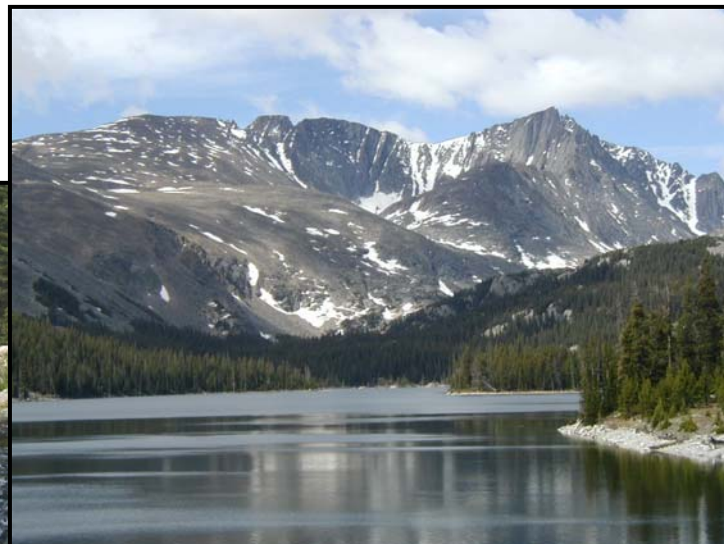
Cloud Peak Reservoir Dam Breach Analysis

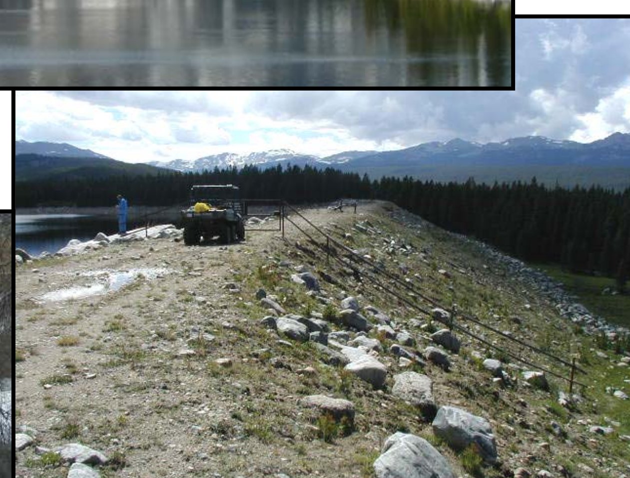
Johnson and Sheridan Counties, Wyoming

April 2005

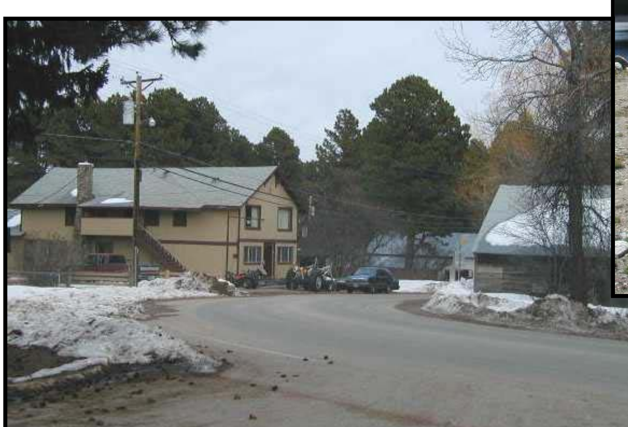
Cloud Peak Reservoir



South Piney Creek



Willow Park Embankment



Story

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NATURAL RESOURCES CONSERVATION SERVICE
ROCKY MOUNTAIN ENGINEERING TEAM

Lakewood, Colorado

April 29, 2005

Cloud Peak Reservoir Dam Breach Analysis

Job Number: WY0302
Short Job Description: Cloud Peak 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 Cloud Peak Reservoir. This report details the dam breach analysis performed on the reservoir for the purpose of an emergency action plan.

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INTRODUCTION

This report details the methods and results of a dam breach analysis performed on the Cloud Peak 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 reservoir company, impacted communities, and emergency response teams for the development of an emergency action plan.

The Cloud Peak Reservoir (Figures 1 and 2) is located on South Piney Creek at an elevation of 9700 feet in the Bighorn Mountains above the town of Story. Average precipitation within the reservoir's 8.1 square mile watershed varies from 31 to 39 inches, according to PRISM (**P**arameter-Elevation **R**egressions on **I**ndependent **S**lopes **M**odel). The embankment has a maximum height of about 35 feet, with a crest elevation of 9732 feet and associated storage of 4590 ac-ft. At the emergency spillway crest elevation of 9726 feet, the associate reservoir storage is 3560 ac-ft. These volumes do not account for accumulated sediment since construction in circa 1960.

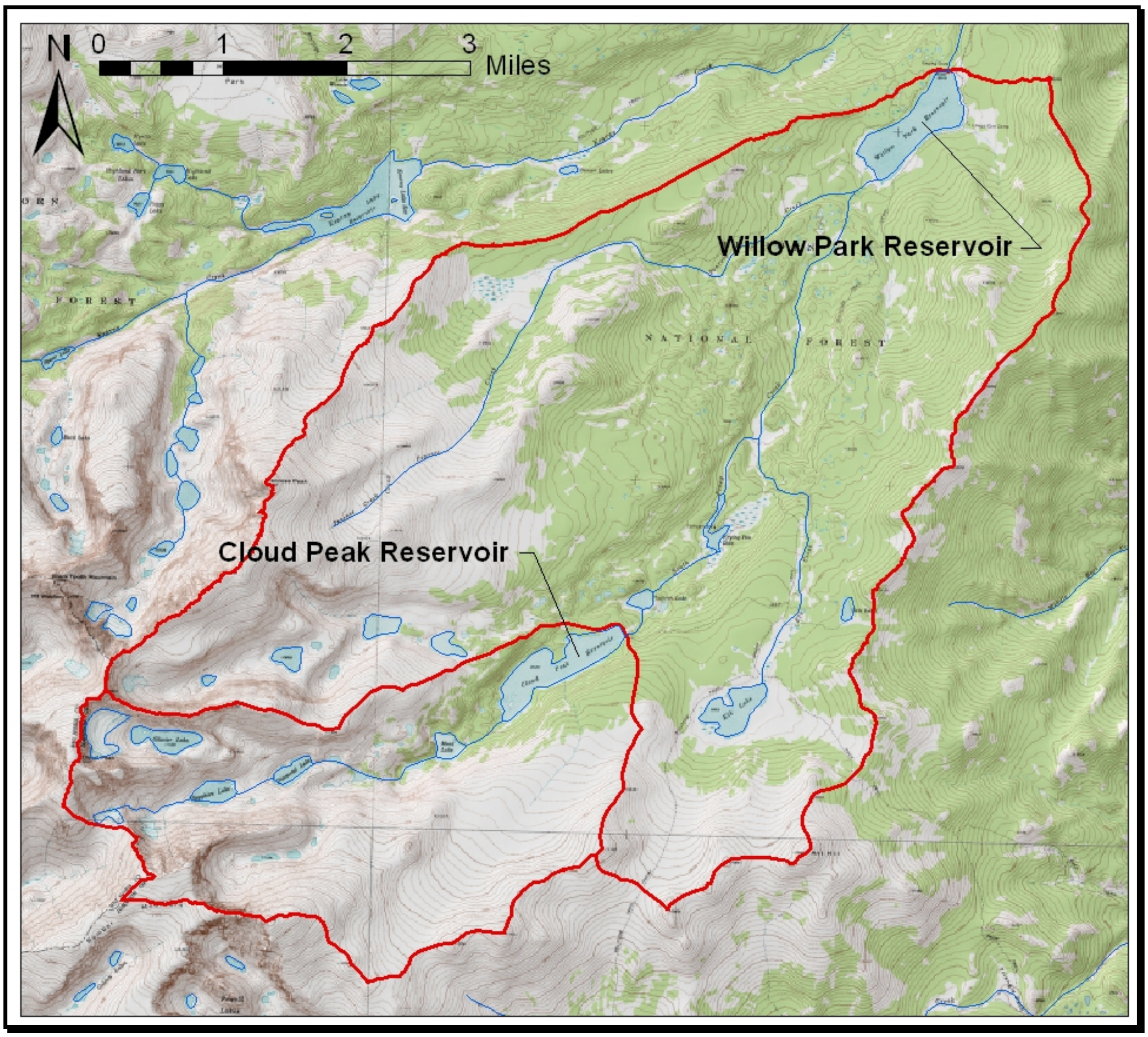


Figure 1: Watersheds, Cloud Peak and Willow Park reservoirs.

Downstream of Cloud Peak Reservoir is Willow Park Reservoir (see Figures 1 and 2). This reservoir's embankment will likely fail in the case of a breach of Cloud Peak's dam. It is located on South Piney Creek at an elevation of 8600 feet. 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.

This analysis is sufficient for the development of an emergency action plan for the catastrophic breach of Cloud Peak's 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.**

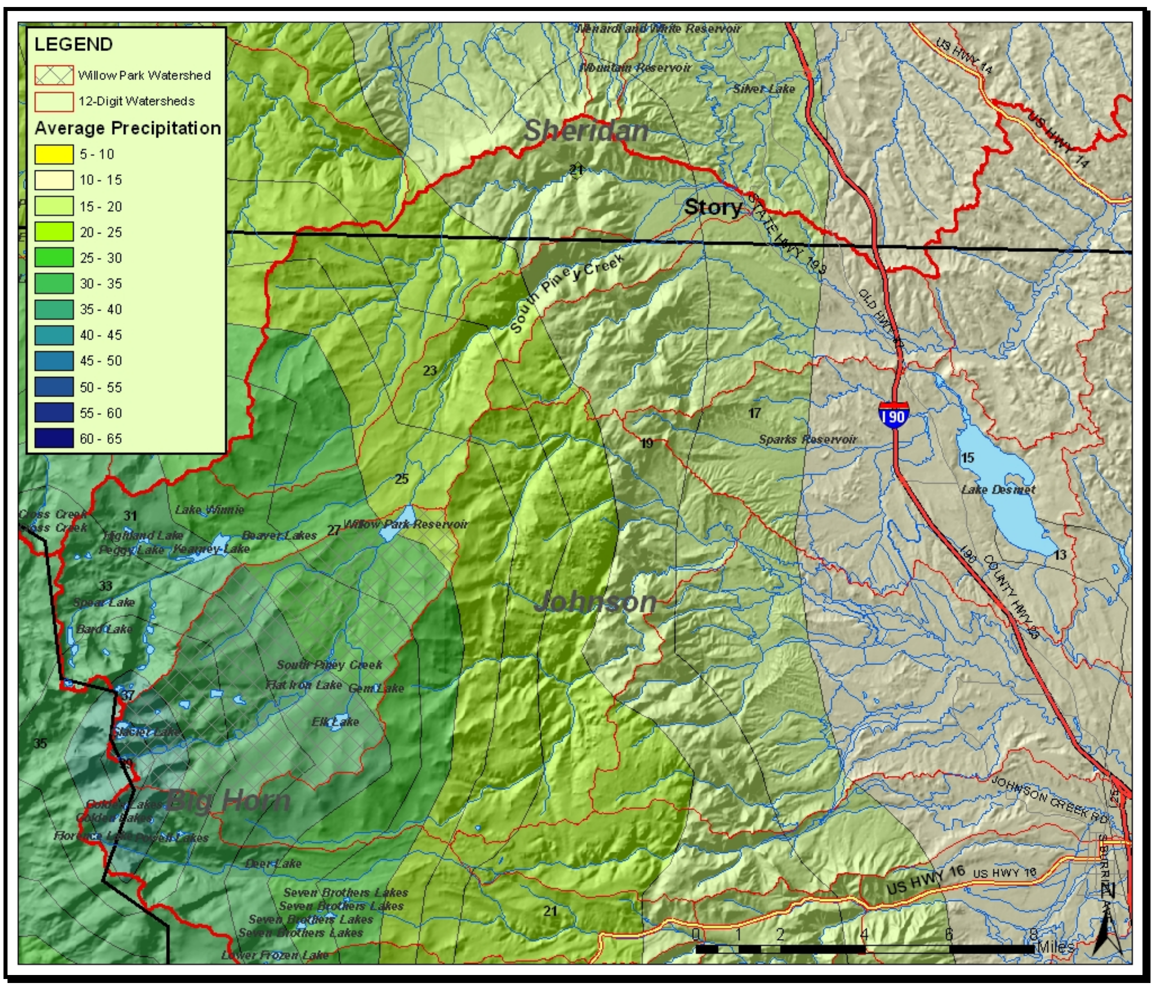


Figure 2: Region of analysis, mountainous portion. Shaded relief, average precipitation (PRISM) estimates, reservoirs and lakes, and 12-digit watersheds are shown. The Cloud Peak and Willow Park watersheds are shown cross-hatched.

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 streamgage 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.**

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.

Cloud Peak reservoir is upstream of Willow Park reservoir – this is a dams in series situation in which failure of the Cloud Peak will lead to the failure of the lower dam.

To help illustrate conditions of the Cloud Peak and Willow Park embankments, Figures 3 through 7 have been provided for reference.

Figure 3: Embankment, Cloud Peak reservoir.

Figure 4: Downstream embankment face, Cloud Peak Reservoir.

Figure 5: Emergency spillway, Cloud Peak reservoir.

Figure 6: Downstream embankment face, Willow Park Reservoir.

Figure 7: Emergency spillway, Willow Park Reservoir.

The breach hydrograph was developed using the breach subroutine in HEC-RAS 3.1.3 (beta). A sine wave breach progression was chosen to simulate the overtopping failure, with a resulting trapezoidal breach form. Breach characteristics used in the modeling include reservoir volume, average breach width, breach side slopes, and time-to-peak estimates. Cloud Peak's emergency spillway maximum flow was modeled to be approximately 2000 cfs. This was based upon design dimensions. Willow Park reservoir was assumed to have a water surface elevation of about 8620.4 feet at the time of the Cloud Peak failure, which provides 0.9 feet of flow depth over the emergency spillway.

Detailed cross sections of the reservoir pool were entered into the model for the Cloud Peak and Willow Park reservoir reaches. These cross sections define the reservoir storage to be routed downstream in the breach model.

Average breach width was estimated using Froehlich's regression equation (Froehlich 1995b). This method uses the equation

$$\overline{B} = 15 k_0 V_w^{0.32} H^{0.19} \qquad (1)$$

where V_w is the reservoir volume at the time of failure (millions of m^3), H is the height of the final breach (meters), and k_o is equal to 1.4 for an overtopping failure mode or 1.0 for other failure modes. This equation provides an average breach width of 188 ft (57.4 m) for Cloud Peak reservoir and 226 ft (69.9 m) for Willow Park reservoir.

Breach side slopes were assumed to be 1 to 1 for Willow Park. This is the average slope that Froehlich (1995b) found in the analysis of 63 embankment dam failures. For the Cloud Peak Failure, the side slopes were assumed to be 1 to 1.5, due to valley constrictions at the embankment base minimizing the bottom width. For this failure, a bottom width of 40 feet was used, which assumes some scour of the abutments.

A time-to-peak estimate was created using Froehlich's regression equation (Froehlich 1995b). This method uses the equation

$$t_f = 3.84 V_w^{0.53} h_b^{-0.90} \qquad (2)$$

where t_f is the breach formation time (hours), V_w = is the reservoir volume at time of failure (millions of m^3) and h_b is the height of breach (m). This method provides a time-to-peak estimate of 1.14 hours for Cloud Peak and 0.91 hours for Willow Park. Characteristics of the Cloud Peak failure are provided in Table 1, with the breach hydrograph illustrated in Figure 8.

Table 1: Breach characteristics, Cloud Peak Reservoir.

Breach Shape				Water	Time		
Average Width (ft)	Bottom Width (ft)	Sideslope (ft/ft)	Height (ft)	Surface Elevation (ft)	to Peak (hrs)	Volume (ac-ft)	Peak Flow (cfs)
90	40	1.5/1	35	9732	1.14	4590	29,900

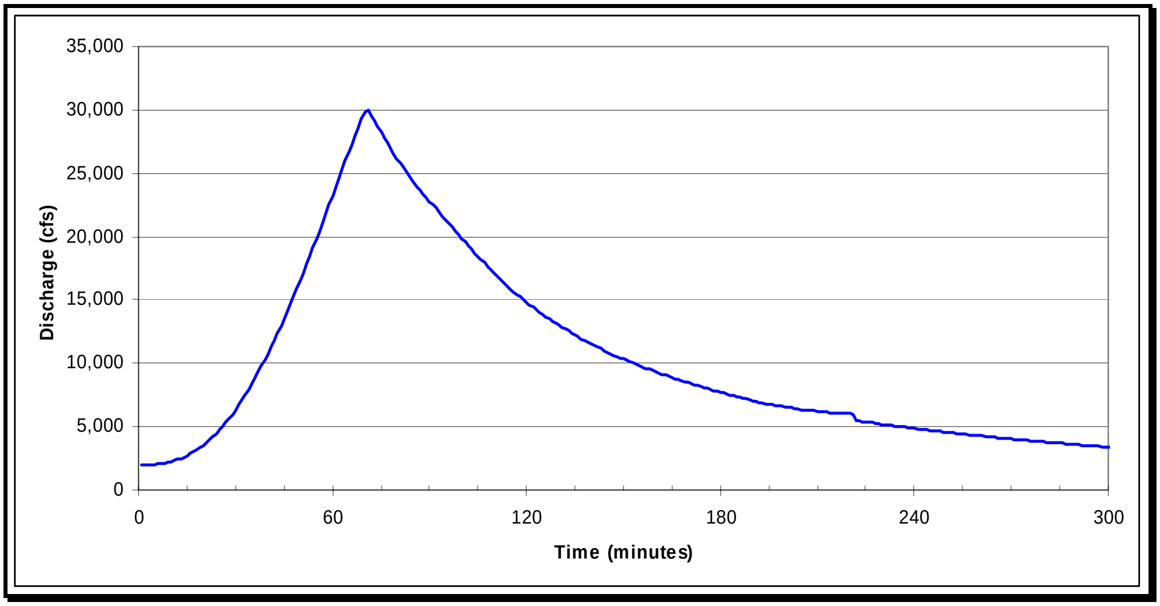


Figure 8: Initial breach hydrograph, Cloud Peak Failure.

To verify the appropriateness of the HEC-RAS predicted peak breach flow for Cloud Peak Reservoir, the estimate was compared to results generated from numerous predictor equations.

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

$$Q_p = 0.607 V_w^{0.295} H_w^{1.24} \tag{3}$$

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 35 ft (10.7 m – to bottom of culvert) and storage at crest of approximately 4594 ac-ft (5,666,000 m^3), a peak discharge of 39,800 cfs was estimated.

Peak flow estimates were also computed 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

$$Q_{\max} = 1100 B_r^{1.35} \tag{4}$$

where

$$B_r = \frac{V_s H_w}{A} \tag{5}$$

But the peak flow is not to be less than

$$Q_{\max} = 3.2 H_w^{2.5} \tag{6}$$

and need not exceed

$$Q_{\max} = 65 H_w^{1.85} \tag{7}$$

where V_s is the reservoir storage at the time of failure (ac-ft), H_w is the depth of water at the dam at the time of failure (ft) and A is the cross-section area at the dam at the location of the breach (ft^2). With an embankment cross-sectional area of 8074 ft^2, results for all methods are provided in Table 2.

Table 2: Breach hydrograph characteristics, Cloud Peak Reservoir.

Description	Reservoir WSEL (ft)	Reservoir Volume (ac-ft)	HEC-RAS Peak (cfs)	Froehlich Peak (cfs)	NRCS Peak Estimates Eq. 4 (cfs)	Eq. 6 (cfs)	Eq. 7 (cfs)
at Embankment Crest	9732.0	4,594	29,900	39,800	62,400	23,200	46,700

The peak flow of 29,900 cfs is significantly smaller than the Froehlich equation's result of 39,800 cfs, but this is not surprising considering the shape of the reservoir and embankment. The reservoir is long and skinny (Figure 1) with a constrained embankment that will not allow a wide breach bottom width. Also, the peak flow is within the range of NRCS's TR-60 criteria – this HEC-RAS breach wave prediction is considered reasonable.

Since an overtopping event is being modeled in this analysis, a large hydrologic event is assumed to occur within the reservoir's watershed, an event large enough to completely fill the reservoir to the capacity of the emergency spillway. However, in the breach routing no adjacent watersheds (to the downstream reaches) are assumed to be contributing flow to South Piney Creek. **Hence, this analysis predicts the maximum likely inundation due only to a breach of Cloud Peak Reservoir's embankment.**

The Cloud Peak breach hydrograph is routed downstream through the South Piney Creek stream valley to Willow Park reservoir. The peak flow attenuates from 29,900 cfs to 27,700 cfs at the head of Willow Park reservoir. The steepness of the stream and the valley landform constraints prevent much attenuation in this 4.4-mile valley-length reach.

Within the reservoir, peak flow attenuates from 27,700 cfs to 19,900 cfs as the flow travels across the reservoir pool. The initial water surface elevation in the reservoir is approximately 8620.4 ft, allowing about 0.9 ft of flow through the emergency spillway (Figure 7). Once the breach flow reaches the dam, the emergency spillway capacity of approximately 8350 cfs will be exceeded and the embankment will be overtopped by about 0.7 ft, forcing a failure. The characteristics of this second failure are provided in Table 3, with the breach hydrograph provided in Figure 9.

Table 3: Breach characteristics, Willow Park Reservoir.

Breach Shape				Water	Time		
Average Width (ft)	Bottom Width (ft)	Sideslope (ft/ft)	Height (ft)	Surface Elevation (ft)	to Peak (hrs)	Volume (ac-ft)	Peak Flow (cfs)
224	170	1/1	54.5	8625.5	0.91	6260	134,000

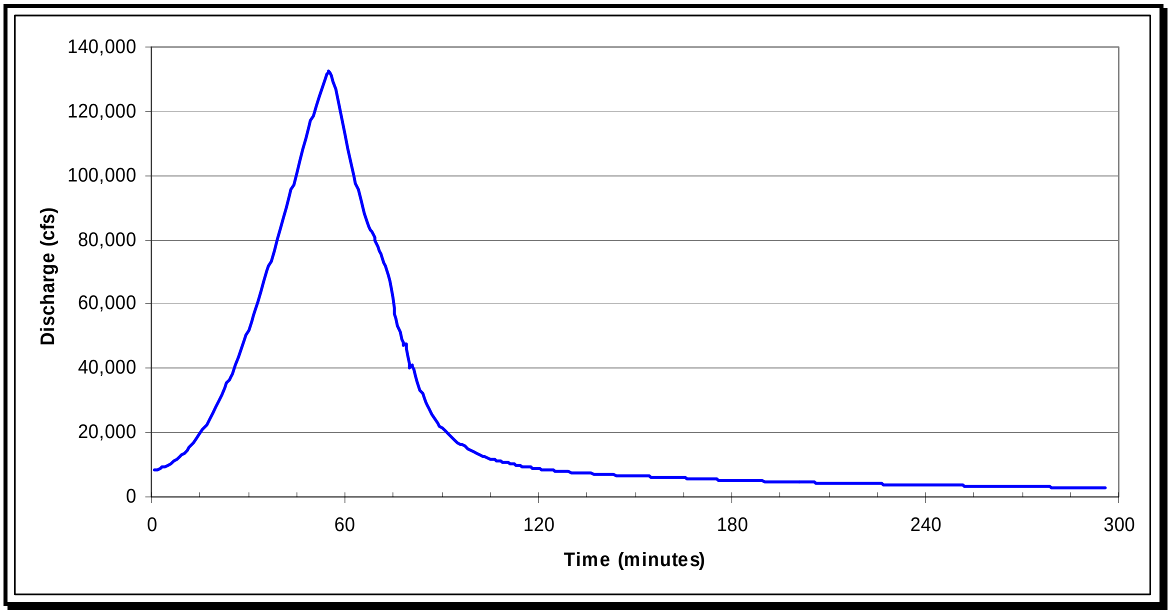


Figure 9: Initial breach hydrograph, Willow Park reservoir.

Since the Cloud Peak dam failure was modeled as an overtopping event, a large hydrologic event was assumed to occur within the upper watershed. Such an event would likely be a rain or rain-on-snow event during the early summer during a wet year, when the reservoir would likely be near its storage capacity, or a large rain event in late summer or early autumn, a storm large enough to fill the reservoir's empty storage towards the end of the irrigation season. However, no adjacent watersheds (to the downstream reaches) are assumed to be contributing flow to the stream – in this way the failure is modeled as a "sunny-day breach."

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 flood waves between the reservoirs and down through the South Piney Canyon, through Story, and through the Piney and Clear creek river valleys downstream of Story. HEC-RAS version 3.1.3 beta 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:

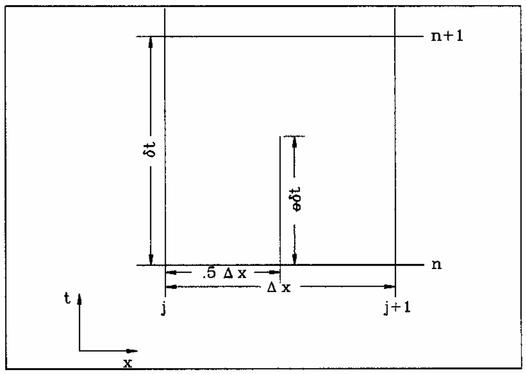
$$\frac{\partial Q}{\partial t} + \frac{\partial QV}{\partial x} + gA\left(\frac{\partial z}{\partial x} + S_f\right) = 0$$

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

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: $\dfrac{\partial f}{\partial t} \approx \dfrac{\Delta f}{\Delta t} = \dfrac{0.5(\Delta f_{j+1} + \Delta f_j)}{\Delta t}$

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

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 for alluvial streams. 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. Very high Froude numbers and velocities computed in the 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, illustrate 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 flow of 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 bedrock channels, but is a questionable practice for the natural alluvial 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

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

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 turbulence (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 dam breach modeling in alluvial-bed streams.

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 practitioners 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 occur 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

This issue of the selection of the appropriate steep-channel *n* values is relevant in four upper reaches in the Cloud Peak breach analysis, specifically Upper South Piney Creek, South Piney Creek, South Piney Creek Canyon, and South Piney Creek in Story. These reaches are all alluvial-bed streams. In the Upper South Piney Creek, initial velocities were predicted to be as high as 48 ft/s, with Froude numbers as high as 2.59. In the South Piney Creek reach, initial velocities were predicted to be as high as 99 ft/s, with Froude numbers as high as 3.39. In the South Piney Creek Canyon reach, initial velocities were predicted to be as high as 101 ft/s, with Froude numbers as high as 3.35. Such high velocities are not considered reasonable in a mobile-bed stream.

To more appropriately model dam breach travel times, velocities, depths, widths, and attenuation, Manning's *n* values have been adjusted in the Cloud Peak breach analysis to prevent the simulation 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 warnings 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. 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 fell 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 a dam-breach flood wave traveling through the steep reaches of South Piney Creek.

Modeled Reaches

To assist in model debugging, the floodwave routing was performed in thirteen linked but separate analyses. These model reaches were Cloud Peak Reservoir; Upper South Piney Creek; Willow Park Reservoir; South Piney Creek; South Piney Creek 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; and Clear Creek from Buffalo

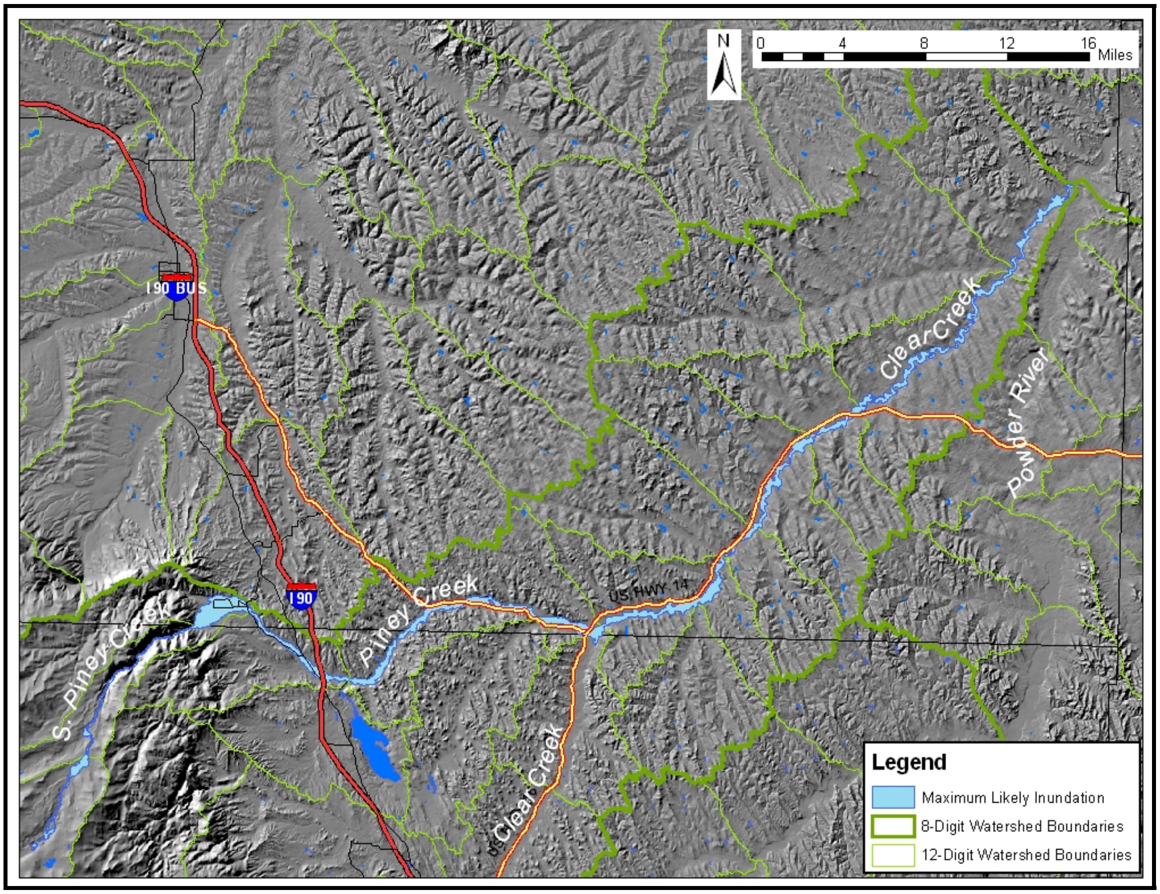


Figure 10: Plan view of Cloud Peak breach analysis.

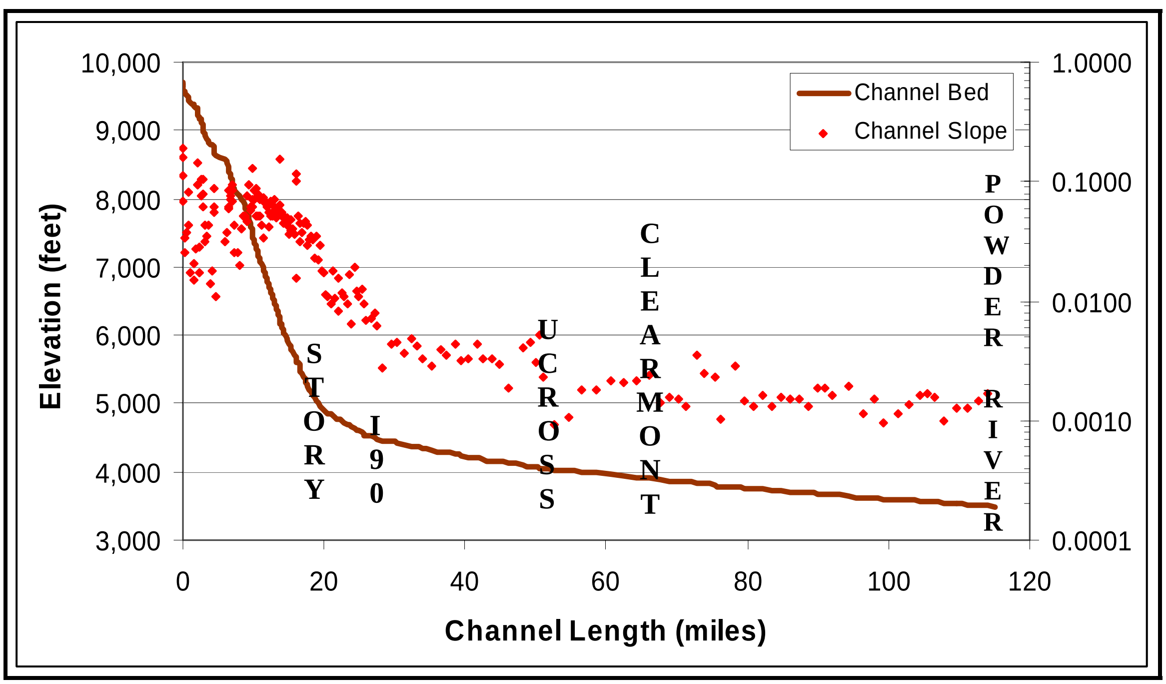


Figure 11: Channel profile and slopes for the Cloud Peak breach analysis.

Creek to the Powder River. The entire model length is illustrated in the plan and profiles of Figures 10 and 11.

Cloud Peak Reservoir

The first segment modeled in this analysis was Cloud Peak reservoir. This segment modeled the reservoir storage volume and produced the initial breach hydrograph. To allow model stability, the downstream normal depth boundary condition was set at only 0.02, instead of the actual slope of 0.12. This steepness of slope on the boundary leads to a great deal of model instability. Due to this adjusted slope, water surface elevations at the boundary section (1014100) will not be correct.

Upper South Piney Creek

Figures 12 is 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. Thirty-four cross-sections were created using the USGS mapping, aerial photography, and site visits over a 4.7 mile stream length (4.4 mile valley length). For model stability, many interpolated cross-sections were created, defined at approximately 20 ft spacing for a total of 1240 cross-sections.

Field determined Manning's roughness was estimated from a field visit an upper portion of the modeled reach. The extent of Manning's roughness segment in each cross-section was estimated using aerial photography. In areas with forested floodplains, a *n* of 0.15 (heavy stand of timber, with a few down trees and flow into branches) was used. In park areas, the *n* varied from 0.15 to 0.06 (floodplain, light brush and trees in summer) using the horizontal variation in *n* value tool. In water bodies with this reach, an *n* of 0.035 was used. These *n* values where chosen using a combination of site visits (almost the entire stream was walked in this reach) and aerial photography. Just downstream of the embankment is a boulder-size rock section. An *n* of 0.07 was used here.

Figure 12: Typical stream channels in the Upper South Piney Creek reach.

The initial HEC-RAS model predicts a flood wave with sustained supercritical flow, with channel velocities as high as 53 ft/s and with Froude numbers as high as 2.7. 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 calibrated to maintain a Froude number between 0.8 and 1.2 for all reaches where supercritical flow was initially indicated.

The resulting model indicates peak flow channel velocities (at non-interpolated sections) ranging from 3 to 26 ft/s and channel Froude values from 0.2 to 1.2. Cross-section variable predictions can be found in Appendix B.

Normal depth was assumed as a boundary condition at the downstream end of this reach. A slope of 7.3%, measured from adjacent contours on the USGS quadrangle, was used.

Willow Park Reservoir

Willow Park reservoir was modeled as a unique segment, for stability. This segment modeled the reservoir storage volume and produced the second breach hydrograph. The downstream normal depth boundary condition was set at the actual slope of 0.032.

South Piney Creek

Figure 13 is 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. Thirteen cross-sections were created using the USGS mapping, aerial photography, and site visits over a 1.8 mile stream (1.7 mile valley length). For model stability, many interpolated cross-sections were created, defined at approximately 20 ft spacing for a total of 483 cross-sections.

Figure 13: Typical stream channel and valley in the South Piney Creek reach.

Field determined Manning's roughness was estimated from a visit to an upper portion 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 (mountain stream, with no vegetation in channel and a bed of cobbles and large boulders) 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 99 ft/s and with Froude numbers as high as 3.4. 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 calibrated to maintain a Froude number between 0.8 and 1.2 for all reaches where supercritical flow was initially indicated.

The resulting model indicates peak flow channel velocities (at non-interpolated sections) ranging from 21 to 36 ft/s and channel Froude values from 0.9 to 1.1. Cross-section variable predictions can be found in Appendix B.

Normal depth was assumed as a boundary condition at the downstream end of this reach. A slope of 2.6%, measured from adjacent contours on the USGS quadrangle, was used.

South Piney Creek Canyon

Figures 14 and 15 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 the 8.5 mile stream length (7.8 mile valley length). For model stability, many interpolated cross-sections were created, defined at approximately 20 ft spacing for a total of 2266 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 Manning's values estimated at 0.050 for the channel (mountain stream, with no vegetation in channel and a bed of cobbles and large boulders) 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 101 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 calibrated to maintain a Froude number between 0.8 and 1.2 for all reaches where supercritical flow was initially indicated.

The resulting model indicates peak flow channel velocities (at non-interpolated sections) ranging from 28 to 46 ft/s and channel Froude values from 0.9 to 1.2. Cross-section variable predictions can be found in Appendix B.

Figure 14: Typical floodprone zone and channel reach, South Piney Creek Canyon reach.

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 the dams-in-series failure of Willow Park reservoir, little attenuation of the flood wave in South Piney Canyon, and the geometry of the alluvial fan that Story is built upon, most of Story will be inundated in the event of a catastrophic breach of Cloud Peak 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. Additionally, the dense vegetation (Figures 15 and 16) and steep slopes (Figure 11) in Story will provide an additional source of debris. This debris will add an additional degree of unpredictability to the inundation.

Figure 15: The alluvial fan of Story. The mouth of South Piney canyon is evident in the top, center.

Figure 16: 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 estimates, 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. Recently gathered LIDAR 1-meter grid data show that significant elevation changes and flow paths do exist throughout the alluvial fan. These data indicate that 2-dimensional model development could be quite helpful for clarifying specific inundation areas. It may be advisable for such a model to be developed for the breach analyses for all the reservoirs upstream of Story, including Cloud Peak, Willow Park, and Kearney.

Seventeen cross-sections were constructed. Due to the steep channel slope, additional interpolated cross-sections were required for model stability and more accurate results. The model was used to interpolate cross-sections every 50 ft for a total of 390 sections for this 3.6 mile stream length (3.4 mile valley length) 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 (mountain stream, with no vegetation in channel and a bed of cobbles and large boulders) and 0.15 for the dense vegetation of the overbank (brush, trees, with flow into branches, Figures

15 and 16), 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 17. 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 17: 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 0.018, 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 11) and significant riparian but less floodplain vegetation (Figure 18), has fairly broad floodplains and provides opportunity for additional floodwave attenuation. 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.

Cross-sections were generated using USGS topography supplemented with field-measured floodplain slopes and a typical stream cross-section (Figure 19). Twenty cross-sections were developed and additional interpolated cross-sections were generated for model stability, for a total of 622 sections for this 5.9 mile stream length (5.0 mile valley length) reach.

Field determined Manning's roughness was estimated from visits to the entire reach, on the ground (Figures 18 & 19) 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.

Figure 18: Piney Creek, from Story to I-90.

Three bridges exist on this reach and are shown in Figures 20 through 22. 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).

Figure 19: Typical section for Piney from Story to I-90.

Figure 20: WY-193 crossing of Piney Creek.

Figure 21: US-87 crossing of Piney Creek.

Figure 22: I-90 crossing of Piney Creek.

Normal depth was assumed as a boundary condition at the downstream end of this reach. A slope of 0.012, measured from adjacent downstream contours on the USGS quadrangle, was used for this reach.

Piney Creek from I-90 to US-14

This reach, with stream slopes ranging from 0.3 to 1.2 percent (Figure 11), 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 23 and 24 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 23) 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 293 sections for this 10.8 mile stream length (6.7 mile valley length) reach.

Figure 23: Typical sections used in I-90 to US-14 reach.

Figure 24: Typical floodplain in I-90 to US-14 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.

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 752 sections for this 14.1 mile stream length (9.0 mile valley length) reach. Figure 25 provides photographs of typical channel and floodplain conditions.

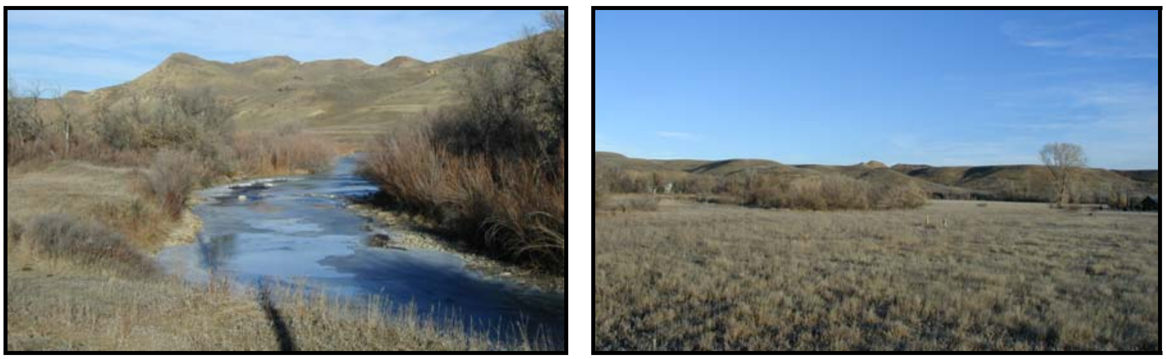


Figure 25: Typical channel and floodplain in the US-14 to Clear Creek reach.

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 17.0 mile stream length (10.1

mile valley length) reach. Figure 26 provides photographs of typical channel and floodplain conditions.

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).

Figure 26: Typical channel and floodplain in the Piney Creek to Clearmont reach.

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 38.0 mile stream length (24.4 mile valley length) reach. Figure 27 provides photographs of typical channel and floodplain conditions.

Figure 27: 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 28 and 29) 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 28: US-14/16 crossing of Clear Creek.

Figure 29: 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 9.7 mile stream length (6.9 mile valley length) reach. Figure 26 provides photographs of typical channel and floodplain conditions.

Figure 30: 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.

LIKELY INUNDATION EXTENT AND TIMING

This analysis provides a prediction of the extent and timing of flooding from a catastrophic breach of the Cloud Peak dam embankment and the resulting dams-in-series failure of the Willow Park dam embankment. 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 Cloud peak dam will initially cause a peak flow of 29,900 cfs. The following Willow Park dam failure will increase the peak flow to 133,000 cfs. Eighty-one linear miles of mountain valleys, floodplains and agricultural production areas along South Piney Creek, Piney Creek, and Clear Creek will be inundated before the floodwave finally attenuates to about 15,900 cfs at the confluence of Clear Creek and the Powder River. This is approximately a 12-year event according to the streamgage on the Powder River at Morehead, near the Montana state line (see discharge-frequency computations in Appendix C). Figure 31 provides the routed breach hydrographs at 11 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 Cloud Peak reservoir. Identification of the extent and timing of a floodwave is the first part of an emergency action plan.

The probable inundation extent and timing is provided on the inundation maps of Appendix A. These sixteen maps 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 32.

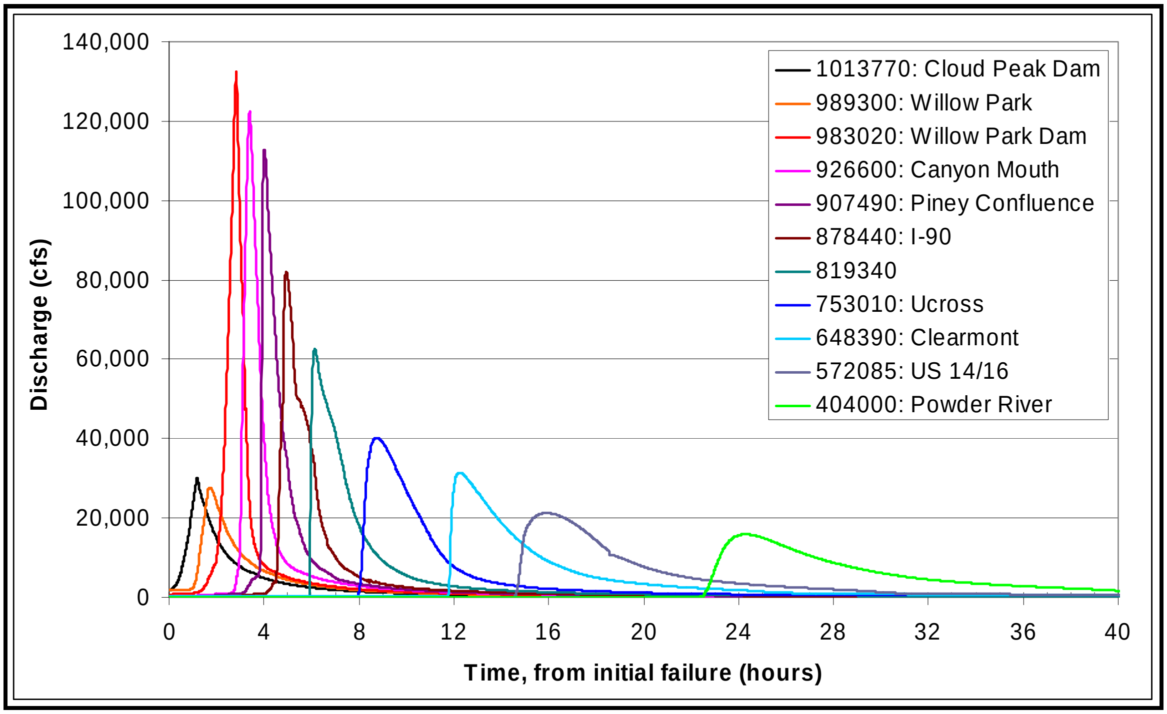


Figure 31: Breach hydrographs.

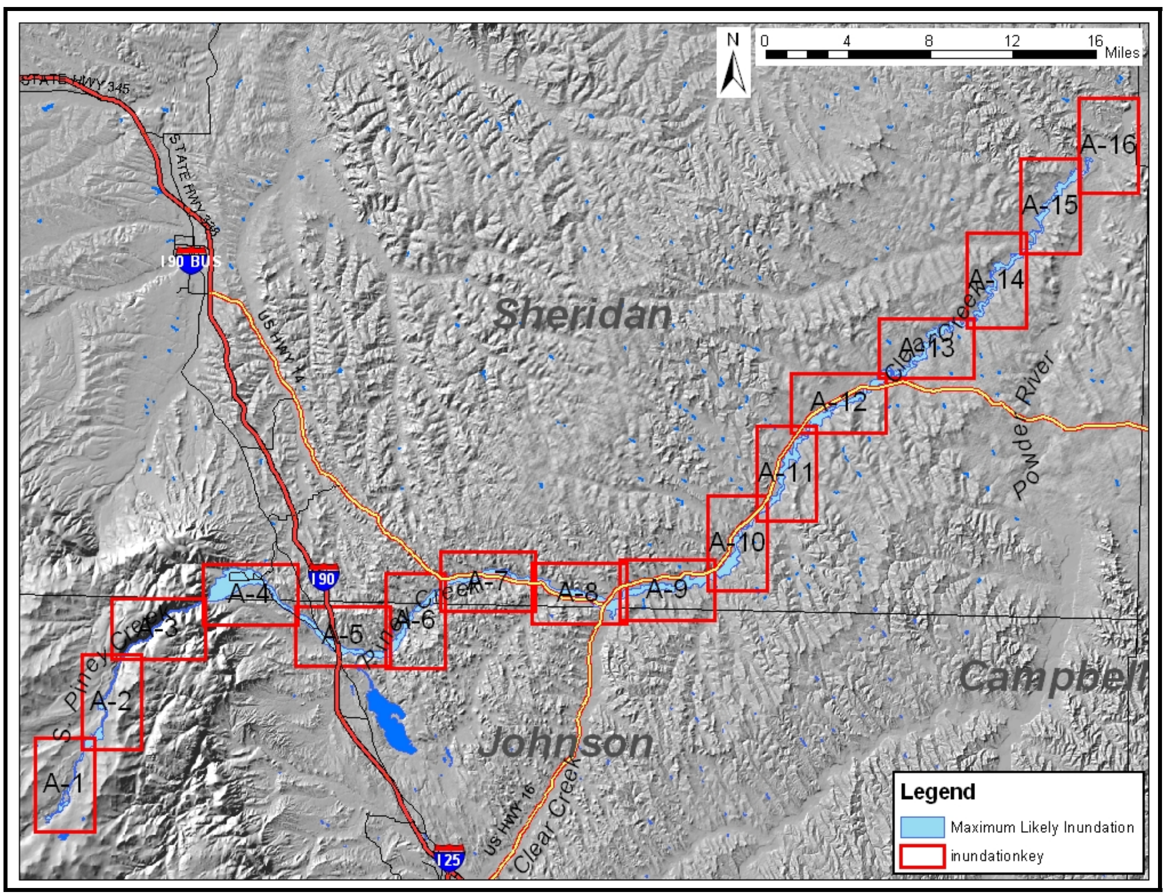


Figure 32: Probable inundation map key.

Based upon the unsteady flow analysis of South Piney Creek, Piney Creek, Clear Creek, the following scenario is presented as the result of a catastrophic breach of Cloud Peak Dam.

A breach of the embankment may occur from overtopping, piping, or embankment sliding or settlement. With an initially completely filled Cloud Peak reservoir a hydrograph with a peak of approximately **29,900 cfs** and a volume of 4590 ac-ft will result from an overtopping failure, the **worst-case scenario**. The time-to-peak of this hydrograph is estimated to be **70 minutes**. In Upper Piney Creek between Cloud Peak and Willow Park reservoirs, peak flow depths will range from 4 to 20 feet, with average peak channel velocities ranging from 3 to 25 ft/s and floodplain velocities ranging from 4 to 16 ft/s. The time-to-peak of the floodwave will shorten from 70 minutes at Cloud Peak dam to 53 minutes at the upper end of Willow Park reservoir. Due to the steep, wooded, alluvial-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 **0.8 and 1.7 hours**, respectively, to reach the upper end of Willow Park reservoir (from the time of the Cloud Peak failure).

The leading edge and peak of the floodwave will take about **1.4 and 2.8 hours,** respectively for this breach wave to reach the Willow Park embankment. The reservoir was assumed to be initially filled to a level with 0.9 ft of flow passing through the emergency spillway. The dams-in-series failure will produce a breach hydrograph with a peak of approximately **133,000 cfs**, a combined volume of about 10,000 ac-ft and an initial time-to-peak of 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 **122,000 cfs**, which is more than 50-times greater than the maximum recorded flow of 2,090 cfs (in 1963) and the estimated 100-year flow of 2,290 cfs (Appendix C). Peak flow depths will range from 15 to 46 feet within this reach, with average peak channel velocities ranging from 21 to 46 ft/s and floodplain velocities ranging from 4 to 22 ft/s. The time-to-peak of the floodwave will shorten from 55 minutes at the dam to 41 minutes at the mouth of the canyon. Due to the steep, wooded, alluvial-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 **2.7 and 3.4 hours**, respectively, to reach the canyon mouth (from the time of the Cloud Peak failure).

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.

Also, there are several permanent camps along these upper reaches (see Figure A-1). This could be a concern - deaths occurred among camping recreationists from the Lawn Lake dam failure in Rocky Mountain National Park.

At the peak flow of **122,000 cfs** at the canyon mouth, maximum flow depths will be approximately 34 feet, with channel velocities of 35 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 13 to 27 ft within the Story reach, with average peak channel velocities of 20 to 31 ft/s and floodplain velocities ranging from 3 to 13 ft/s. The time-to-peak of the floodwave will range from 40 to 50 minutes. Within this 3.6 mile reach the peak flow is expected to attenuate to **102,000 cfs**, with the floodwave leading edge and peak taking approximately **3.2 and 4.0 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 **102,000 cfs** to **82,000 cfs** just above I-90 within this 4.1 miles reach. The floodwave leading edge and peak will take **4.2 and 5.0 hours**, respectively, to reach section 878,440, just upstream of the I-90 bridge. Peak flow depths in this reach will range from 17 to 34 feet, with average peak channel velocities of 7 to 31 ft/s and floodplain velocities ranging from 3 to 7 ft/s. Time-to-peak will range from 38 to 50 minutes within this reach. Numerous roads, structures, and lives will be threatened.

Flow over the I-90 bridge and embankment is likely, especially considering the possibility of partial debris blockage. 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.

Within the next reach, from I-90 to US-14 at section 819,340, peak flow will attenuate to **62,500 cfs** with the floodwave leading edge and peak arriving at **5.9 and 6.1 hours**, respectively. Peak flow depths within this reach will range from 13 to 21 ft, with average peak channel velocities of 11 to 24 ft/s and floodplain velocities ranging from 4 to 9 ft/s. Time-to-peak will range from 12 to 46 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 **62,500 cfs** to **39,900 cfs**, with the floodwave leading edge and peak arriving at **8.3 and 9.0 hours**, respectively. Peak flow depths will range from 9 to 21 ft, with average peak channel velocities of 9 to 15 ft/s and floodplain velocities ranging from 2 to 9 ft/s. Time-to-peak will range from 43 to 77 minutes. US-14/16, various structures, and lives will be threatened. The 39,900 cfs flow at Ucross is more than 11-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 **39,900 cfs** to **31,300 cfs**, with the floodwave leading edge and peak arriving at Clearmont in **11.8 and 12.3 hours**, respectively. Peak flow depths will range from 9 to 17 ft, with average peak channel velocities of 6 to 12 ft/s and floodplain velocities ranging from 2 to 7 ft/s. Time-to-peak will vary from 18 to 43 minutes in this section of 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 **31,300 cfs** to **15,900 cfs**, with the floodwave leading edge and peak arriving at the Powder River at **22.5 and 24.3 hours**. Peak flow depths will range from 8 to 18 ft, with average peak channel velocities of 3 to 30 ft/s and floodplain velocities ranging from 1 to 11 ft/s. Time-to-peak will vary from 30 to 107 minutes. The 15,900 cfs flow in Clear Creek near its mouth is greater than the maximum recorded flow of 9600 cfs (in 1954) and approximately equivalent to 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.

The 15,900 cfs flow in the Powder River is a fairly minor event, approximately equivalent to a 12-year event (Appendix C) at the next streamgage near the Montana boarder. This flow, which will continue to attenuate, will have minimal potential for impact to the sparsely-populated Powder River.

Due to the rapid inundation potential for Story due to both the Cloud Peak and Willow Park dams and the remoteness of these sites, some sort of automated alert system is needed to adequately safeguard the impacted lives downstream. It is recommended that such an automated system be installed for both the Willow Park and Cloud Peak reservoirs. Inclusion of Kearney reservoir in such an automated system is also recommended.

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