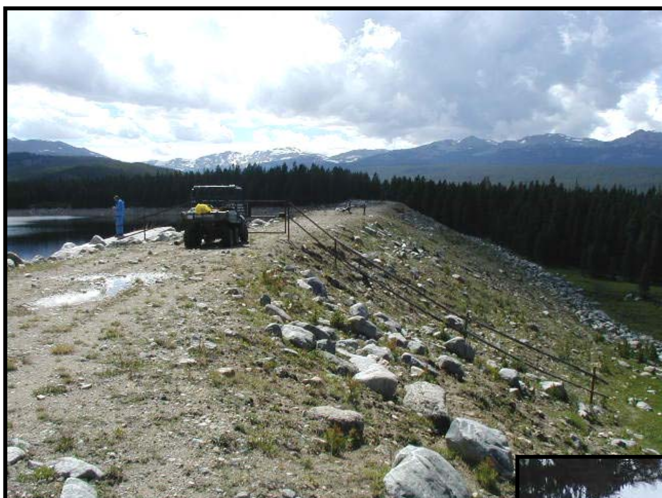
Willow Park Reservoir Dam Breach Analysis

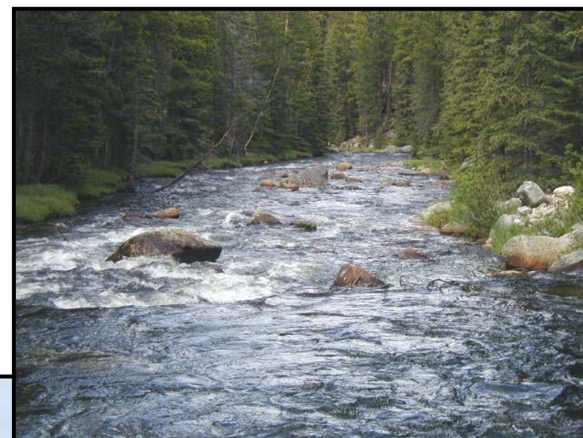
Johnson and Sheridan Counties, Wyoming

July 2004



Willow Park Embankment

South Piney Creek



Story

Emergency Spillway

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Lakewood, Colorado

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Willow Park Reservoir Dam Breach Analysis

Job Number: WY0301
Short Job Description: Willow Park dam breach analysis.
Location: Johnson County, Wyoming near Story on South Piney, Piney, and Clear Creeks.

Summary: Predictions have been made of the probable extent and timing of out-of-bank flow resulting from a catastrophic breach of Willow Park Reservoir. This report details the dam breach analysis performed on the reservoir for the purpose of an emergency action plan.

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NRCS

The Natural Resources Conservation Service provides leadership in a partnership effort to help people conserve, maintain, and improve our natural resources and environment.

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INTRODUCTION

This report details the methods and results of a dam breach analysis performed on the Willow Park Reservoir of Johnson County, Wyoming. The analysis consists of breach hydrograph development and hydrograph routing through the stream valleys, agricultural lands, and communities below the structure. This report is intended for use by the Willow Park Reservoir Company, the impacted communities, and emergency response teams for the development of an emergency action plan.

The Willow Park reservoir (Figures 1 and 2) is located on South Piney Creek at an elevation of 8600 feet in the Bighorn Mountains above the town of Story. Average precipitation within the reservoir's 33.9 square mile watershed varies from 25 to 39 inches, according to PRISM. The embankment dam has a maximum height of about 54.5 feet, with a crest elevation of 8625.5 feet and associated storage of about 6260 ac-ft. At the emergency spillway crest elevation of 8619.5 feet the associated reservoir storage is 5123 ac-ft. These volumes do not account for accumulated sediment since dam construction in the late 1950's.

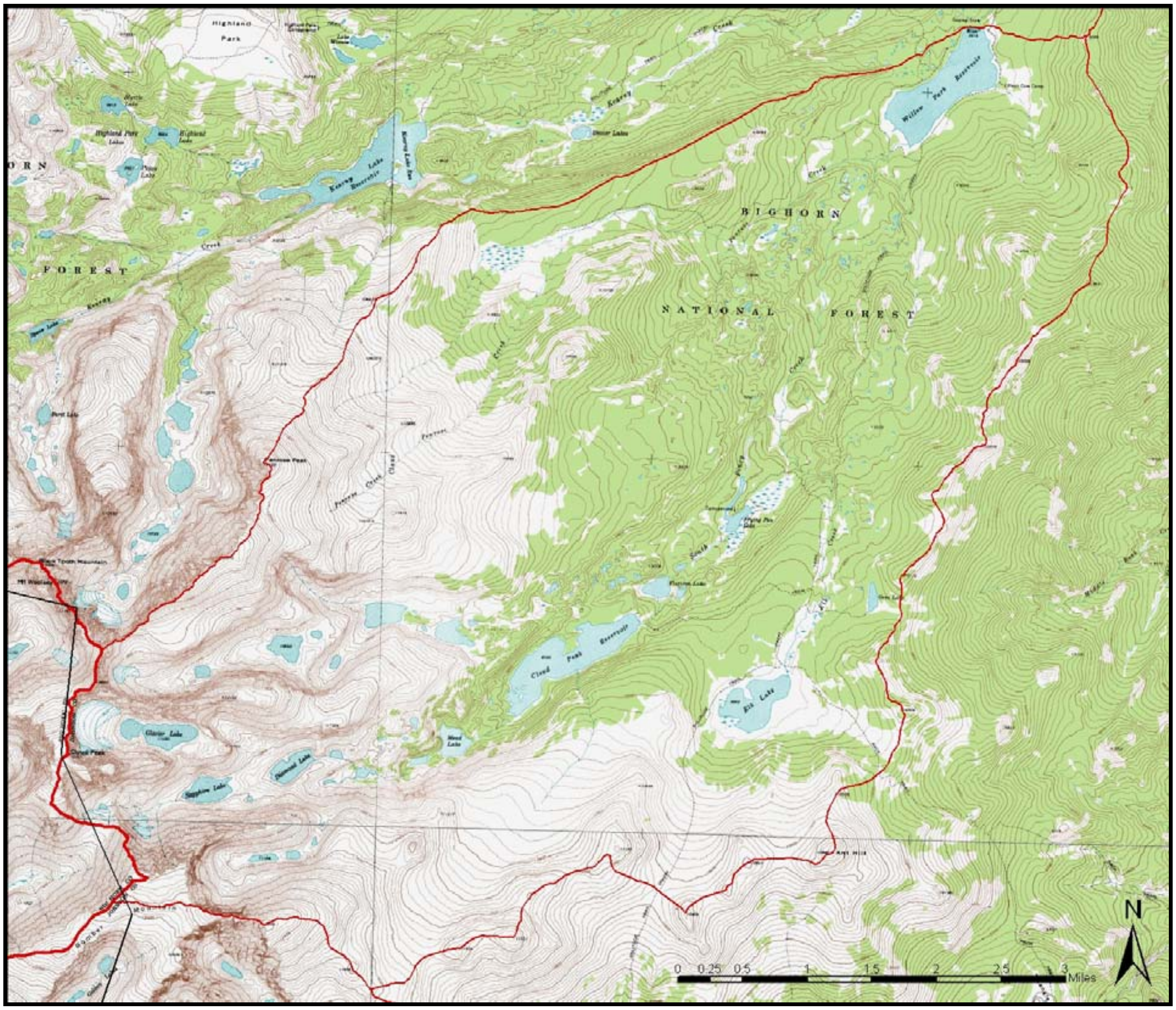


Figure 1: Reservoir watershed.

This analysis is sufficient for the development of an emergency action plan for the catastrophic breach of the Willow Park embankment. However, due to the assumptions regarding the mechanism of failure as well as limitations in the understanding of and the ability to model unsteady flow dynamics of the large, severe and abrupt debris-saturated flood wave that would result from an embankment failure, these modeling results are approximate. The nature and limitations of the predictions provided in this report must be kept in mind when using these results.

This report details the methodology used to determine the likely effects of a catastrophic breach. The primary sections include an Introduction, Breach Hydrograph Development, Hydrograph Routing, and Likely Inundation Extent and Timing. In addition, most likely inundation maps, modeling output tables, and stream gage flood frequency computations are included in three appendices. **For analysis results, see the Likely Inundation Extent and Timing section and the Maximum Likely Inundation mapping of Appendix A.**

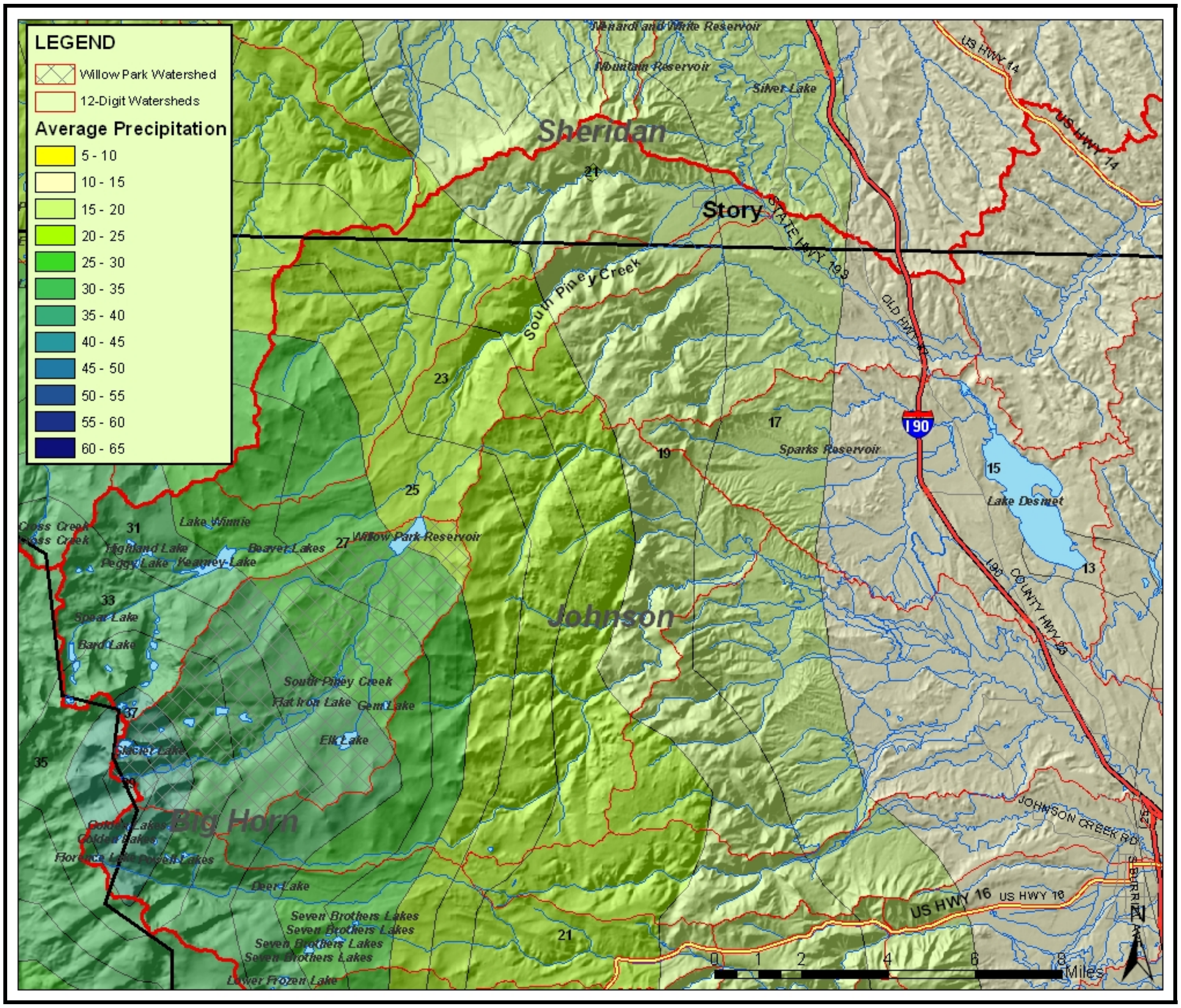


Figure 2: Region of analysis, mountainous portion. Shaded relief, average precipitation (PRISM) estimates, reservoirs and lakes, and 12-digit watersheds are shown. The Willow Park watershed is shown cross-hatched.

BREACH HYDROGRAPH DEVELOPMENT

As mentioned in Froehlich 1995, the International Commission on Large Dams reports that roughly a third of embankment dam failures are caused by overtopping due to inadequate spillway capacity; another third result from piping failure; and the last third result from embankment sliding, embankment settlement, and inadequate wave protection. An embankment overtopping failure is modeled in this analysis, which is the most likely worst-case failure type in this situation. (See Figures 3 and 4 for photos of the embankment.) As was noted in the Army Corp of Engineers Phase I Inspection Report (1978), an overtopping event is not out of the realm of possibility - the Army Corps found that the emergency spillway (Figure 5) can pass 75 to 40 percent of the probable maximum flood (PMF) without overtopping (the range was generated by both accounting and not accounting for the numerous lakes found in the watershed). Current NRCS design procedures (Soil Conservation Service 1985) mandate the passage of the full PMF without overtopping for a high hazard (C) dam such as Willow Park. This is the common procedure throughout the world. Hence, the emergency spillway is undersized and overtopping is a possibility.

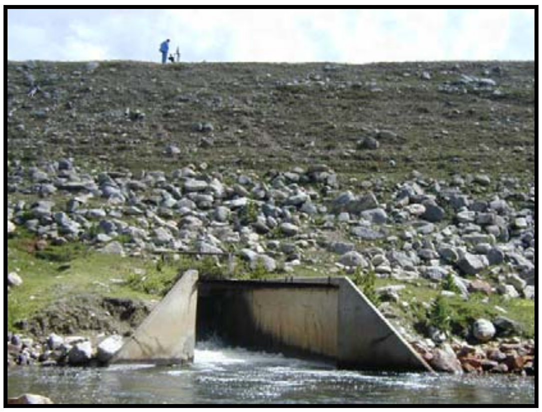


Figure 3: Principal spillway outflow.

Figure 4: Downstream embankment face.

Figure 5: Emergency spillway, Willow Park Reservoir.

Published (peer reviewed) regression equations were relied upon in this analysis for predicting the breach hydrograph characteristics. A cubed power relationship was used to provide both the ascending and descending portions of the hydrograph. The hydrograph volume was set equal to the reservoir storage with the water surface elevation set to the crest of the embankment. Peak flow was generated using Froehlich's peak flow equation (Froehlich 1995). The initial flow was set as the emergency spillway maximum capacity and the time-to-peak was estimated using another Froehlich equation (Wahl 1998). Peak flow estimates were verified using the NRCS equations in TR-60 (amendment to SCS 1985).

The regression equation developed by Dave Froehlich (Froehlich, 1995) was used to estimate the peak flow to be expected by a breach of Willow Park Dam. This well-documented peer reviewed equation, which was developed from 22 embankment dam failures and has a R^2 of 0.934, is

$$Q_p = 0.607 V_w^{0.295} H_w^{1.24} \tag{1}$$

where V_w is the reservoir volume at time of failure (m^3) and H_w is the height of water in the reservoir at the time of failure above the final bottom elevation of the breach (m). With an embankment height of approximately 54.5 ft (16.6 m) and storage at crest of approximately 6260 ac-ft (7,721,000 m^3), a peak discharge of 75,200 cfs is estimated. It should be noted that the storage volume is as noted on the design plans and excludes the sedimentation that has occurred since reservoir construction in 1957.

This peak flow estimate was verified using the lesser-documented equations developed by NRCS. In accordance with the NRCS TR-60 1990 addendum, the criteria for peak flow prediction for an embankment height less than 103 ft is

$$Q_{\max} = 1100 B_r^{1.35} \tag{2}$$

where

$$B_r = \frac{V_s H_w}{A} \tag{3}$$

But the peak flow is not to be less than

$$Q_{\max} = 3.2 H_w^{2.5} \tag{4}$$

and need not exceed

$$Q_{\max} = 65 H_w^{1.85} \tag{5}$$

where V_s is the reservoir storage at the time of failure (ac-ft), H_w is depth of water at dam at time of failure (ft), and A is the cross-sectional area of the embankment at the location of the breach (ft^2). With an embankment cross-sectional area of 31,810 ft^2, results are provided in Table 1.

Table 1: Breach hydrograph characteristics.

Description	Reservoir WSEL (ft)	Reservoir Volume* (ac-ft)	Froehlich Peak (cfs)	NRCS		
				Peak, Eq. 2 (cfs)	Peak, Eq. 4 (cfs)	Peak, Eq. 5 (cfs)
at Embankment Crest	8625.5	6,260	75,200	39,300	70,200	106,000

*estimated, due to limits of reservoir capacity table

A time-to-peak estimate was created using Froehlich's method as described in Wahl, 1998. This method uses the equation

$$t_f = 0.00254 V_w^{0.53} h_b^{-0.90} \quad (6)$$

where t_f is the breach formation time (hours), V_w is the reservoir volume at time of failure (m^3), and h_b is the height of breach (m). This method provides a time-to-peak estimate of 0.91 hours. A 55 minute time-to-peak estimate was used.

A cubed power relationship was implemented to generate the breach hydrograph, to better approximate an actual hydrograph than a triangular hydrograph.

This dam breach analysis is essentially a "sunny-day breach", with no adjacent watersheds (to the downstream reaches) contributing flow to the stream. The hydraulic model used in this analysis does require flow initiation - 8650 cfs, the approximate capacity of the emergency spillway, was used as this initial condition. Hence the model discussed below routes a breach from the dam with the spillway initially at capacity and the breach occurring with an initial reservoir water surface at the crest of the embankment.

HYDROGRAPH ROUTING

The Hydrologic Engineering Center – River Analysis System (HEC-RAS) one-dimensional computer program, by the U.S. Army Corps of Engineers, was used to route the floodwave from the dam breach through the South Piney Canyon, through Story, and through the Piney and Clear creek river valleys downstream of Story. HEC-RAS version 3.11 was used in this analysis.

Computation Methodology

To support the basis of the modeling used in this dam breach analysis and to discourage a "black box" mentality, the basic equations used in these computations are briefly presented.

The physical laws that govern unsteady flow modeling, as presented in the HEC-RAS Hydraulic Reference Manual (Brunner and Goodwell, 2002), are conservation of mass (the continuity equation) and conservation of momentum. The general continuity equation (not separately written for both the channel and floodplain) is:

$$\frac{\partial A}{\partial t} + \frac{\partial S}{\partial t} + \frac{\partial Q}{\partial x} - q_1 = 0$$

Where: ∂ = partial differential.
A = cross-sectional area.
t = time.
S = storage from non conveying portions of cross section.
Q = flow.
x = distance along the channel.
q_l = lateral inflow per unit distance.

The momentum equation can be stated as "the net rate of momentum entering the volume (momentum flux) plus the sum of all external forces acting on the volume be equal to the rate of accumulation of momentum" (Brunner and Goodwell, 2002). In differential form, it is:

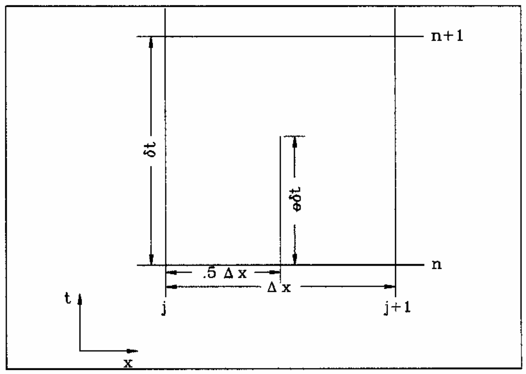
$$\frac{\partial Q}{\partial t} + \frac{\partial QV}{\partial x} + gA\left(\frac{\partial z}{\partial x} + S_f\right) = 0$$

$$S_f = \frac{Q|Q|n^2}{2.208 R^{4/3} A^2}$$

Where: V = velocity
g = acceleration due to gravity.
$\frac{\partial z}{\partial x}$ = water surface slope.
S_f = friction slope.
n = Manning's roughness estimate.
R = hydraulic radius = area/wetted perimeter.

The most successful and accepted procedure for approximating solutions to the non-linear unsteady flow equations is with a four-point implicit solution scheme, also known as a box scheme (Brunner and Goodwell, 2002). The HEC-RAS Hydraulic Reference Manual describes this as follows:

> *Under this scheme, space derivatives and function values are evaluated at an interior point, $(n+\theta)\Delta t$. Thus values at $(n+1)\Delta t$ enter into all terms in the equations. For a reach of a river, a system of simultaneous equations results. The simultaneous solution is an important aspect of this scheme because it allows information from the entire reach to influence the solution at any one point Consequently, the time step can be significantly larger than with explicit numerical schemes.*



[Typical finite difference cell used in HEC-RAS computations (from Brunner and Goodwell, 2002).]

The general implicit finite difference forms are as follows:

The time derivative is approximated as: $\frac{\partial f}{\partial t} \approx \frac{\Delta f}{\Delta t} = \frac{0.5(\Delta f_{j+1} + \Delta f_j)}{\Delta t}$

The space derivative is approximated as: $\frac{\partial f}{\partial x} \approx \frac{\Delta f}{\Delta x} = \frac{(f_{j+1} - f_j) + \theta(\Delta f_{j+1} - \Delta f_j)}{\Delta x}$

The function value is: $f \approx \overline{f} = 0.5(f_j + f_{j+1}) + 0.5\theta(\Delta f_j + \Delta f_{j+1})$

Where: Δ = difference or change in.

Using this methodology, the finite difference form of the continuity equation used by HEC-RAS (which separates channel and floodplain flow) is:

$$\Delta Q + \frac{\Delta A_c}{\Delta t}\Delta x_c + \frac{\Delta A_f}{\Delta t}\Delta x_f + \frac{\Delta S}{\Delta t}\Delta x_f - \overline{Q_l} = 0$$

Where: c = channel.
f = floodplain.
$\overline{Q_l}$ = average lateral inflow.

Assuming a horizontal water surface across the cross section and perpendicular flow to the plane of the cross section, the finite difference form of the momentum equation is:

$$\frac{\Delta(Q_c \Delta x_c + Q_f \Delta x_f)}{\Delta t \Delta x_e} + \frac{\Delta(\beta VQ)}{\Delta x_e} + g\overline{A}\left(\frac{\Delta z}{\Delta x_e} + \overline{S_f} + \overline{S_h}\right) = \xi \frac{Q_l V_l}{\Delta x_e}$$

Where: Δx_e = equivalent flow path

$\Delta(\beta VQ) = \Delta(V_c Q_c) + \Delta(V_f Q_f)$

S_f = frictional slope for the entire cross section.

S_h = local frictional slope, from bridge piers, navigation dams, cofferdams, ect.

Q_l = lateral inflow.

V_l = average velocity of lateral inflow.

ξ = fraction of momentum entering a receiving stream.

If the implicit finite difference solution scheme is applied directly to these non-linear equations, a series of non-linear algebraic equations result. To avoid the resulting slow and unstable iteration solution schemes, these equations are linearized for their use in HEC-RAS (Brunner and Goodwell, 2002).

For a more comprehensive presentation of the solution equations and techniques used in HEC-RAS, please see the HEC-RAS Hydraulic Reference Manual (Brunner and Goodwell, 2002).

Roughness Estimates for Steep Reaches

Dam breaches and other flow events of such extreme intensity can have profound effects upon channel and valley morphology. During such extreme flows the steep wooded stream channels and floodplains prevalent in mountainous areas can be stripped of woody material and alluvial beds may be scoured and mobilized. This may produce a cascading debris flow. A debris flow is a type of mudflow with a prevalence of large material (larger than sand-sized) mixed with fines and water.

In unsteady modeling, the typical methods and guides for predicting Manning roughness (*n*) values by inspection, such as those provided in Chow (1959), Arcement & Schneider (1989), Brunner & Goodell (2002), though sufficient for many situations, are oftentimes not adequate for high gradient streams (Trieste 1994). This is especially the case during extreme events, since current conditions likely don't reflect the prediction conditions. The energy loss in hydraulic jumps, turbulence, and obstructions are not adequately incorporated in these *n* estimates. The great deal of bed material and debris liberation and movement that is expected during very high flows further increases the uncertainty in *n* since existing flow conditions and roughness are not equivalent to extreme flow conditions and roughness. The very high Froude numbers and velocities often computed during modeling of high flows on steep gradient streams indicate the problem with the roughness estimates.

Breach Case Study

The catastrophic breach of the Lawn Lake embankment dam, a 26 ft high embankment dam located in Rocky Mountain National Park, illustrates the problems often encountered in modeling unsteady flow from breaches in mountainous terrain. As described in Jarrett and Costa (1984), the catastrophic breach occurred on July 15, 1982 from a piping failure. The failure released 674 ac-ft of water, with an estimated time-to-peak flow of 10 minutes and an estimated peak discharge of 18,000 ft^3/s. The breach wave occurred over slopes from 5 to 25 percent in the canyon of the Roaring River, 0.7 percent in Horseshoe Park, and up to 8 percent in the Fall River above the town of Estes Park and the Big Thompson River. The breach created a flood wave in the Roaring River that was characterized by eyewitnesses as a "wall of water" 20 to 30 ft high. The leading edge of the wave was not likely to have been a vertical wall of water but the peak was likely to have been very close to the wave front, which would have been accentuated by the mass of entrained debris. Besides the mass of alluvium mobilized on the Roaring River reach, the flood wave consisted of a mass of vegetation mobilized from the valley over a wide swath, from 70 to 500 ft wide. The leading edge, due to all of the debris, moved much slower than expected for a steep channel. Flow likely alternated from supercritical for short reaches to subcritical behind temporary debris dams that formed, and again as supercritical flow for a short reach as the dam breached and until the next dam formed (Jarrett and Costa, 1984).

An unsteady flow model was developed by Jarrett and Costa (1984) for the breach analysis, in an attempt to match the model to actual conditions. The model used an initial *n* estimate of 0.125 and a calibrated value 0.200. Velocity estimates ranged from 3.3 to 12.6 ft/s. Maximum flow depths ranged from 6.4 to 23.8 ft and maximum flow widths ranged from 97 to 1112 ft. Flood peaks from the Lawn Lake dam failure, depending upon the reach, were 2.1 to 30 times the 500-year flood magnitude (Jarrett and Costa, 1984).

The geomorphic effects of this breach were significant. On the Roaring River channels were widened tens of feet, locally scouring 5 to 50 ft with the valley alternately scoured and filled, depending upon valley slope. At the mouth of the Roaring Fork, at Horseshoe Park, a 365,000 cubic yard alluvial fan was deposited. The largest boulder known to be moved during the event is 14x17.5x21 ft (Jarrett and Costa, 1984).

According to Jarrett and Costa (1984), the Lawn Lake breach analysis indicates that to more appropriately model a breach flow through steep, moveable bed, debris saturated stream valleys, Manning *n* estimates need to reflect a flow with entrained debris, with bed scouring and deposition, instead of existing conditions. This was the reason for the need to calibrate *n* to the value of 0.200.

Conclusions regarding the appropriateness of modeling such flow events as supercritical have been reached in other breaches in steep terrain. For example, a hydraulic analysis performed on the Quail Creek Dike Failure flood in Utah, which flowed for the first 1.6 km (1 mile) through a steep (0.032 m/m) slope drainage, showed that the model depths could not match the actual field depths unless the reach was modeled as being entirely subcritical (Trieste 1992).

Supercritical vs. Subcritical Flows in Natural Channels

Analysts often model high flows on steep reaches as supercritical flow. This assumption can be valid for rigid boundary channels, such as concrete or natural bedrock channels, but is a questionable practice for the natural channels typically modeled (Trieste 1994).

For cobble and boulder bed high-gradient streams with extreme flows, Jarrett (1984) suggests that a limiting assumption of critical depth in subsequent hydraulic analyses appears to be reasonable. Trieste (1994) suggests that modeling supercritical flow for long reaches within the National Weather Service's DAMBRK (Freud 1988) or its successor FLDWAV (Fread and Lewis, 1998) may be invalid except for possibly bedrock channels. For steep boulder and cobble-bed streams, high Froude numbers likely indicate that not all energy losses have been fully accounted for (Jarrett 1987).

Critical Depth Assumption

Grant (1997) asserts that in steep (slope greater than 1%) mobile-bed channels, interactions between hydraulics and bed configurations prevent the Froude number from exceeding 1 for more than short distances and time periods. The Froude number is defined as

$$Fr = \frac{\alpha^{0.5} v}{(gd)^{0.5}} \tag{3}$$

where Fr is the Froude Number, α is the kinetic energy correction factor, v is velocity, g is acceleration due to gravity, and d is flow depth. The Froude number equals 1 at critical flow, is greater than 1 for supercritical flow, and is less than 1 for subcritical flow. At critical flow, specific energy is minimized, hence maximizing discharge per unit width – critical flow is highly efficient.

Critical flow in steep channels is maintained by the interaction of the mobilized bed and vegetation with the water surface at high Froude numbers, resulting in the oscillating creation and destruction of bed forms. This has been shown in field observations of sand-bed streams, active braided rivers, step-pool streams, laboratory rills, lahar runout channels and some bedrock channels (Grant 1997). Empirical analysis of mobile bed streams indicate that competent (with bed load transport) flows tend to asymptotically approach critical flow. In sand bed streams, Grant found that the Froude number oscillated between 0.7 and 1.3, with and average of 1.0 in the thalweg. He asserts that critical flow represents a point of high efficiency in flow, beyond which turbulance (hydraulic jumps) interact with bed materials, resulting in rapid energy dissipation and a return to near critical flow (Grant, 1997).

Assuming critical flow in the modeling of flow hydraulics during extreme events in steep, mobile bed streams may likely be an accurate and appropriate method for modeling flow in steep channels. In any case, it is indicated that a critical depth assumption is more appropriate than assuming current roughness values for many or most steep streams encountered in dam breach modeling.

This technique has been adopted for certain applications. Since an assumption of supercritical flow was made in many indirect measurements of peak flow using the slope-area method, many high outliers can be found in gage records for steep streams. These estimates may be significantly overestimated (Jarrett 1987, Webb and Jarrett 2002). A critical depth method is now preferred by many practitioners in such situations. The critical depth technique is also being used in paleoflood studies, as discussed in Webb and Jarrett (2002).

Hence, it is believed by many hydrologic practicioners that supercritical flow is not usually sustainable for significant distances in steep erodable-bed channels but that critical flow is common in streams with slopes greater than about 1 percent (Webb & Jarrett, 2002; Grant 1997). Supercritical flow is usually only sustained in steep, hydraulically smooth, rigid channels, such as concrete channels. Knowing this, it would be best to use a critical depth methodology within an unsteady flow model, but such a feature has yet to be included within FLDWAV or HEC-RAS. In the meantime, a quasi-calibration can be performed on Manning's *n*, to adjust it to prevent supercritical flow for more than short distances and time periods.

Calibration using Froude Number

To more appropriately model dam breach travel times, velocities, depths, widths, and attenuation, *n* values have been adjusted in the Willow Park analysis to prevent the occurrance of supercritical flow for all but the shortest reach lengths. For steep reaches (stream segments that produce Froude numbers greater than 1.0 using ordinary methods), the following procedure was used in the selection of *n* values in this dam breach analysis:

First, *n* values were chosen using visual inspection and the recommendations of Chow (1959) and Brunner & Goodell, 2002. This initial model was developed for the steeper reaches, to the point where the profile significantly flattens out and critical or supercritical flow was no longer expected.

The results were then inspected, looking for, besides the usual warning and errors that would need to be corrected, high Froude numbers in the computed model. If the Froude number at the modeled cross-sections was typically greater than 1.0 (and above 1.2 to 1.3 at any particular section) the roughness estimates (*n*) for the affected cross-sections were increased and an additional model run performed. The regression equation developed by Jarrett (1984) for selecting *n* in steep reaches was used as a guide, though it was found that this method may overestimate *n* estimates. Jarrett's equation is:

$$n = 0.39 S_f^{0.38} R^{-0.16}$$

where S_f is the frictional slope and *R* is the hydraulic radius in feet. If the Froude numbers for the revised model didn't fall within the expected range (below 1.2 to 1.3 but above 0.8 for sections that were previously computed as supercritical) this process was repeated in a trial-and-error manner until Froude numbers all fall below 1.3, with an average of 1.0 for the affected sections. Such a method likely provides more realistic estimates of velocity and travel time for dam breach flood waves through steep reaches.

Modeled Reaches

To assist in model debugging, the floodwave routing was performed in nine linked but separate analyses. These model reaches were South Piney Canyon; South Piney Creek in Story; Piney Creek from Story to I-90; Piney Creek from I-90 to US-14; Piney Creek from US-14 to Clear Creek Confluence; Clear Creek from Piney Creek to Clearmont; Clear Creek from Clearmont to Buffalo Creek; Clear Creek from Buffalo Creek to Powder River; and the Powder River. The entire model length is illustrated in the plan and profiles of Figures 6 and 7.

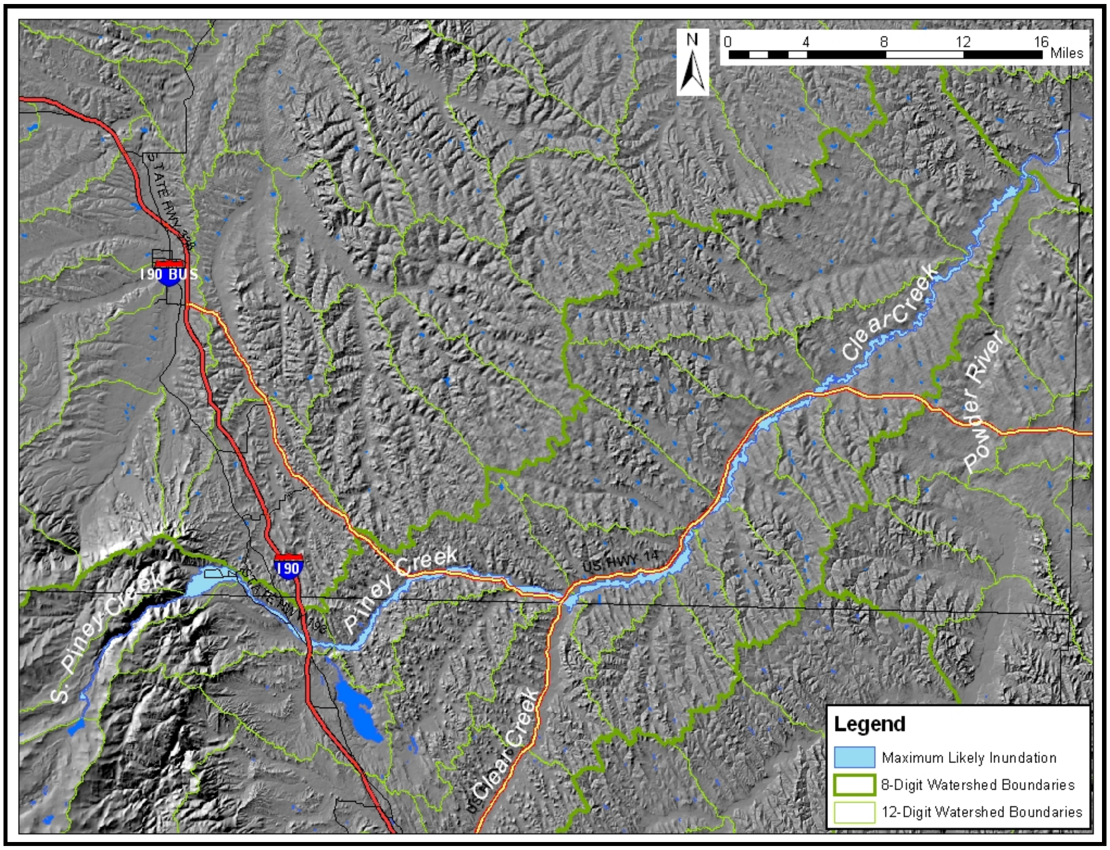


Figure 6: Plan view of Willow Park breach analysis.

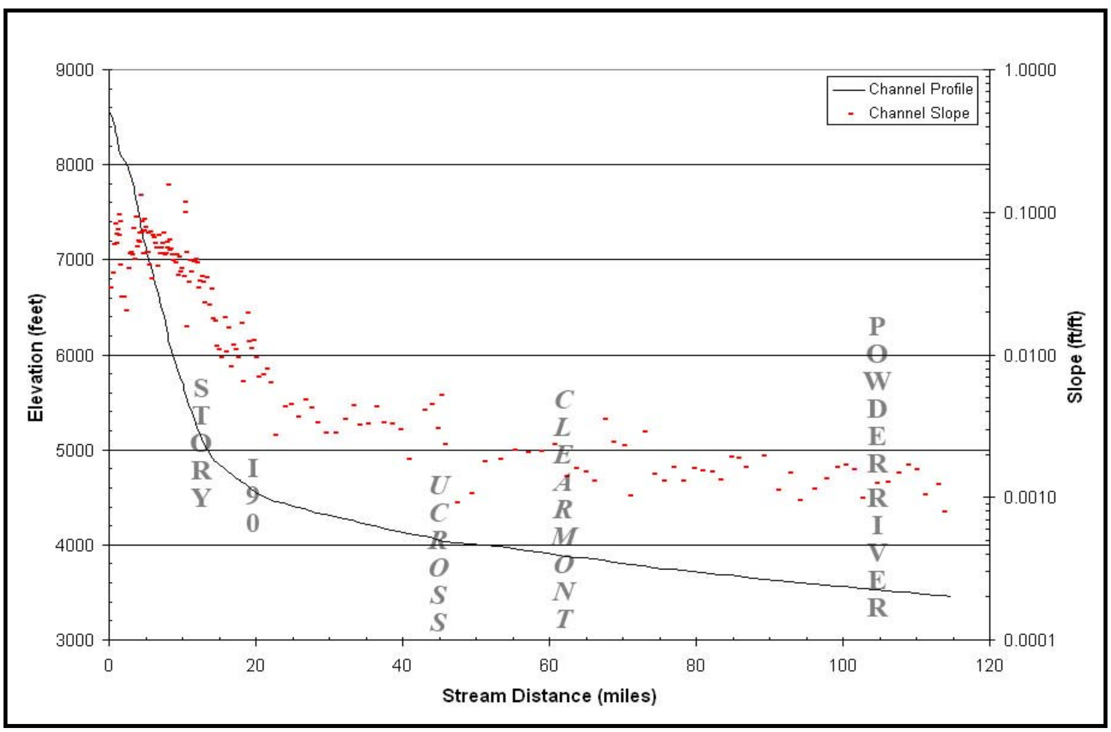


Figure 7: Profile view of Willow Park breach analysis.

South Piney Canyon

Figures 8 through 10 are provided to document general reach characteristics.

Cross-sections were developed in GIS from 40-ft contours on USGS 7.5 minute quadrangles. This accuracy is appropriate for modeling routing and attenuation through the valley and canyon but not for accurately predicting water surface elevations at any but the lowest most cross section, which had a cross-section measured in the field. Seventy-three cross-sections were created using the USGS mapping, aerial photography, and site visits over a reach length of 56520 ft (10.7 miles). For model stability, many interpolated cross-sections were created, defined at approximately 20 ft spacing for a total of 2753 cross-sections.

Field determined Manning's roughness was estimated from a field visit to the upper and lower portions of the modeled reach. The extent of Manning's roughness segment in each cross-section was estimated using aerial photography. With a Manning's values estimated at 0.050 for the channel (fairly clean, winding, some pools and shoals with weeds and stones) and 0.15 for the overbank (brush, trees, with flow into branches), the resulting HEC-RAS model predicts a flood wave with sustained supercritical flow, with channel velocities as high as 77 ft/s and with Froude numbers as high as 3.2. From the above literature search, it is evident that these high values are likely not possible for a mobile-bed stream. Accordingly, Manning's *n* values for channel portions of cross sections were individually calibrated to maintain a Froude number between 0.8 and 1.2 for all reaches where supercritical flow was initially indicated. Appendix B provides the final Froude numbers, as well as other cross-sectional variables.

The resulting model indicates peak flow channel velocities (at non-interpolated sections) ranging from 14 to 37 ft/s and channel Froude values from 0.8 to 1.2. These modeling results are believed to provide the best estimates of flow for such a breach wave in South Piney Canyon.

Figure 8: Typical stream channel and valley (above canyon) in the South Piney reach.

Figure 9: Typical wooded floodprone zone on South Piney Creek.

Figure 10: Typical reach in South Piney Canyon.

The gage datum at the downstream boundary of this reach could not be located in the field due to the gage being decommissioned. Thus, elevations of the downstream section were set by matching approximate bank flow indicators with the discharge-frequency results from the streamgage. The accuracy of this was checked by computing high and low normal flow with roughness based upon existing conditions and comparing water-surface-elevations with the gage rating table.

Normal depth was assumed as a boundary condition at the downstream end of this reach. A slope of 4.7%, measured from adjacent contours on the USGS quadrangle, was used.

South Piney Creek in Story

Due to little attenuation of the flood wave in South Piney Canyon, most to all of the alluvial fan of Story will be inundated in the event of a catastrophic breach of the Willow Park reservoir. A portion of the mass of debris that will be stripped from the upstream canyon will likely be deposited in debris dams as the flow expands on the alluvial fan in Story. Additionally, the dense vegetation (Figures 11 and 12) and steep slopes (Figure 12) in Story will provide an additional source of debris. Hence, flow paths are unpredictable and it can only be said that a probable inundation zone is at likely risk of flooding.

Figure 11: The alluvial fan of Story. The mouth of South Piney canyon is evident in the top, center.

Figure 12: Typical reach of South Piney in Story.

Modeling is simplified by using cross-sections approximated with a typical stream cross-section (Figure 12) and a relatively flat floodplain to generate a composite cross-section. These simplified cross sections allow modeling in Story with limited data, approximating storage and floodwave attenuation within this reach.

If more accurate predictions are desired for maximum likely inundation, depth, and velocity, finely detailed data of the alluvial fan will need to be gathered and a 2-D hydraulic model, such as SMS FESWMS, would need to be constructed. This is the case since flow from the canyon onto the alluvial fan is a 2-dimensional situation and is only roughly approximated by the 1-dimensional model HEC-RAS. However, the nature of a debris flow will still generate an inherently unpredictable situation – the time, effort and money required to generate a 2-D model may not provide more predictable results due to this unpredictability.

Seventeen cross-sections were constructed. Due to the steep channel slope, many additional interpolated cross-sections were required for model stability and more accurate results. The model was used to interpolate cross-sections every 20 ft for a total of 963 sections for this 3.6 mile reach.

Field determined Manning's roughness was estimated from visits to numerous sections within the reach. With Manning's values estimated at 0.05 for the channel (fairly clean, winding, some pools and shoals with weeds and stones, Figure 12) and 0.15 for the dense vegetation of the overbank (brush, trees, with flow into branches, Figures 11 and 12), the resulting HEC-RAS model again predicts a flood wave with sustained supercritical flow. Manning's n was calibrated to maintain a Froude number close to critical for all previously supercritical reaches. Overbank roughness was maintained at 0.15 and calibrated channel roughness ranged from 0.10 to 0.05.

Two bridges exist on this reach and are shown in Figure 13. These low bridges were not modeled since they will very likely be filled with debris and insignificant to flow conveyance during a breach event.


Figure 13: County road (left) and WY193 bridges on South Piney in Story.

Normal depth was assumed as a boundary condition at the downstream end of this reach. A slope of 1.8%, measured from adjacent downstream contours on the USGS quadrangle, was used for this reach.

Piney Creek from Story to I-90

This reach, with slopes ranging from 1.0 to 2.9 percent (Figure 7) and significant riparian but less floodplain vegetation (Figure 14), has fairly broad floodplains and provides opportunity for additional floodwave attenuation, especially behind the I-90 roadway embankment. The steep slopes and available vegetation indicate that significant debris load will be carried and generated. However, most of the debris load will likely drop out of transport at the bottom end of this reach in the vicinity of I-90, within the backwater induced from the highway embankment and reduced valley slopes.

Figure 14: Piney Creek, from Story to I-90.

Cross-sections were generated using USGS topography supplemented with field-measured floodplain slopes and a typical stream cross-section (Figure 15). Twenty cross-sections were developed and additional interpolated cross-sections were generated for model stability, for a total of 625 sections for this 5.2 mile reach.

Field determined Manning's roughness was estimated from visits to the entire reach, on the ground (Figure 15) and from overlooks. Manning's values were estimated as 0.045 for the channel (fairly clean, winding, some pools and shoals with weeds and stones) and 0.10 for the floodplain (heavy stand of timber, a few down trees, some undergrowth alternating with medium to dense brush). Some areas of the floodplain, especially upper areas, have less vegetation but a constant floodplain n was used to reflect the likeliness that significant debris load will be available within this reach.

Three bridges exist on this reach and are shown in Figures 16 through 18. Neither the WY-193 nor US-87 bridges were modeled since they would likely be insignificant to flow conveyance due to debris. However, the I-90 bridge, with much greater conveyance capacity, was modeled. For simplicity in this analysis, the true vertical alignment of the highway was not modeled. Ineffective flow areas were stipulated at both the adjacent upstream and downstream sections, for non-overtopping flows (less than about 40,000 cfs).

Normal depth was assumed as a boundary condition at the downstream end of this reach. A slope of 1.2%, measured from adjacent downstream contours on the USGS quadrangle, was used for this reach.

Figure 15: Typical section for Piney from Story to I-90.

Figure 16: WY-193 crossing of Piney Creek.

Figure 17: US-87 crossing of Piney Creek.

Figure 18: I-90 crossing of Piney Creek.

Piney Creek from I-90 to US-14

This reach, with stream slopes ranging from 0.3 to 1.2 percent (Figure 7), is significantly shallower, with a broad flat floodplain with side slopes of 0 to 2 percent. Cottonwoods and willows line the stream banks forming a vegetated riparian corridor but upper floodplain slopes are lightly vegetated. Figures 19 and 20 provide photographs of typical channel and floodplain conditions.

Cross-sections were generated using USGS topography supplemented with field-measured floodplain slope, and typical stream cross-sections (Figure 19) as well as terrace locations from aerial photography. Fourteen cross sections were developed and additional interpolated cross-sections were generated for model stability, for a total of 583 sections for this 10.8 stream mile reach.

Field determined Manning's roughness was estimated from visits to the entire reach. Manning's values were estimated as 0.040 for the channel (fairly clean, winding, some pools and shoals). Some areas of the floodplain, especially upper areas, have less vegetation and an *n* of 0.040 (pasture, high grass, little to no brush) while riparian floodplain areas have a higher *n* of 0.10 (timber, some down trees or medium to dense brush. Depending upon the section, an “average” *n* of 0.05 or a variable *n* by vegetation extent (as shown on USGS quads and aerial photography) was used.

The one county road bridge and the several private bridges within this reach are not modeled due to the likely insignificant effects that they will have upon the breach routing.

Normal depth was assumed as a boundary condition at the downstream end of this reach. A slope of 0.36%, measured from adjacent contours on the USGS quadrangle, was used.

Figure 19: Typical sections used in I-90 to US-14 reach.

Figure 20: Typical floodplain in I-90 to US-14 reach.

Piney Creek from US-14 to Clear Creek Confluence

Cross-sections were generated using USGS topography supplemented with field-measured floodplain slope and typical stream cross-sections, as well as terrace locations from aerial photography. Thirteen cross sections were developed and additional interpolated cross-sections were generated for model stability, for a total of 385 sections for this 14.8 stream mile (9.2 valley mile) reach. Figure 21 provides photographs of typical channel and floodplain conditions.

Figure 21: Typical channel and floodplain in the US-14 to Clear Creek reach.

Field-determined Manning's values were estimated as 0.040 for the channel (fairly clean, winding, some pools and shoals). Some areas of the floodplain, especially upper areas, have less vegetation and an *n* of 0.040 (pasture, high grass, little to no brush) while riparian floodplain areas have a higher *n* of 0.08 (medium brush) to 0.10 (timber, some down trees or medium to dense brush. Depending upon the section, either an average *n* of 0.05 or a variable *n* by vegetation extent (as shown on USGS quads and aerial photography) was used.

Two US-14 bridges and several private bridges cross Piney Creek within this reach. These bridges were not modeled due to the likely insignificant effects that they will have upon the breach routing.

Normal depth was assumed as a boundary condition at the downstream end of this reach. A slope of 0.38%, measured from adjacent contours on the USGS quadrangle, was used.

Clear Creek from Piney Creek to Clearmont

Cross-sections were generated using USGS topography combined with stream channel cross-sections provided by Cheryl Harrelson of Steady Stream Hydrology in Sheridan, Wyoming. These data were supplemented by locating terraces and floodplains with aerial photography. Ten cross sections were developed and additional interpolated cross sections were generated for a total of 307 sections in this 16.0 stream mile (9.3 valley mile) reach. Figure 22 provides photographs of typical channel and floodplain conditions.

Figure 22: Typical channel and floodplain in the Piney Creek to Clearmont reach.

Field determined Manning's values were estimated as 0.040 for the channel (fairly clean, winding, some pools and shoals) and 0.05 for the floodplain. This floodplain estimate represents a mix of an upper floodplain roughness of 0.04 (pasture, high grass, little to no brush) and a riparian zone floodplain roughness of 0.07 (medium brush).

A county and a private bridge have not been modeled due to likely insignificant effects that they will have upon the breach routing.

Normal depth was assumed as a boundary condition at the downstream end of this reach. A slope of 0.18%, measured from adjacent contours on the USGS quadrangle, was used.

Clear Creek from Clearmont to Buffalo Creek

Cross-sections were generated using USGS topography combined with a typical channel section. These data were supplemented by locating terraces and floodplains with aerial photography. Thirty-one cross sections were developed and additional interpolated cross sections were generated for a total of 695 sections in this 38.4 stream mile (24.8 valley mile) reach. Figure 23 provides photographs of typical channel and floodplain conditions.

Figure 23: Typical channel and floodplain in the Clearmont to Buffalo Creek reach.

Field determined Manning's values were estimated as 0.040 for the channel (fairly clean, winding, some pools and shoals) and 0.05 for the floodplain. This floodplain estimate represents a mix of an upper floodplain roughness of 0.04 (pasture, high grass, little to no brush) and a riparian zone floodplain roughness of 0.07 (medium brush).

Two county bridges and several private bridges within this reach are not modeled due to likely insignificant effects that they will have upon the breach routing. However, the US-14/16 and the Burlington Northern Railroad bridges (Figures 24 and 25) were included in the model.

Normal depth was assumed as a boundary condition at the downstream end of this reach. A slope of 0.15%, measured from adjacent contours on the USGS quadrangle, was used.

Figure 24: US-14/16 crossing of Clear Creek.

Figure 25: Burlington Northern Railroad crossing of Clear Creek.

Clear Creek from Buffalo Creek to Powder River

Cross-sections were generated using USGS topography combined with a typical channel section. These data were supplemented by locating terraces and floodplains with aerial photography. Seven cross sections were developed and additional interpolated cross sections were generated for a total of 179 sections in this 9.3 stream mile (6.9 valley mile) reach. Figure 26 provides photographs of typical channel and floodplain conditions.

Figure 26: Typical channel and floodplain in the Buffalo Creek to Powder River reach.

Field determined Manning's roughness was 0.040 for the channel (fairly clean, winding, some pools and shoals). Some areas of the floodplain, especially upper areas, have less vegetation and an *n* of 0.040 (pasture, high grass, little to no brush) while riparian floodplain areas have a higher *n* of 0.08 (medium brush) to 0.10 (timber, some down trees or medium to dense brush. Depending upon the section, an average *n* of 0.06 or a variable *n* by vegetation extent (as shown on USGS quads and aerial photography) was used.

Normal depth was assumed as a boundary condition at the downstream end of this reach. A slope of 0.16%, measured from adjacent contours on the USGS quadrangle, was used.

Powder River

Cross-sections were generated using USGS topography supplemented with a typical stream cross-section that was generated from USGS streamgage information (063245000, Powder River at Moorhead, MT), which notes a bankfull width of 215 ft and an average depth at bankfull of 9 ft. This information was retrieved from http://montana.usgs.gov/freq. The floodwave was routed from the Clear Creek confluence to Moorhead, Montana however only the uppermost 6.1 stream miles (5.7 valley miles) of this reach is addressed in this report. Figure 27 provides a photograph of the Powder River a few miles downstream of the Clear Creek confluence.

Figure 27: Powder River near the Clear Creek confluence.

Field determined Manning's roughness was 0.040 for the channel (fairly clean, winding, some pools and shoals). Some areas of the floodplain, especially upper areas, have less vegetation and an *n* of 0.040 (pasture, high grass, little to no brush) while riparian floodplain areas have a higher *n* of 0.08 (medium brush) to 0.10 (timber, some down trees or medium to dense brush. Depending upon the section, an average *n* of 0.06 or a variable n by vegetation extent (as shown on USGS quads and aerial photography) were used.

Normal depth was assumed as a boundary condition at the downstream end of this reach, at the gaging station at Moorhead, Montana. A slope of 0.13%, measured from adjacent contours on the USGS quadrangle, was used in this normal depth computation.

LIKELY INUNDATION EXTENT AND TIMING

This analysis provides a prediction of the extent and timing of flooding from a catastrophic breach of Willow Park dam. These results are sufficient for developing an emergency action plan for such a situation. However, due to limitations in the understanding of and ability to model flow dynamics of such a severe, abrupt, and debris saturated breach wave within a steep, wooded channel, the modeling only provides an approximation of what will actually occur. Also, flow from the mouth of the South Piney Canyon onto the alluvial fan of Story is a 2-dimensional process that was approximated using a 1-dimensional model. Additionally, since funding and personnel were not available for surveying full cross-sections, topographic contours from USGS 7.5-minute quadrangles and typical sections were used in their place. For all of these reasons, the results of this analysis should be considered approximate. The nature and limitations of these predictions must be kept in mind when using these results.

A catastrophic breach of Willow Park dam, with an initial peak flow of about 75,200 cfs, will inundate 80 miles of floodplains and agricultural production areas along South Piney Creek, Piney Creek, and Clear Creek before finally attenuating to about 18,800 cfs in the Powder River a few miles downstream of the Clear Creek confluence. This is approximately a 25-year event for this point on the Powder River (see discharge-frequency computations in Appendix C). Figure 28 provides the routed breach hydrographs at 12 points within the analysis zone. In the case of such a breach, hundreds of homes and businesses will be threatened with damage or destruction, farm and ranch land will be flooded, several highways and one interstate will be inundated, bridges may be damaged, and many lives could be lost. Due to this potential, an Emergency Action Plan is being developed for the Willow Park reservoir. Identification of the extent and timing of a floodwave is the first part of an Emergency Action Plan.

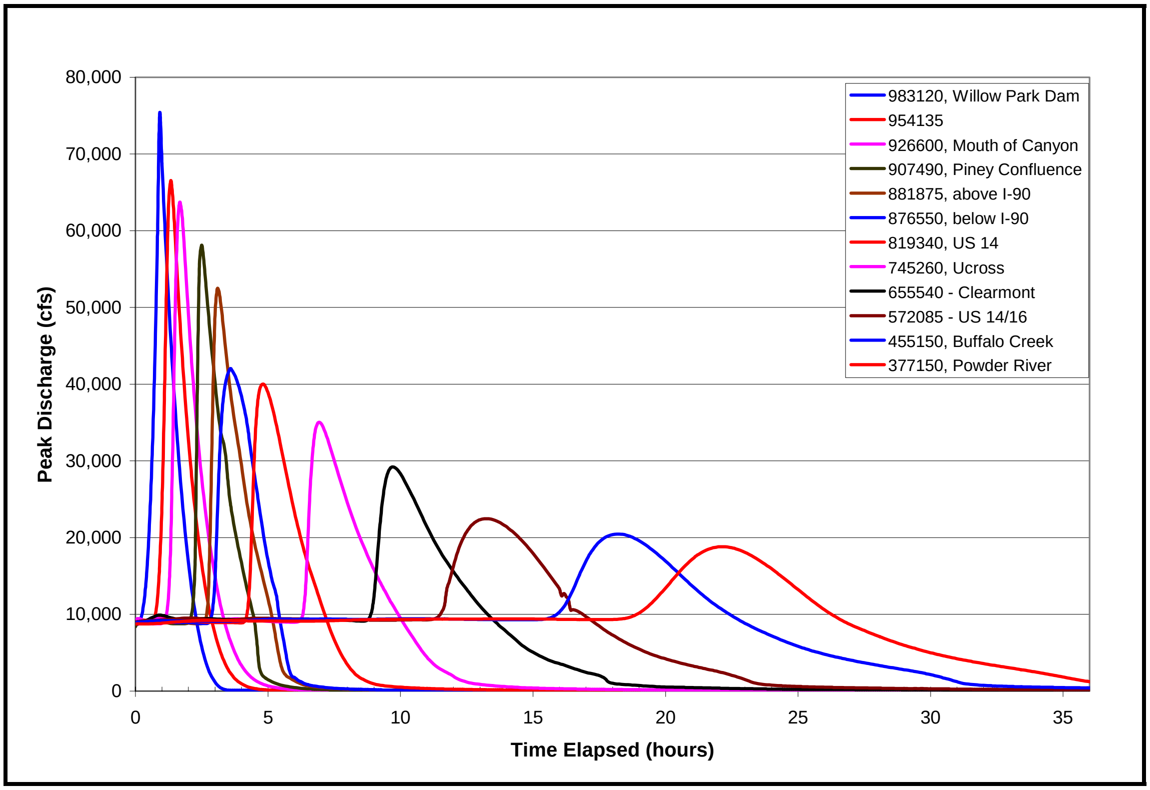


Figure 28: Breach hydrographs.

The probable inundation extent and timing is provided on the inundation maps of Appendix A. These fifteen maps, which were created using ArcMAP 8.3, provide a probable inundation extent superimposed upon USGS 7.5 minute quadrangles. Tables imbedded within these plots (and elaborated upon in Appendix B) indicate peak discharge at each section, approximate maximum depth and velocities, and breach wave timing and steepness for selected sections. Also included within these plots are photographs of some of the structures that will be threatened by a breach, with the associated times to initial and peak inundation provided for convenience. A key to these maps is provided in Figure 29.

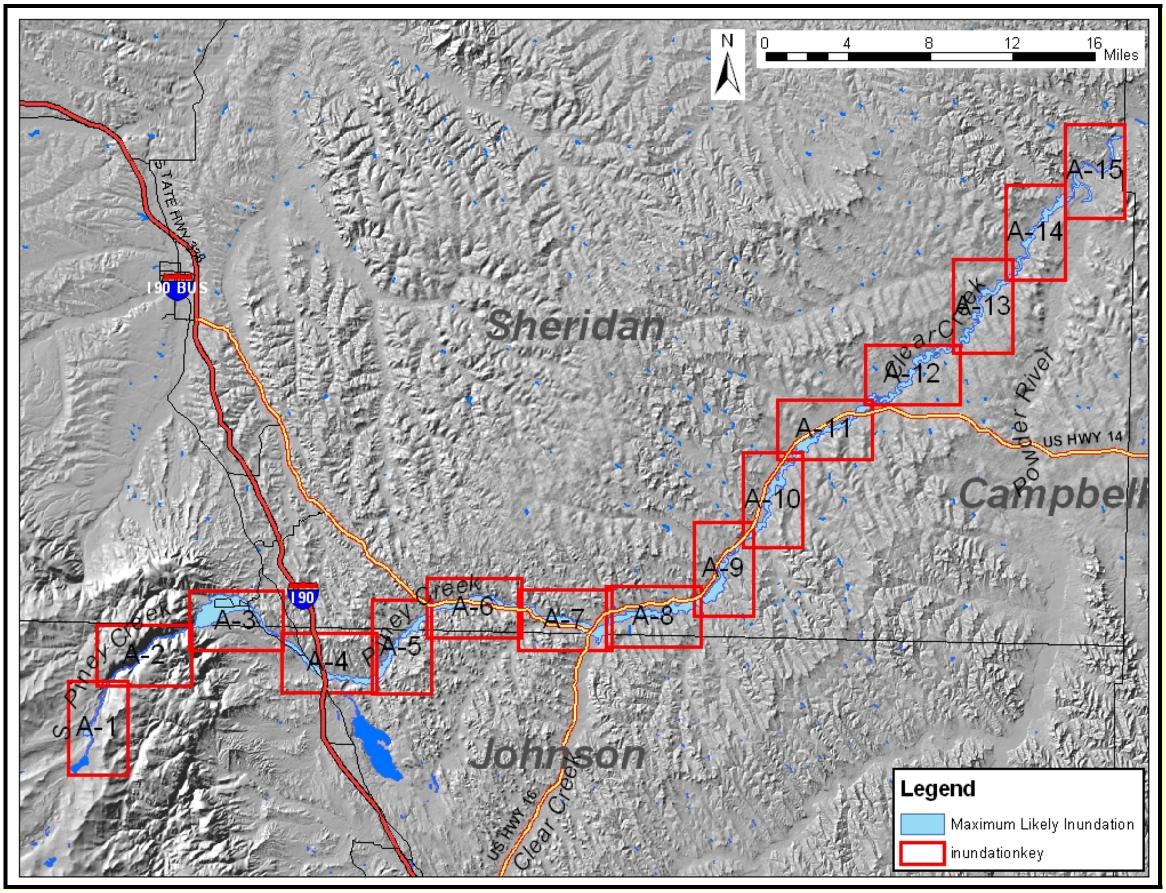


Figure 29: Probable inundation map key.

Based upon the unsteady flow analysis of South Piney Creek, Piney Creek, Clear Creek and the Powder River, the following scenario is presented as the result of a catastrophic breach of Willow Park Dam.

A breach of the embankment may occur from overtopping, piping, or embankment sliding or settlement. With an initially completely filled reservoir, a hydrograph with a peak of approximately **75,200 cfs** and a volume of 6260 ac-ft will result. The time-to-peak of this hydrograph is estimated to be **55 minutes**. The resulting floodwave will envelope the valley bottom of South Piney Creek for the entire 9.5 mile (valley length) reach, to the mouth of South Piney Canyon above Story. At this point peak flow will likely be attenuated to roughly **63,700 cfs**, which is more than 30-times greater than the maximum recorded flow of 2,090 cfs (in 1963) and almost 28-times greater than the estimated 100-year flow of 2,290 cfs (Appendix C). Peak flow depths will range from 11 to 37 feet within this reach, with average peak channel velocities ranging from 14 to 37 ft/s and floodplain velocities ranging from 2 to 22 ft/s. The time-to-peak of the floodwave will shorten from 55 minutes at the dam to 40 minutes at the mouth of the canyon. Due to the steep, wooded, alluvium-bedded nature of this reach, this extreme flow will likely cause a great deal of woody debris liberation and bed scouring, with channel erosion in the tens of feet and the stripping of most vegetation within the flood path. It may be the case that as more of the floodway is inundated and stripped, the resulting debris flow will periodically lose its capacity to transport this entrained debris, become subcritical, and set up a temporary debris dam which will shortly break, remobilizing a portion of the debris dam until another dam is formed downstream. The floodwave leading edge and peak will take approximately **1.0 and 1.7 hours**, respectively, to reach the canyon mouth.

This canyon reach is on public land so no residences should be inundated. However, anyone camping or recreating in the stream valley will be threatened by the floodwave. Deaths occurred in just such a situation from the Lawn Lake dam failure in Rocky Mountain National Park.

At the peak flow of **63,700 cfs** at the canyon mouth, maximum flow depths will be approximately 24 feet, with channel velocities of 27 ft/s. As this breach wave exits the canyon mouth above Story, the flow will spread out upon the alluvial fan, likely adding a significant amount of debris to the fan in places as the flow expands and becomes shallower and velocities are reduced. However, the stream and alluvial fan are still steep and vegetation and alluvium are still prevalent - significant scouring and vegetation stripping in areas is expected. Cascading debris dams or debris deflections will also likely form, creating unpredictable flow paths throughout the width of the alluvial fan. Hence most of the community of Story could be threatened in the event of a breach. Peak flow depths will range from 12 to 18 ft within the Story reach, with average peak channel velocities of 17 to 22 ft/s and floodplain velocities ranging from 3 to 10 ft/s. The time-to-peak of floodwave will shorten from 40 to 25 minutes. Within this 3.6 mile reach the peak flow is expected to attenuate to **58,200 cfs**, with floodwave leading edge and peak taking approximately **2.1 and 2.5 hours**, respectively, to reach the Piney confluence. Most of the homes, businesses, and roads in Story will be threatened with damage or destruction by the floodwave. There is a high potential for loss of life.

As the floodwave proceeds down Piney Creek, flow will attenuate from **58,200 cfs** to **52,500 cfs** just above I-90 within this 4.1 miles reach. The floodwave leading edge and peak will take **2.6 and 3.1 hours**, respectively, to reach section 881,875, two sections above the I-90 bridge. Peak flow depths in this reach will range from 15 to 19 feet, with average peak velocities of 13 to 26 ft/s and floodplain velocities ranging from 3 to 8 ft/s. Time-to-peak will range from 25 to 30 minutes within this reach. Numerous roads, structures, and lives will be threatened.

Flow over the I-90 bridge and embankment is possible, due to partial debris blockage. Storage from backwater behind the embankment is expected to reduce the peak flow to **42,300 cfs.** Bridge failure due to abutment or pier scour may be a possibility. Danger exists to any vehicles (and occupants) caught in the possible overflow or failure.

Downstream of the bridge, at section 876,550, flow will be attenuated to **42,100 cfs**, with the floodwave leading edge and peak arriving at **2.8 and 3.6 hours**, respectively. Within this next reach, from I-90 to US-14 at section 819,340, peak flow will attenuate to **40,000 cfs** with the floodwave leading edge and peak arriving at **4.2 and 4.8 hours**, respectively. Peak flow depths within this reach will range from 8 to 19 ft, with average peak channel velocities of 9 to 19 ft/s and floodplain velocities ranging from 3 to 8 ft/s. Time-to-peak will range from 35 to 40 minutes within this reach. A county road, structures, and lives will be threatened.

Within the next reach, from section 819,340 to 745,260 (at Ucross), flow will attenuate from **40,000 cfs** to **35,000 cfs**, with the floodwave leading edge and peak arriving at **6.3 and 6.9 hours**, respectively. Peak flow depths will range from 9 to 17 ft, with average peak channel velocities of 8 to 18 ft/s and floodplain velocities ranging from 2 to 9 ft/s. Time-to-peak will range from 40 to 45 minutes. US-14/16, various structures, and lives will be threatened. The 35,000 cfs flow at Ucross is almost 10-times the maximum recorded flow of 3570 cfs (in 1963) and the estimated 100-year flow of 3,620 cfs. (Appendix C).

Within the next reach, from Ucross to Clearmont, flow will attenuate from **35,000 cfs** to **29,200 cfs**, with the floodwave leading edge and peak arriving at Clearmont in **8.8 and 9.8 hours**, respectively. Peak flow depths will range from 8 to 17 ft, with average peak channel velocities of 6 to 11 ft/s and floodplain velocities ranging from 3 to 7 ft/s. Time-to-peak will increase from 40 to 60 minutes in Clear Creek. Numerous structures and lives will be threatened. However, Clearmont itself should not be directly impacted **–** the flow will likely remain in the floodplain to the immediate East of town.

Within the reach from Clearmont to the Powder River flow will attenuate from **29,200 cfs** to **19,600 cfs**, with the floodwave leading edge and peak arriving at the Powder River at **17.8 and 20.7 hours**. Peak flow depths will range from 9 to 17 ft, with average peak channel velocities of 4 to 23 ft/s and floodplain velocities ranging from 2 to 12 ft/s. Time-to-peak will increase from 60 to 175 minutes. The 19,600 cfs flow in Clear Creek near its mouth is greater than the maximum recorded flow of 9600 cfs (in 1954). This flow is also greater than the estimated 200-year flow of 15,700 cfs. County roads, various structures, and lives will be threatened. However, the US-14/16 and railroad crossings of Clear Creek will not likely be overtopped, unless a significant quantity of debris becomes lodged in the bridge structure before the peak passes. Also, bridge failure due to abutment or pier scour is a possibility.

Within the first few miles of the Powder River the floodwave's peak flow will attenuate to **18,900 cfs**, with the floodwave leading edge and peak flow arriving at section 377,150 at **18.6 and 22.0 hours,** respectively. Time-to-peak will be 205 minutes at this section. Peak channel flow velocities will range from 5 to 7 ft/s with the little bit of floodplain flow providing velocities of about 1 ft/s. This 18,800 cfs flow is approximately a 25-year event, according to the Powder River streamgage near the state line (Appendix C). This flow, which will continue to attenuate, will have minimal potential for danger to structures and lives within the sparsely-populated Powder River valley.

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