order to get a stable and hydraulically reasonable model.

Solution Description

Trial 1. The first step was to open the model and execute the unsteady flow plan that was already developed. When the computations were performed, the model went unstable very early in the run. Shown in Figure 1 is the computation window for the unsteady flow calculations. As shown in Figure 1, the model had very larger errors starting on 02JAN1999 at 1830 hours.

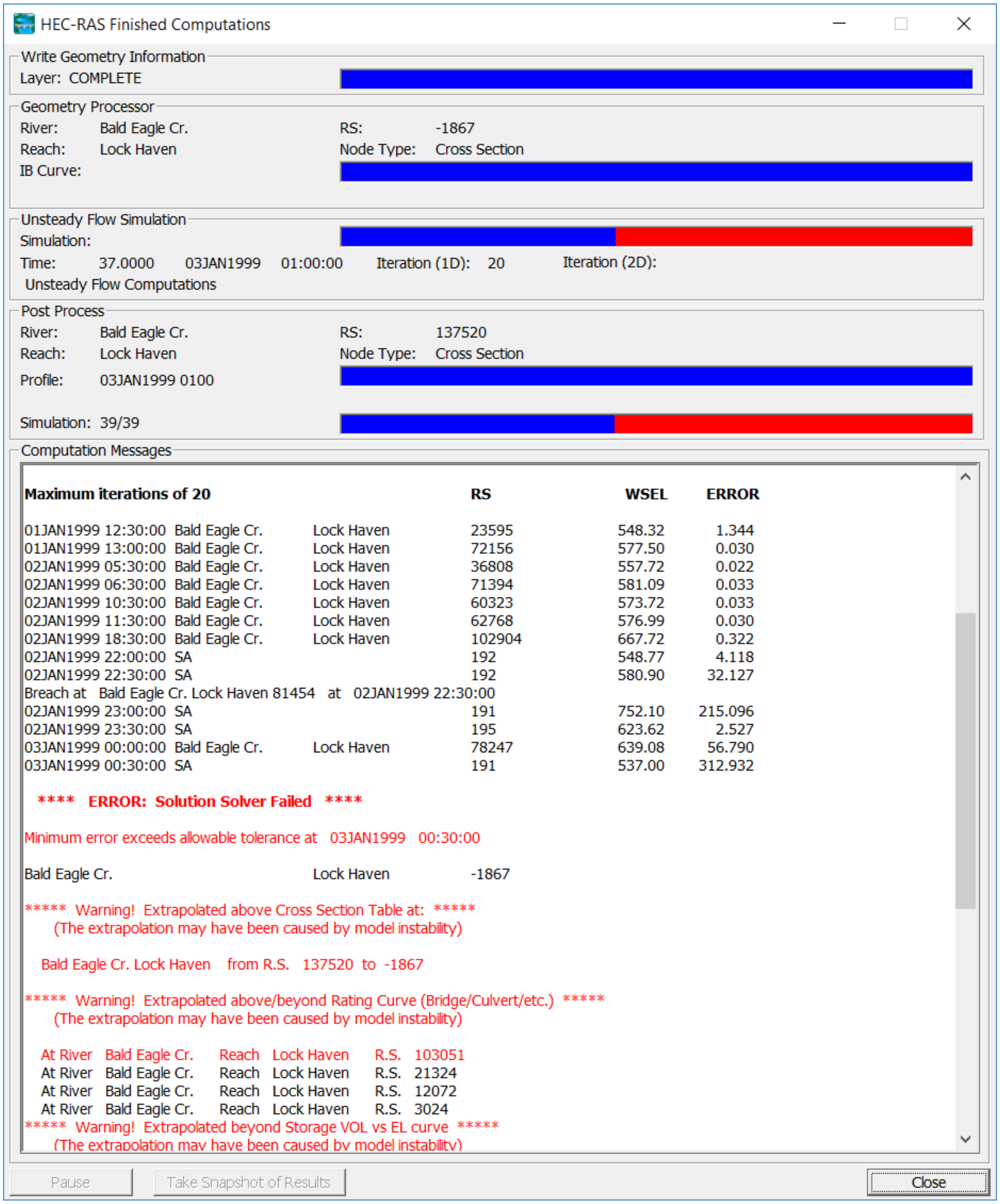


Figure 1. Computation Window with First Run of the Original Model

When a model blows up early in a run, it usually means that the initial conditions are bad, or that the computational time step is way off. A quick look at the computational time step shows that it was set to 30 minutes. 30 minutes is way too long for a dambreak model, and is generally too high for an unsteady flow model of a river system of this size.

In order to estimate an appropriate time step, the Courant condition should be considered. The first step is to look at the reach lengths between the cross sections. For this model, although the reach lengths vary quite a bit (approximately 300 to 1500 feet), the main channel reach lengths for most cross sections are in the 400 to 500 foot range. The next step is to estimate a wave speed for the flood wave. Since the model is not running at this point, an exact wave speed can not yet be computed, but an estimate can still be made. It is not uncommon to have velocities of over 20 ft/s downstream of the dam for the breached floodwave hydrograph. For now, let's assume 20 ft/s wave speed and 400 feet for the average channel distance (leaning towards the shorter lengths in the model). This should be checked again after the run is up and going. Solving the Courant condition would then yield the following for the time step:

$$C = \frac{V_w \, \Delta t}{\Delta X} \leq 1.0$$

Therefore:

$$\Delta t \leq \frac{\Delta X}{V_w} = \frac{400}{20} = 20 \sec$$

Trial 2. The computational interval was set to 20 seconds and the hydrograph output interval was set to 5 minutes, and the computations were run again. Additionally the Detailed output interval was also set to 5 minutes for now, so we can see more output. This time the program starts running and gets a little further, but not much. A large error occurred at river station 57790, at time step 01JAN1999 12:02:20. Shown in Figure 2 is the computation window for this run.

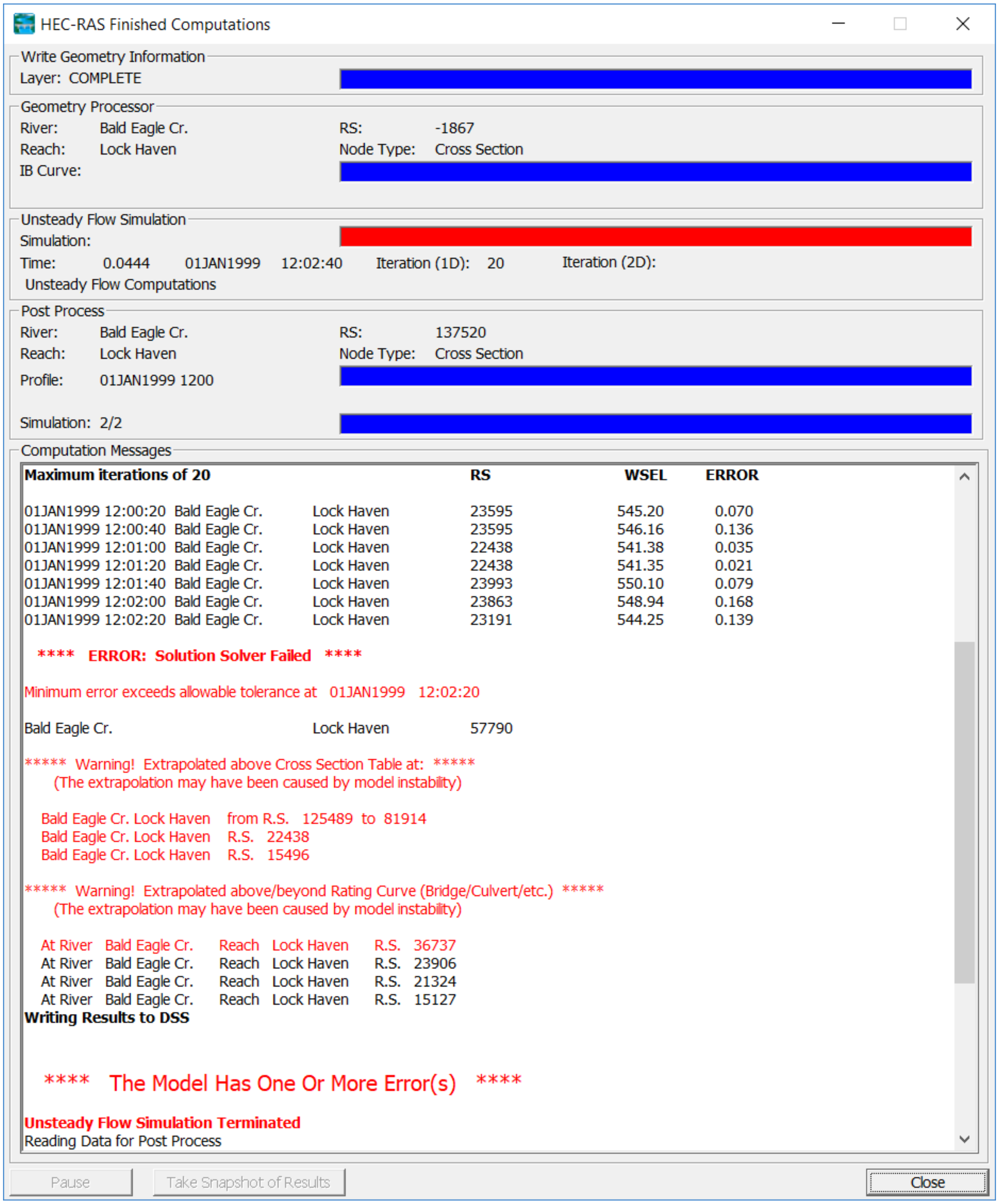


Figure 2. Computation Window For Trial Number 2.

Since the program is still blowing up very early in the run, and the computational time step is set very low, then there must be some kind of problem with the initial conditions. At this point, it is always helpful to look at the profile plot for the initial time step, as well as the computations for the initial backwater condition (which is what is being used for the initial conditions of this model). Shown in Figure 3 is the profile plot for the reach below the dam at the initial time step.

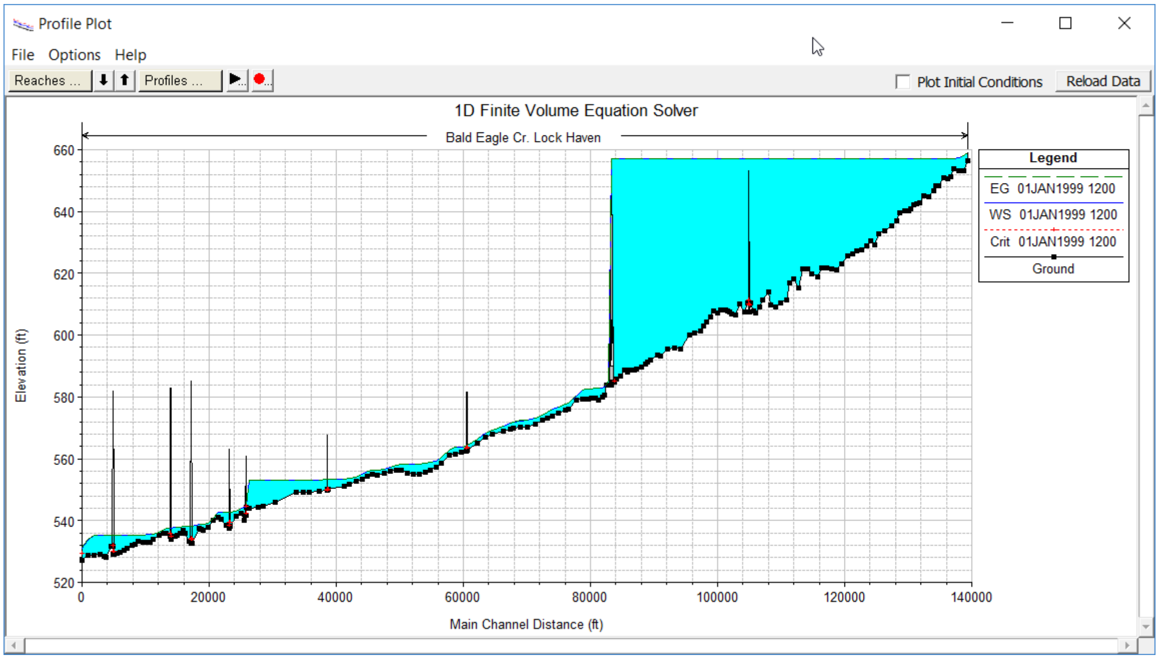


Figure 3. Profile PLot for Initial Conditions

As shown in Figure 3, there appears to be a large jump in the water surface downstream of where the program was getting its first large computational error (R.S. 23863). Upon closer review, there is a large spike in the water surface just upstream of the bridge at River station 23906. A zoomed in view is shown in Figure 4.

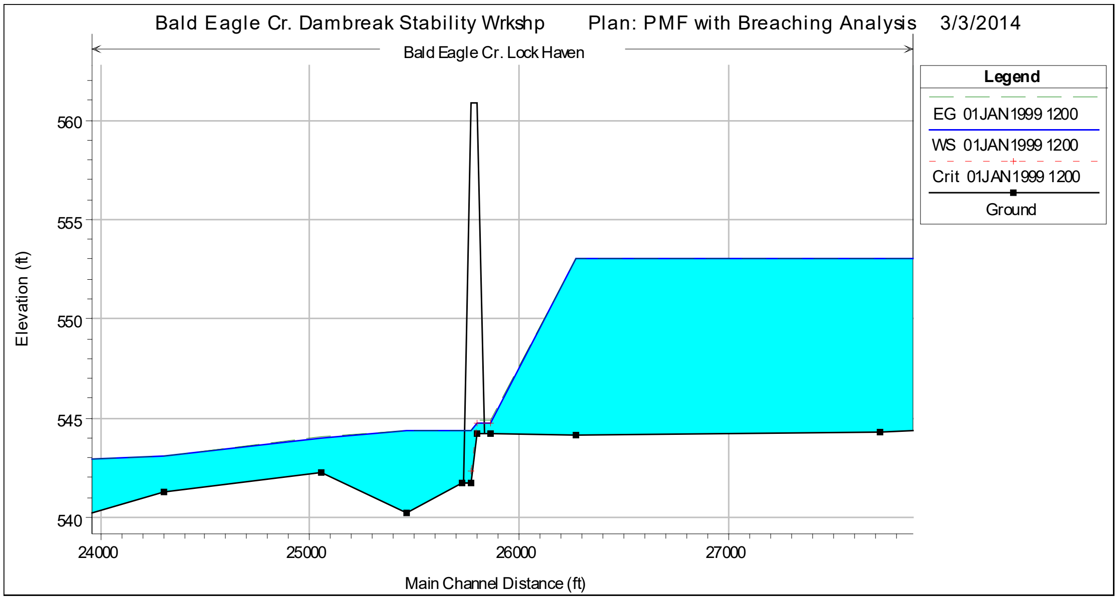


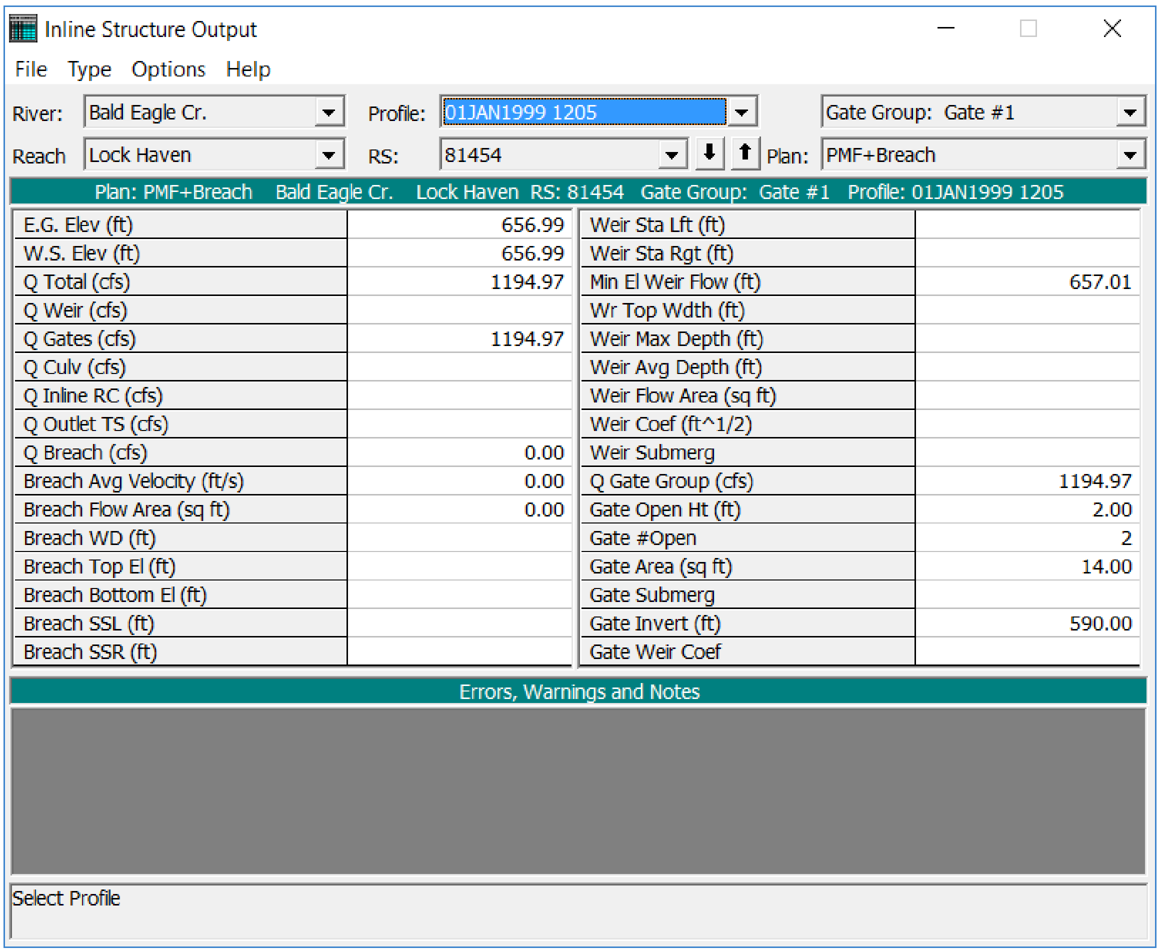
Figure 4. Zoomed in View of Bridge at R.S. 23906

As shown in Figure 4, the invert drops about 2 feet through the bridge. The backwater computations up to the bridge have a very low water surface. When it goes through the bridge, the flow is at critical depth (which is still a large rise from the downstream water surface). This was also confirmed by looking at the log output for the initial conditions. If it were very important to model low flows through the bridge, we would probably try to smooth out the invert profile through the bridge. (Another thing to remember, is that the cross section data for this model was gathered from digital data. The invert data at such low flows may not be reflecting the true invert of the model. This could be determined by field investigation.) However, for this dam break model, we can fix the problem by upping the base flow.

Investigation, of the flow set for the initial conditions computations, shows that the flow coming out of the dam was set to only 100 cfs for the initial backwater computations. This is a very low flow, and not really a good idea for this type of model. So, what should the initial flow be set to?

Before we answer that, it is important to consider how the hydraulic computations differ between unsteady and steady flow, especially at an inline structure. Unsteady flow will solve for both water surfaces and flows. At an inline structure modeling a large reservoir, the upstream water surface changes very slowly for normal discharges. In this case, unsteady flow is essentially using the gate settings and the upstream water surface [and the downstream water surface, if tailwater influenced] in order to compute a flow leaving the dam. Steady flow, on the other hand, only computes water surfaces from known flows. It will use the flow in the given part of the reach and the gate settings to compute the upstream water surface. However, the water surface upstream of the dam is frequently already known. When this happens (as is the case with this model), a "known" water surface should be used to set the initial water surface at the dam [that is, to override the steady flow answer]. The result of this is that the initial flow downstream of the dam (for the steady initial conditions) may not "match" the unsteady flow based on the upstream water surface and gate settings.

So with this in mind, we return to the question of what the initial flow should be. Further investigation shows that the initial gate openings were set to 2.0 feet with an initial water surface of 657 set for the Dam. By looking at the Detailed tabular output for the inline structure, we can see the flow leaving the dam. At the initial [unsteady flow] output time step (01JAN1999 1205), it shows a computed flow of 1195 cfs (Figure 5). If this is the correct gate settings, and the correct initial water surface for the pool elevation, then the initial conditions flow below the dam should be set to 1195 cfs.


Inline Structure Output

File Type Options Help

River: Bald Eagle Cr. Profile: 01JAN1999 1205 Gate Group: Gate #1

Reach Lock Haven RS: 81454 Plan: PMF+Breach

Plan: PMF+Breach Bald Eagle Cr. Lock Haven RS: 81454 Gate Group: Gate #1 Profile: 01JAN1999 1205

E.G. Elev (ft)	656.99	Weir Sta Lft (ft)	
W.S. Elev (ft)	656.99	Weir Sta Rgt (ft)	
Q Total (cfs)	1194.97	Min El Weir Flow (ft)	657.01
Q Weir (cfs)		Wr Top Wdth (ft)	
Q Gates (cfs)	1194.97	Weir Max Depth (ft)	
Q Culv (cfs)		Weir Avg Depth (ft)	
Q Inline RC (cfs)		Weir Flow Area (sq ft)	
Q Outlet TS (cfs)		Weir Coef (ft^1/2)	
Q Breach (cfs)	0.00	Weir Submerg	
Breach Avg Velocity (ft/s)	0.00	Q Gate Group (cfs)	1194.97
Breach Flow Area (sq ft)	0.00	Gate Open Ht (ft)	2.00
Breach WD (ft)		Gate #Open	2
Breach Top El (ft)		Gate Area (sq ft)	14.00
Breach Bottom El (ft)		Gate Submerg	
Breach SSL (ft)		Gate Invert (ft)	590.00
Breach SSR (ft)		Gate Weir Coef	

Errors, Warnings and Notes

Select Profile

Figure 5. Sayers Dam Detailed Output for Initial Time Step.

Trial 3. The initial conditions flow for below the dam was changed to 1195 cfs and the model was run again. This time the model ran all the way through and had no iteration problems until it got to time 18:07:40 of 02JAN1999. The largest errors in the run were only a few tenths of a foot, so no major numerical problems. However, at the end of the run, the computational program gave a "**WARNING!**" message that said "**Extrapolated above Cross Section Tables at:**". It then listed that every cross section table used in the model was overtopped. Figure 6, shows the message in the computational window.

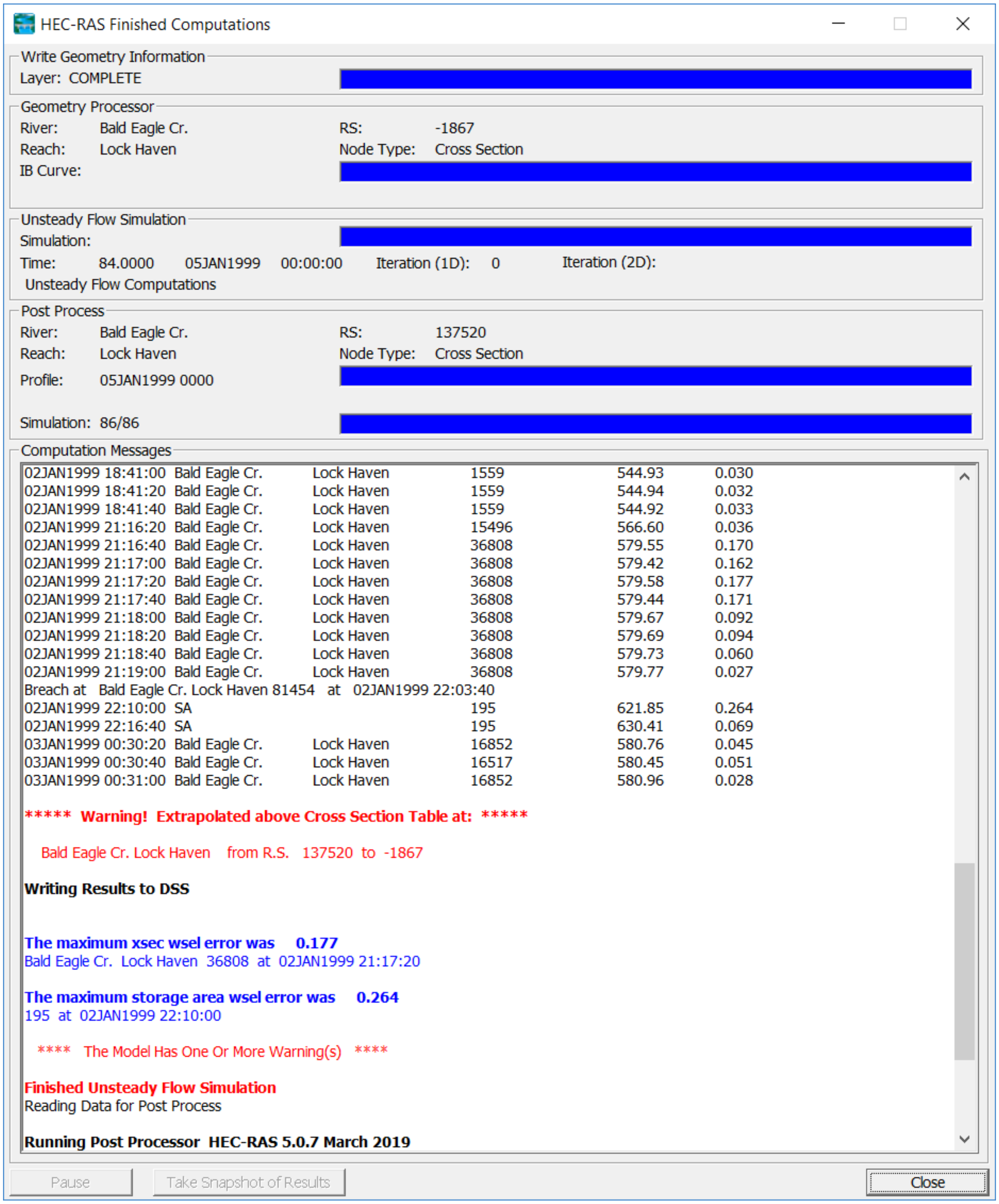
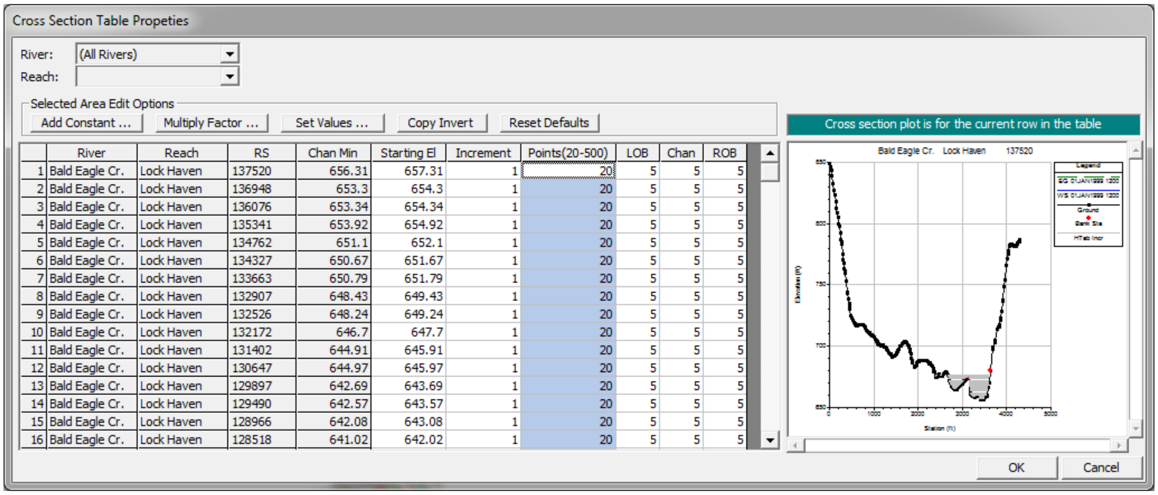


Figure 6. Computation Window With Warning Message at End of Run.

The unsteady flow program pre-processes the geometry into a series of curves (Cross sections) and family of curves (Bridges/Culverts). It is up to the user to set the table limits for all cross sections and structures to the correct limits. From the Geometric Editor the **HTab Param.** button was pressed to bring up the Hydraulic Tables editor for the cross sections (Figure 7).



	River	Reach	RS	Chan Min	Starting El	Increment	Points(20-500)	LOB	Chan	ROB
1	Bald Eagle Cr.	Lock Haven	137520	656.31	657.31	1	20	5	5	5
2	Bald Eagle Cr.	Lock Haven	136948	653.3	654.3	1	20	5	5	5
3	Bald Eagle Cr.	Lock Haven	136076	653.34	654.34	1	20	5	5	5
4	Bald Eagle Cr.	Lock Haven	135341	653.92	654.92	1	20	5	5	5
5	Bald Eagle Cr.	Lock Haven	134762	651.1	652.1	1	20	5	5	5
6	Bald Eagle Cr.	Lock Haven	134327	650.67	651.67	1	20	5	5	5
7	Bald Eagle Cr.	Lock Haven	133663	650.79	651.79	1	20	5	5	5
8	Bald Eagle Cr.	Lock Haven	132907	648.43	649.43	1	20	5	5	5
9	Bald Eagle Cr.	Lock Haven	132526	648.24	649.24	1	20	5	5	5
10	Bald Eagle Cr.	Lock Haven	132172	646.7	647.7	1	20	5	5	5
11	Bald Eagle Cr.	Lock Haven	131402	644.91	645.91	1	20	5	5	5
12	Bald Eagle Cr.	Lock Haven	130647	644.97	645.97	1	20	5	5	5
13	Bald Eagle Cr.	Lock Haven	129897	642.69	643.69	1	20	5	5	5
14	Bald Eagle Cr.	Lock Haven	129490	642.57	643.57	1	20	5	5	5
15	Bald Eagle Cr.	Lock Haven	128966	642.08	643.08	1	20	5	5	5
16	Bald Eagle Cr.	Lock Haven	128518	641.02	642.02	1	20	5	5	5

Figure 7. HTAB Editor for Cross Section Hydraulic Properties Tables.

Shown in Figure 7, the cross section tables were set for a 1 foot computation interval, with only 20 intervals. In other words, the tables were only set for a 20 foot depth. This table height is not adequate for the cross sections through the reservoir, which will have depths of close to 100 feet. Also, once the breach occurs, all of the downstream cross sections will experience very large depths, especially just below the dam.

The HEC-RAS program allows the user to have up to 500 values in the cross section properties tables. The maximum depth of water will occur just upstream of the dam. For this model the depth upstream of the dam got to a high of about 92 feet before the breach started to occur. In order to keep the property tables accurate, the increment of 1 foot was left alone, and the number of points in the tables was increased. To make it simple, we changed every cross section to use a 1 foot increment with 100 points in the tables. This could be refined but it is not necessary, having more points in the table does not slow the computations down, it only uses [slightly] more memory.

Trial 4. The cross section properties tables were set to 100 points, and the model was run again. The model ran all the way through again, but this time an error of 0.866 feet occurred at storage area 195, and another warning came up at the end of the run. This warning said "**Extrapolated above/beyond Rating Curve (Bridge/Culvert/etc.)**" for the Bridge at River Station 58673. Since the warning is a definite problem, that will be investigated first, before looking at Storage Area 195. Looking at the profile plot, the results show a 20 foot high wall of water building up behind the bridge at River Station 58673 (Figure 8).

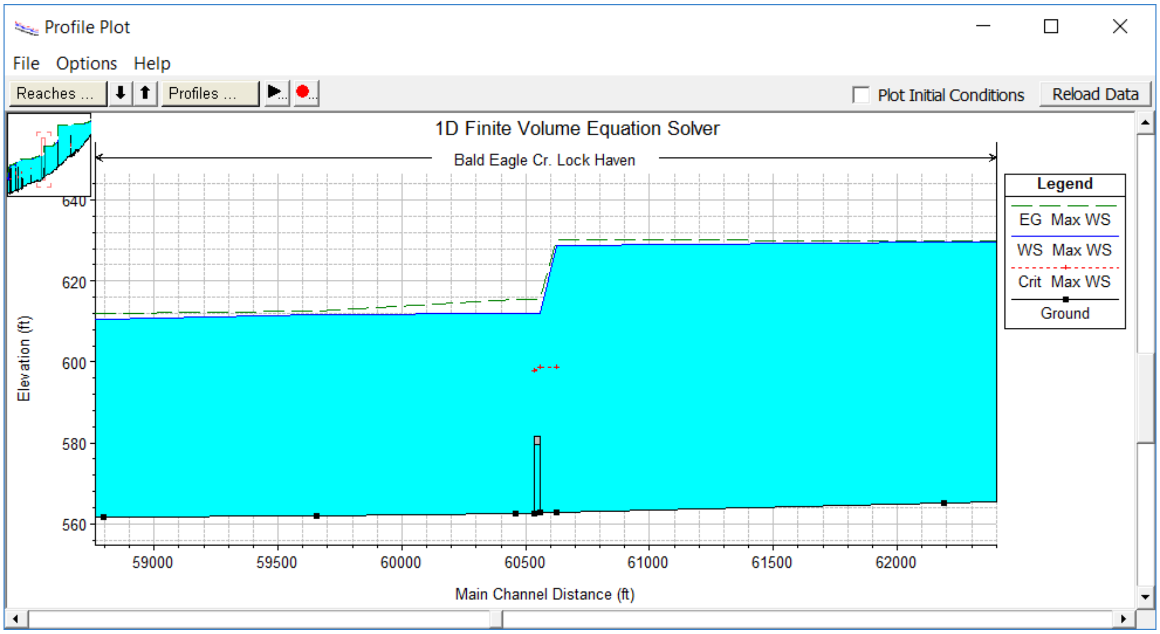


Figure 8. Wall of Water Building up at Bridge (R.S. 58673).

As shown in Figure 8, the water surface is well above the bridge, and there is no obvious reason for this wall of water being built up. Given the profile plot and the warning about the bridge, the properties tables for the bridge were investigated. Shown in Figure 9, is the Bridge Hydraulic Properties editor.

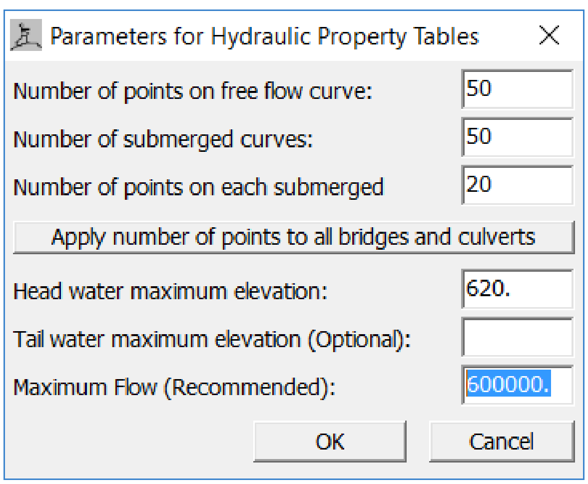


Figure 9. Hydraulic Properties Editor for Bridge at R.S. 58673.

The hydraulic properties limits were set for a **Head water maximum elevation** of 620, and a **Maximum flow** of 600,000 cfs. In reviewing the hydrograph output for around and just upstream of this structure, the flow rate is higher than 600,000 cfs upstream of this bridge. Figure 10 shows the progression of the flow hydrographs approaching and at the bridge. The Maximum flow setting of 600,000 cfs has caused the bridge to cap the flow at 600,000 cfs, thus acting like a dam at that location. This flow limitation for the bridge curves is causing the build up of the water surface behind the bridge.

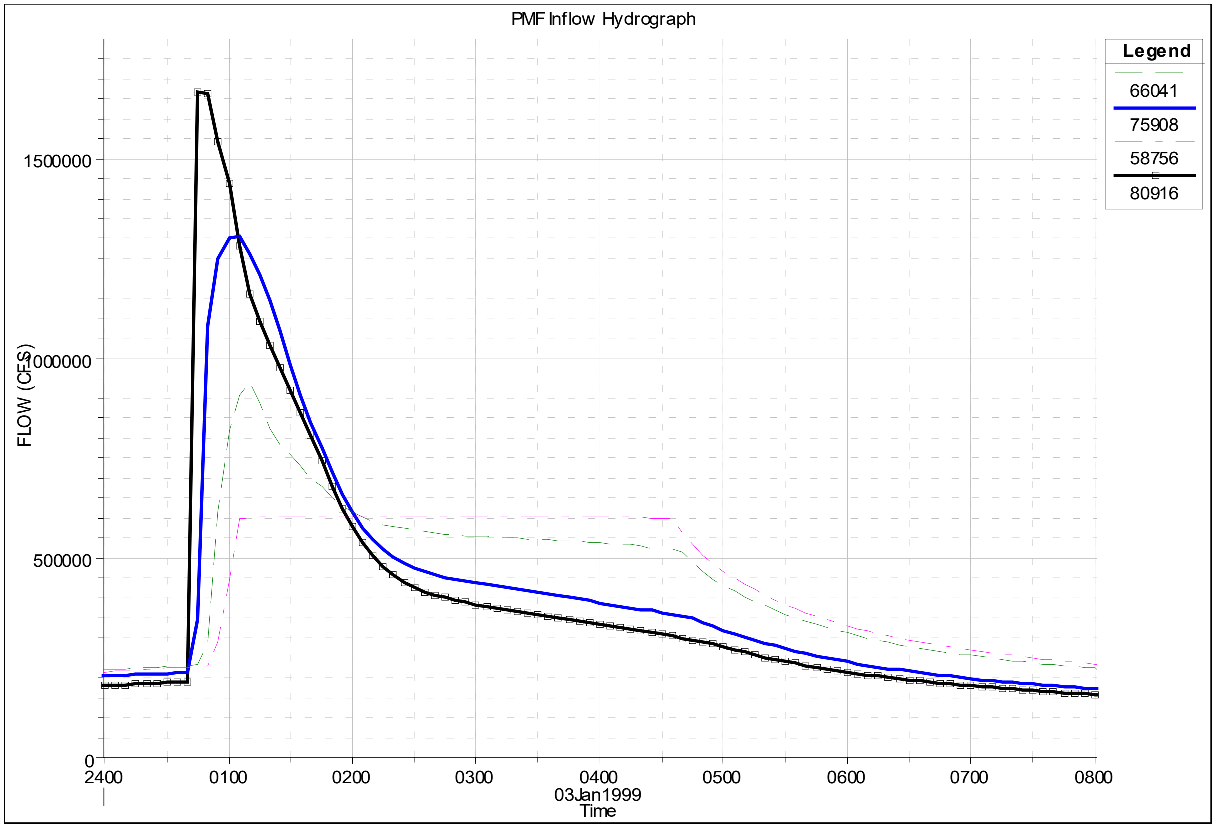


Figure 10. Flow Hydrographs from Dam to Bridge at R.S. 58673.

Shown in Figure 10, the flow at river station 66041 is around 950,000 cfs, and the flow at the bridge is 600,000 cfs. It is obvious that the table limits are causing this problem.

Trial 5. The table limits for the bridge at river station 58673 were change to have a maximum flow limit of 1,000,000 cfs, and the model was rerun. Again the model ran all the way through, but this time there were no warning messages at the end of the run. **However, there was still some significant numerical errors at Storage Area 195.** This error occurred at time 00:40:00 on 03JAN1999. A plot of the Stage and Net inflow hydrograph for this storage area is shown in Figure 11.

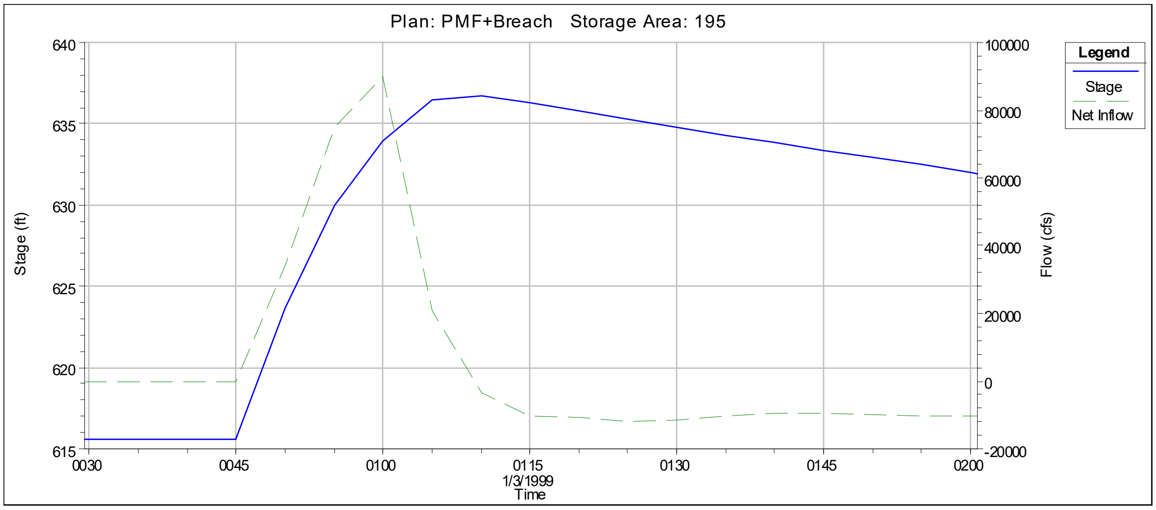
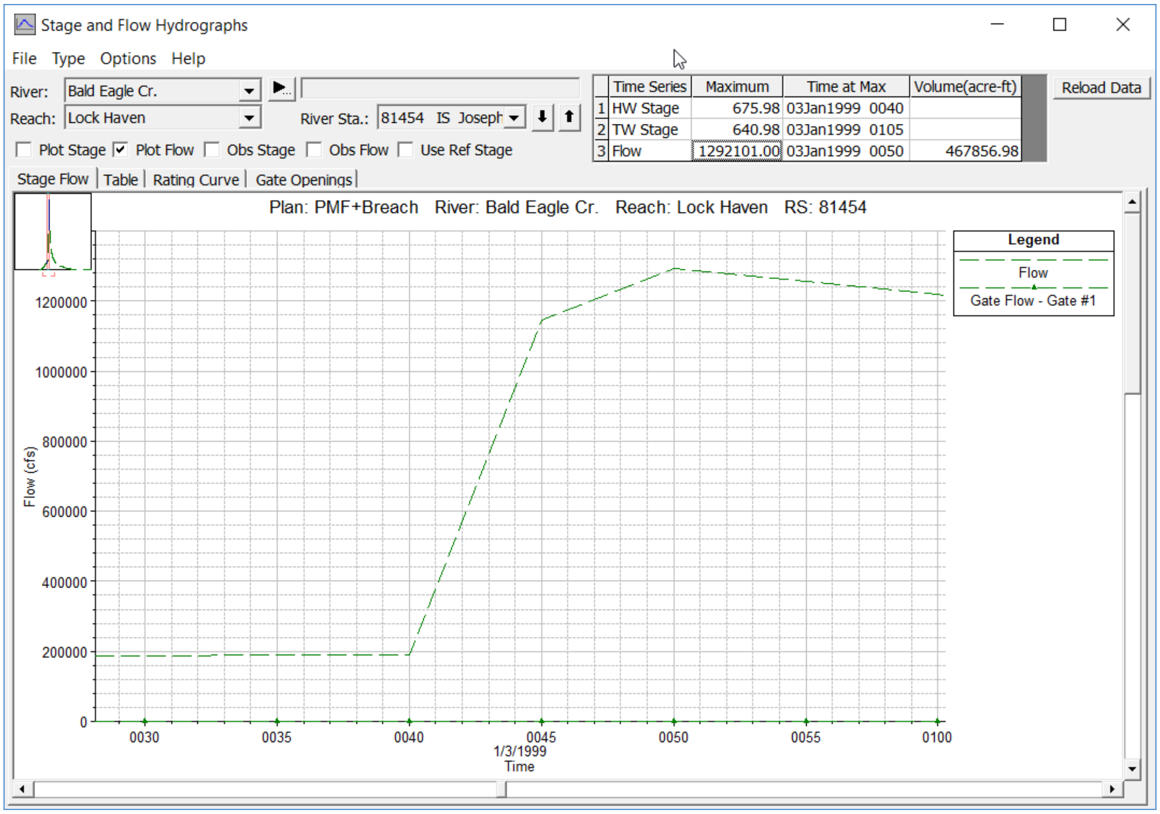


Figure 11. Stage and Net Inflow Hydrographs for Storage Area 195.

As shown in Figure 11, there is no obvious stability problem with the results for storage area 195 at the time of 00:49:40 on the 3rd of January. However, it is obvious that at time 00:45:00 the hydrograph begins to rise very rapidly. This very rapid rise, from no flow going into the storage area, to a peak flow of around 90,000 cfs is probably causing the numerical error at the beginning of the rise. This storage area is just downstream of the Dam, and it is connected to the main river with a lateral structure. The storage area represents a large tributary, and it is modeled this way to capture the backwater, and loss of storage water, that will occur when the dam break floodwave backs up water into the tributary.

The basic problem is that the dam break flood wave is rising so fast that it is causing a numerical error in water surface calculation of storage area 195. In general, possible solutions to this problem are to either use a smaller computation interval, or to try and reduce how rapidly the flow is coming into the storage area through the lateral weir. The current computational time step is set at 20 seconds. This seems reasonable but should be investigated closer for the rising dam break floodwave. A plot of the Dambreak hydrograph coming out of the dam shows the hydrograph going from its base flow to its peak flow (1.3 million cfs) in 5 to 10 minutes (Figure 12). Remembering that the hydrograph output interval is set to 5 minutes, this means that the actual peak could be occurring in less than 10 minutes. This seems very quick for a breach hydrograph for this dam. Therefore the breach parameters were investigated.



	Time Series	Maximum	Time at Max	Volume(acre-ft)
1	HW Stage	675.98	03Jan1999 0040	
2	TW Stage	640.98	03Jan1999 0105	
3	Flow	1292101.00	03Jan1999 0050	467856.98

Figure 12. Breah Outflow Hydrograph from Dam.

Looking at the breach parameters editor, as shown in Figure 13, reveals that the breach Full Formation Time was set to 0.08 hours (4.8 minutes). This is much quicker than what was computed from our Breach Parameters Workshop, which yielded a breach time of 0.8 hours (48 minutes) for the Von Thun and Gilette method, which was the selected parameter set.

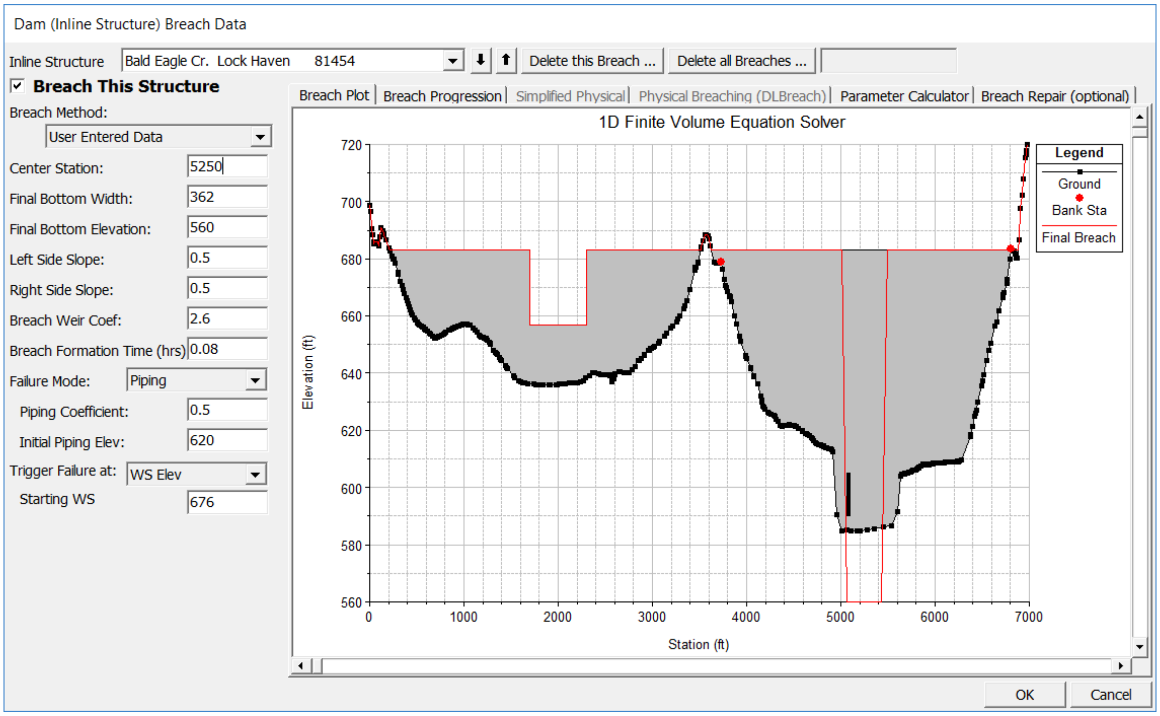


Figure 13. Dam Breach Data Editor for Sayers Dam.

Upon further investigation it is also noted that the **Bottom Elevation** of the dam was set to 560, which is way below the invert elevations of the cross sections just upstream and downstream of the dam. The invert of the breach should be set at 585 ft, which is right at the invert of the cross sections. HEC-RAS will allow the user to set an elevation of the invert below the cross sections, but this could lead to model instability in the area of the dam itself.

Trial 6. The breach time was set to 0.8 hours and the bottom elevation was set to 585 ft., and the model was rerun. The review of the model output shows that everything looks ok, except for the water surface at the downstream end of the model. As the floodway rises downstream, the water surface at the very end of the model stays very low, causing a very steep sloping water surface at the downstream boundary (Figure 16).

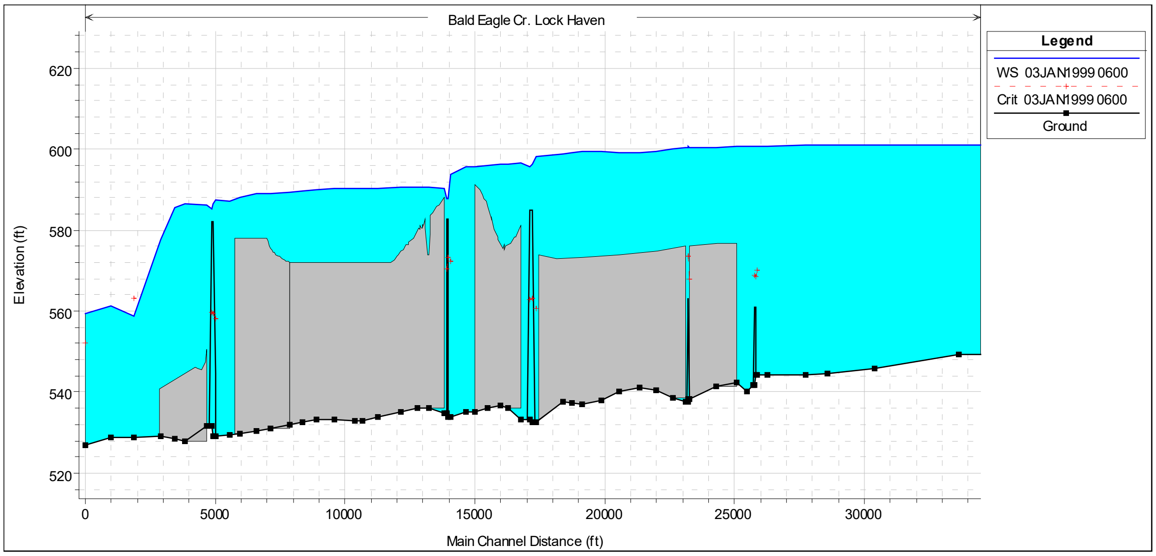


Figure 14. Water Surface Profile Plot.

Based on the water surface profile plot in Figure 16, there appears to be something wrong with the downstream boundary condition. Review of the boundary condition data shows that a Normal Depth boundary condition has been applied with a slope of 0.003 ft/ft. To verify that this slope is appropriate, the downstream invert profile slope is checked from the profile plot using the measuring tool (control key plus clicking the mouse). This shows that the downstream slope should be 0.0003 ft/ft, so it appears that a zero was not entered when entering the slope. The slope was changed and the model was re-run.

Trial 7 - Final Results. Reviewing the model again shows good results for the entire run. A profile plot of the maximum water surface is shown in Figure 17.

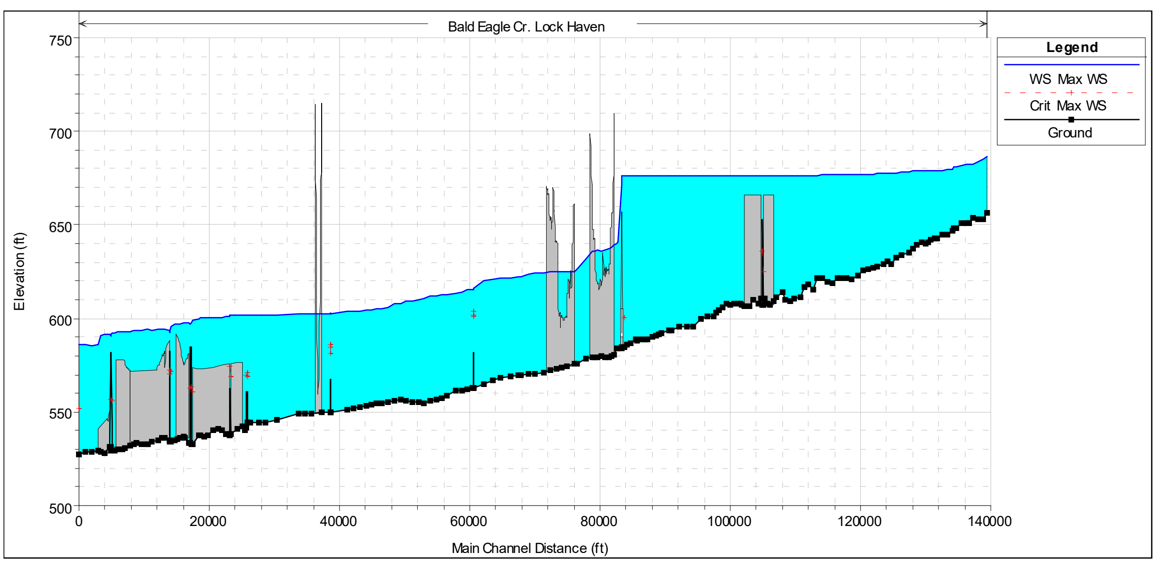


Figure 15. Final Water Surface Profile Plot of Corrected Model.