Kearney Reservoir Dam Breach Analysis

Johnson and Sheridan Counties, Wyoming.

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Job Number: WY0200
Short Job Description: Kearney Dam breach analysis.
Location: Johnson County, Wyoming near Story on Kearney, South Piney, Piney, and Clear Creeks.

Description of Job: Dam breach analysis of the Kearney Reservoir, for the Kearney Lake, Land, & Reservoir Company and Wyoming NRCS state office.

Purpose of Analysis: To predict probable extent and timing of out-of-bank flow resulting from the catastrophic breach of the Kearney Dam, for the development of an emergency action plan for the impacted communities.

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Acknowledgments

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Introduction

Kearney Reservoir is located in Johnson County, Wyoming at an elevation of 9200 ft in the Bighorn Mountains above the town of Story. Precipitation within the reservoir's 16.6 mi^2 watershed ranges from 29 to 37 inches (from PRISM). The embankment dam has a maximum height of 63 ft (dam crest at approximately 9201 ft), with an associated maximum storage of approximately 7400 ac-ft. At the emergency spillway crest (approximate elevation of 9194 ft), the associated reservoir storage is 6131 ac-ft. Neither of these volumes account for accumulated sediment since dam construction in 1962. This irrigation reservoir was constructed to replace a smaller, breached structure.

Figure 1 provides a plan view of the reservoir and watershed. Figures 2 and 3 are background photos of the dam and emergency spillway.

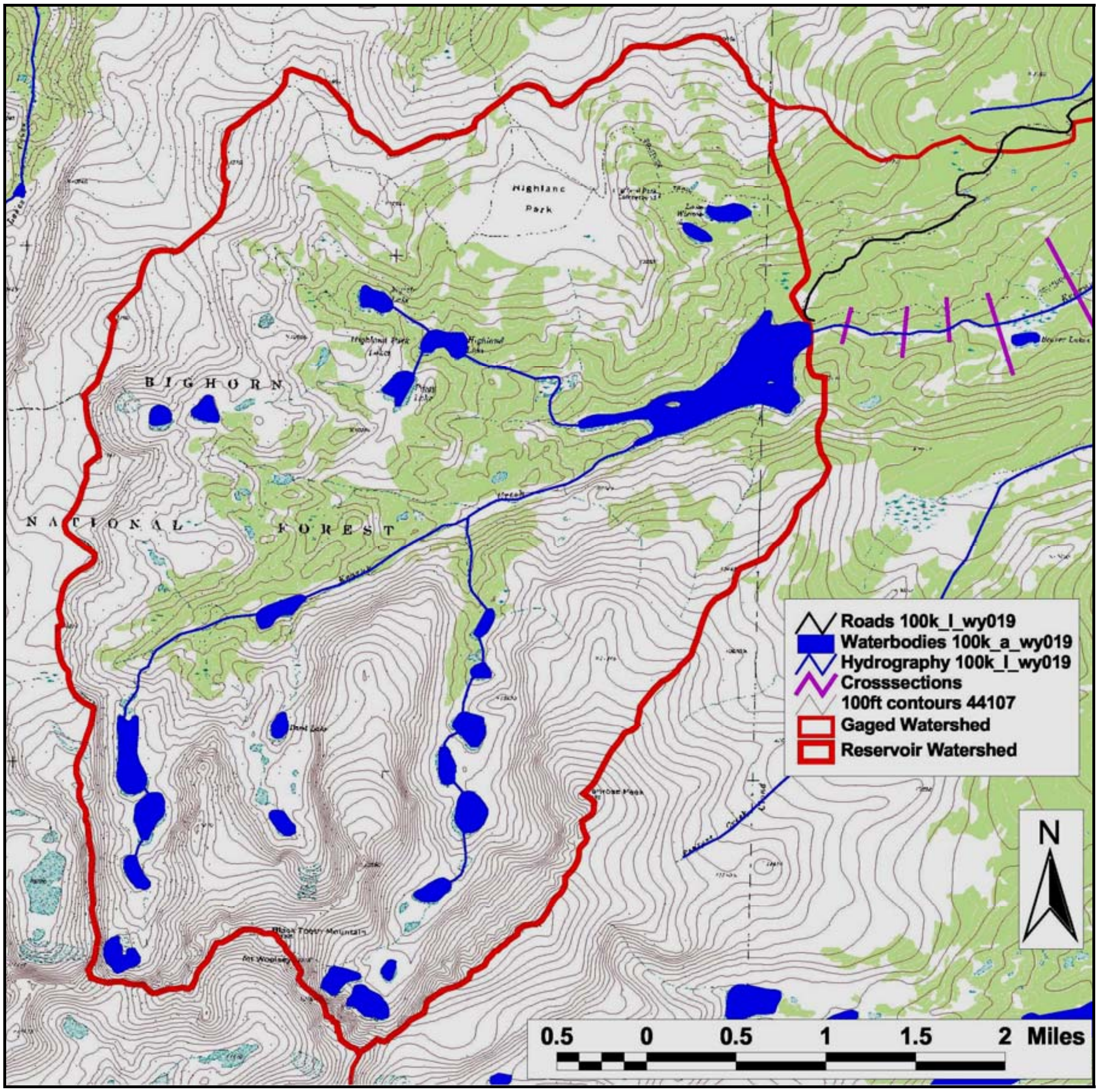


Figure 1: Plan view of Kearney reservoir and watershed. Watershed area = 16.6 mi^2.

Figure 2: Emergency spillway entrance, facing downstream. Flow capacity is approximately 1800 cfs with water-surface-elevation at the dam crest.

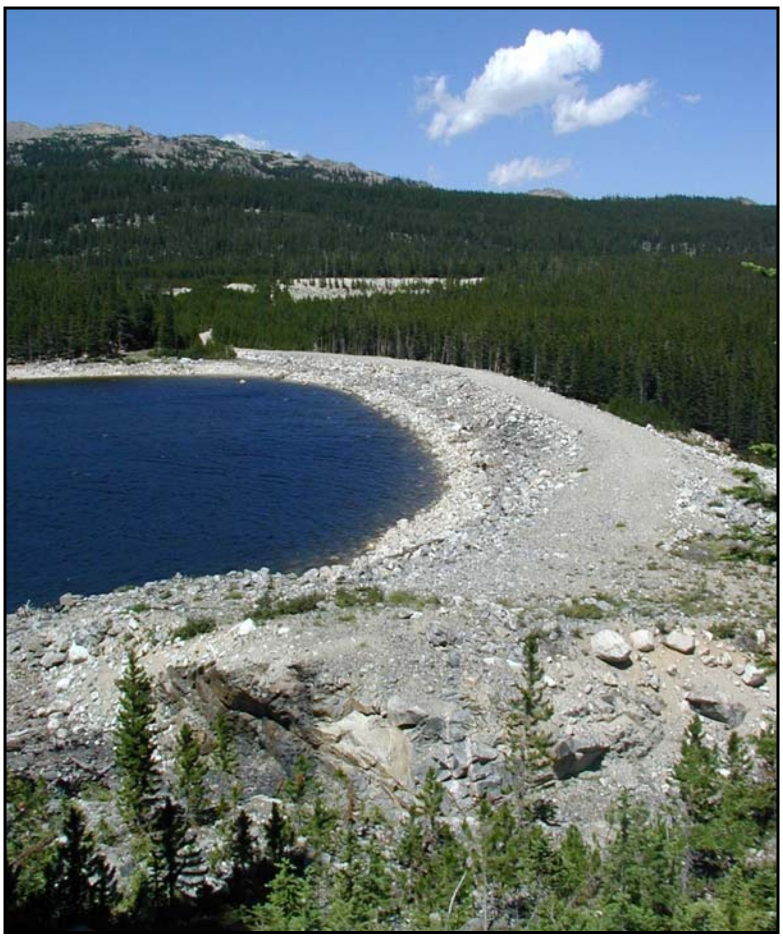


Figure 3: Kearney Dam, with emergency spillway in foreground.

This analysis provides a prediction of the extent and timing of flooding from a catastrophic breach of Kearney 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 is approximate. The results must be, in turn, considered approximate. The nature and limitations of these predictions must be kept in mind when using these results.

This report details the methodology used to determine the likely effects of a catastrophic breach. It's primary sections include this Introduction, Breach Hydrograph Development, Hydrograph Routing, and Likely Inundation Extent and Timing. Additionally, probable inundation maps, a HEC-RAS output table, and flood-frequency computations are included in three appendixes. For the results of this analysis, see the Inundation Extent and Timing section and the probable inundation maps in Appendix A.

Breach Hydrograph Development

As documented 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.

Several methods exist or are being developed for predicting dam breach hydrographs of any failure type. One method, parametric models (offered in NWS DAMBRK & FLDWAV), require a number of variables to be defined including embankment breach width, depth, and side slope, as well as breach initiation time and breach formation time. (Breach initiation time is defined as the time from when flow first begins to flow over or through a dam until the start of the breach formation phase. Breach formation time is defined, for overtopping events, as the time after the overtopping flow has eroded the downstream face and through the crest width, to the point of the upstream face.) The magnitude of peak outflow has been found to be more correlated with failure time for smaller reservoirs, while, conversely, peak outflow has a greater correlation with breach width for larger reservoirs (Wahl 1998). Hence, breach formation time is key in defining the steepness of the floodwave hydrograph. Conversely, breach initiation time is not particularly helpful in a breach routing analysis since it does not significantly effect the peak outflow nor the routing of an actual flood event, but it is most certainly helpful in developing a dam breach warning and evacuation plan. It is important to note that a lengthened breach initiation time due to downstream embankment armoring (figures 4 & 5) can create greater endurance in the case of an overtopping/headcutting scenario, allowing time for the overtopping event to be reduced to only the use of principal and emergency spillways before a full breach occurs. In any case, such methods rely heavily on case study data and these data are sparse due to the limited number of actual dam breaches analyzed.

More physically based models, with principles based upon hydraulics, sediment transport, and soil mechanics are not yet fully developed. One effort of this type is being worked on by NRCS and Agricultural Research Service (ARS) staff at the ARS hydraulics lab near Stillwater, Oklahoma. (For more information, see the ARS web site http://www.pswcrl.ars.usda.gov/embot.htm.)

NRCS engineers and hydrologists typically rely upon a third method, predictor equations, when performing dam breach analyses. This method directly estimates peak discharge

Figure 4: Downstream face of Kearney dam.

Figure 5: Armoring detail, on downstream face.

from an empirical equation based upon case study data of actual breaches. Unfortunately, this predictor method is likely to have a high level of uncertainty associated with the peak discharge estimates (Wahl, 1998).

Despite this shortcoming this predictor equation method, combined with a prediction equation for breach formation time, is used in this analysis. A simple triangular hydrograph was developed, with the volume being equal to the reservoir storage, the peak generated from a predictor equation, and time-to-peak generated by another predictor equation. Peak flow estimates were generated by two methods: from NRCS equations in TR-60 (NRCS 1990), with some supporting documentation in the report *A study of predictions of peak discharge from a dam breach* by Kalkanis, Alling and Ralston (SCS National Bullitin No. 210-6-19); and Dave Froehlich's peak flow equation (Froehlich, 1995).

According to the TR-60 1990 addendum, the criteria for peak flow prediction for an embankment of this height is as follows:

$$Q_{\max} = 1100 B_r^{1.35} \qquad \text{(A)}$$

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

But is not to be less than:

$$Q_{\max} = 3.2 H_w^{2.5} \qquad \text{(B)}$$

And need not exceed:

$$Q_{\max} = 65 H_w^{1.85} \qquad \text{(C)}$$

Where: V_s = reservoir storage at the time of failure (ac-ft)
H_w = depth of water at dam at time of failure (ft)
A = cross-section area at dam at location of breach (ft^2)

Results are provided in Table 1.

Table 1: Breach characteristics, NRCS methodology. WSEL = water surface elevation.

Description	Reservoir WSEL (ft)	Reservoir Volume (ac-ft)	Estimated Peak, Eq. A (cfs)	Estimated Peak, Eq. B (cfs)	Estimated Peak, Eq. C (cfs)
at spillway crest	9194.0	6,131	62,200	75,100	111,400
one foot below dam crest	9200.0	7,223	62,300	96,900	134,500
at dam crest	9201.0	7,405*	63,400	100,800	138,600

*estimated, due to limits of reservoir capacity table

However, the development and peer review of the NRCS predictor equations are not well documented. Due to the uncertainty associated with the use of predictor equations for peak breach flows, another equation, developed by Dave Froehlich (Froehlich, 1995), that is well documented and peer reviewed is also used to predict a peak flow estimate. This equation, which was developed from 22 embankment dam failures and has a R^2 of 0.934, is provided below:

$$Q_p = 0.607 V_w^{0.295} H_w^{1.24}$$

Where: V_w = Reservoir volume at time of failure (m^3).
H_w = Height of water in the reservoir at the time of failure above the final bottom elevation of the breach (m).

Table 2: Peak breach discharge, Froehlich method. WSEL = water surface elevation.

Description	Reservoir WSEL (ft)	Estimated Peak (cms/cfs)
at spillway crest	9194.0	2,194 / 77,500
one foot below dam crest	9200.0	2,607 / 92,100
at dam crest	9201.0	2,678 / 94,600

Due to its greater documentation, high R^2 value, confirmed peer review, relative consistency with the TR-60 equations, and the accuracy of the data used in the breach routing, the results from Froehlich's predictor equation are used in this dam breach analysis.

The breach hydrograph was developed by combining the peak breach flow of 94,600 cfs with the reservoir storage volume of 7405 ac-ft (from the original plans, excluding sedimentation that has occurred since the dam's construction) and a time-to-peak estimate (assumed equal to breach formation time) using Froehlich's method that was provided in Wahl, 1998. This method uses the following equation:

$$t_f = 0.00254 V_w^{0.53} h_b^{-0.90}$$

Where: t_f = breach formation time (hours).
V_w = reservoir volume at time of failure (m^3).
h_b = height of breach (m).

This equation provided a time-to-peak estimate of 0.87 hours. 0.83 hours (50 minutes) was used in this analysis.

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 - 1800 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 dam embankment.

<u>Hydrograph Routing</u>

The Hydrologic Engineering Center – River Analysis System (HEC-RAS) one-dimensional (1-D) 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. Due to the need to route the floodwave through both subcritical and supercritical reaches, the HEC-RAS 3.1 version was used, which allows mixed flow computations.

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:

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

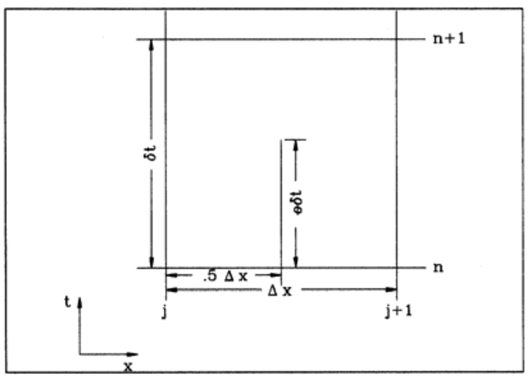
$$S_f=\frac{Q|Q|n^2}{2.208R^{4/3}A^2}$$

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.

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

Kearney Breach Summary

Due to the large number of required cross-sections for model stability on supercritical slopes and the extensive model length from little attenuation in the steep, narrow reaches, the hydraulic analysis was broken into 9 separate but linked analyses: Kearney Creek, South Piney Canyon; South Piney Creek in Story; Piney Creek from Story to I90; 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 Powder River. The entire model length is illustrated in the plan and profile provided in figures 6 and 7.

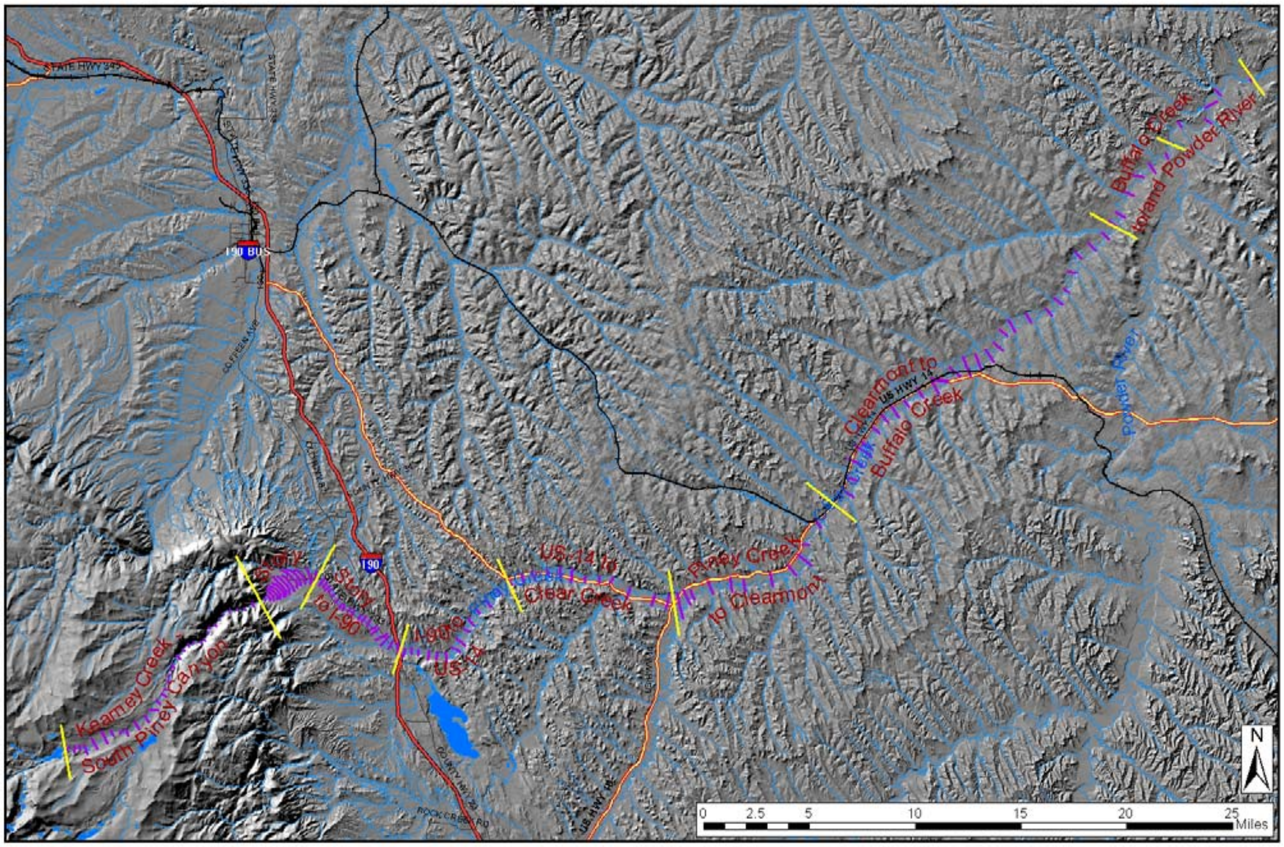


Figure 6: Model plan view, with DEM shaded relief. The yellow slashes separate model reaches and purple lines indicate cross-sections used in the analysis.

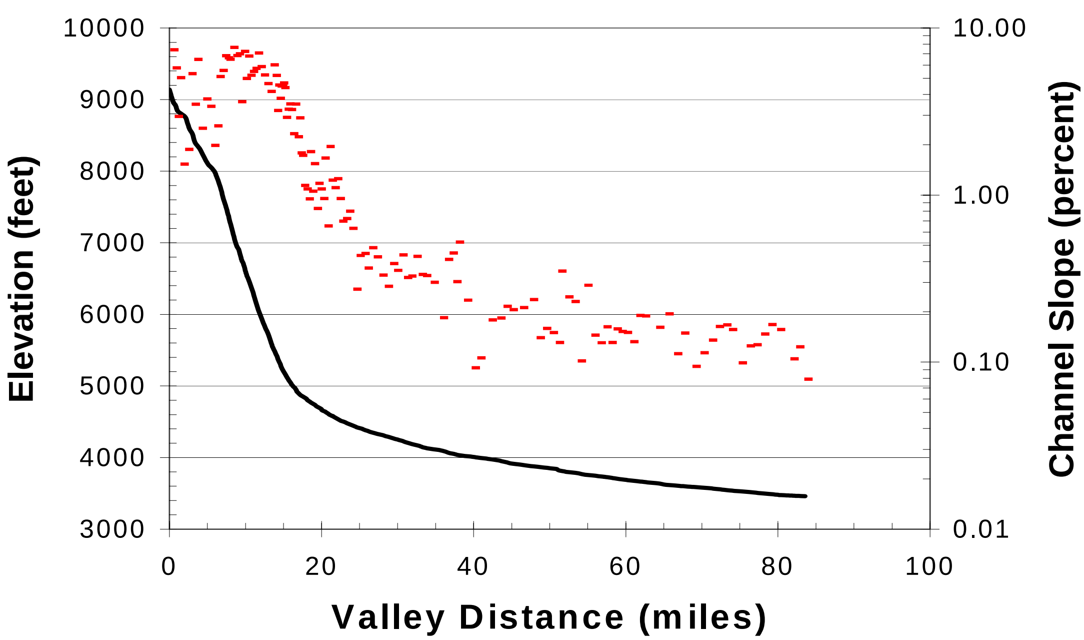


Figure 7: Stream profile, with channel slopes.

Dam Breaches on Steep, Mountain Streams

Dam breaches and other flow events of such extreme intensity occur very infrequently. Unless a breach analysis is for an event that actually occurred, direct calibration and verification of a model is not possible. This is especially a concern since unsteady computer modeling assumes a clear-water condition, which is the not the case for such an extreme event. The steep, wooded stream channel and floodplain (see figures 8 through 10) will be stripped of woody material and the predominantly alluvial bed will be scoured to produce a cascading debris flow through the steep reaches. Additionally, the typical methods and guides for predicting Manning's roughness values are not appropriate for extreme events on these debris saturated streams. To develop the best prediction of timing and extent of a flood wave, it is important to consult analyses of actual breaches in similar geomorphological conditions. The Lawn Lake and Cascade Lake dam failures in Rocky Mountain National Park, Colorado provides just such an opportunity. The U.S. Geological Survey, in cooperation with the Colorado Department of Natural Resources, Office of the State Engineer and the U.S. Bureau of Reclamation, published the report *Hydrology, geomorphology, and dam-break modeling of the July 15, 1982, Lawn Lake Dam and Cascade Lake Dam failures* (Jarrett and Costa, 1984). It is an excellent compilation of the investigation performed on this dam break above a steep narrow canyon of similar morphology to the South Piney.

As described in Jarrett and Costa, 1984, the catastrophic breach of the Lawn Lake Dam, a 26 ft high embankment dam located in Rocky Mountain National Park, 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 cfs. Maximum flow depths ranged from 6.4 ft to 23.8 ft and maximum flow widths ranged from 97 to 1112 ft. Velocity estimates, from a calibrated unsteady flow model, ranged from 3.3 to 12.6 ft/s. Channel slopes ranging from 5 to 25 percent in the canyon of the Roaring River (through a series of bedrock falls and gentle mountain meadows), a 0.7 percent valley slope in Horseshoe Park, and slopes of up to 8% in the Fall River above the town of Estes Park and the Big Thompson River. The slopes of the upper Roaring River reach are a bit steeper but still similar to the South Piney Canyon reach, which also passes through a series of steep bedrock falls and gentle mountain meadows. Observations made of the breach in the Roaring River reach helped develop a better hydraulic model for a breach of the Kearney Dam.

Flood peaks form the Lawn Lake dam failure, depending upon the reach, were 2.1 to 30 times the 500-year flood. This was likely the most severe flood in this area during the Halocene – no events of this magnitude were likely to have occurred since the last glacial retreat 10,000 years ago. The geomorphic effects 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 14 by 17.5 by 21 feet. Interestingly, the well-vegetated tight meanders in Horseshoe Park were not altered by this event.

The catastrophic breach of Lawn Lake dam 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. This same phenomena has been observed in other steep, well-vegetated reaches such as the Johnstown, Pennsylvania dam break of 1889. The flow from Lawn Lake formed a series of large debris dams (15 feet+ heights), composed of trees and boulders. Flow appears to have consisted of a debris flow or torrent that alternated from being subcritical behind a temporary dam to supercritical for a short distance downstream until the debris formed another temporary dam. This cyclic phenomena occurred throughout the steep reach of the Roaring River. Due to this type of flow, the normal methods for determining roughness (Manning's "n") estimates are not appropriate. The unsteady flow model developed for the Lawn Lake dam failure used an initial, uncalibrated, field determined "n" estimate of 0.125, but used a calibrated value 0.20.

To appropriately estimate the amount of attenuation and travel times of a breach hydrograph for the steep, well-vegetated South Piney reach in the Kearny Dam breach analysis, Manning's "n" estimates need to reflect a debris flow with bed scouring, not existing conditions. But with current field conditions not being directly appropriate for choosing roughness values for an unsteady flow model, how should these estimates be made?

Gordon Grant, in the paper *Critical flow constrains flow hydraulics in mobile-bed streams: A new hypothesis* (1997) asserts that in steep (slope greater than 1%) mobile-bed channels, the dynamic hydraulics and bed configurations prevent the Froude number from exceeding 1 for more than short distances and time periods. In channels with a significant ability to adjust it's boundaries (such as the Roaring River and the South Piney), Froude numbers oscillate between 0.7 and 1.3, with an average of 1.0 in the thalweg. Critical flow is maintained by the interaction of the mobile 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 active braided rivers, step-pool streams, laboratory rills, lahar runout channels, and some bedrock channels. Empirical analysis of mobile bed streams indicate that competent (with bed load transport) flows tend to asymptotically approach critical flow. Hence, 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. This technique is being used in paleoflood and historic flood studies in high energy channels (slope greater than 1%), as detailed in *One-Dimensional Estimation Techniques For Discharges Of Paleofloods and Historical Floods*, by Webb and Jarrett in the text Ancient Floods, Modern Hazards: Principles and Applications of Paleoflood Hydrology (2002).

Hence, current hydrologic theory indicates that supercritical flow is not sustainable for significant distances in steep natural channels but that critical flow is common in streams with slopes greater than 1% (Webb & Jarrett, 2002; Grant 1997). Supercritical flow can only be maintained in steep, hydraulically smooth channels, such as concrete channels. With a critical flow assumption, the standard resistance equations of Manning's or Chezy can be replaced with a simple critical depth equation (Grant 1997). But to model attenuation for the steep and forested South Piney, roughness values of the unsteady flow model have simply been chosen to keep the Froude number predominantly between 0.7 and 1.3, with an average of about 1.0 in the channel. This method will provide the most likely estimates of attenuation, travel time (warning time), and flood duration for communities threatened by a breach of the Kearney Reservoir.

The model discussion for this breach analysis is broken down by model reach.

Kearney Creek, South Piney Canyon

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

Cross-sections were developed from 20-ft contours created using a 30 m Digital Elevation Model (DEM). 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. Thirty-six cross-sections were created using the DEM, aerial photography, and a site visit over a total reach length of 73,400 ft (13.9 miles). For model stability, many interpolated cross-sections were created, defined at 20 ft spacing for a total of 3700 cross-sections.

Channel lengths used in the model reflect valley lengths, not actual stream lengths which include small-scale meanders.

Field determined Manning's roughness was estimated from a field visit to the upper third and lower limit of the modeled reach. The extent of Manning's roughness for each cross-section was estimated using aerial photography. With a Manning's values estimated at 0.045 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 75 ft/s and with Froude numbers as high as 2.9. From the above literature search, it is evident that these high values are not possible for a mobile-bed stream. Accordingly, a Manning's "n" was calibrated to maintain a Froude number close to critical for most of the reaches. A global "n" value of 0.15 for the floodplain and 0.105 for the channel achieves this criteria. The resulting model indicates peak flow channel velocities (at non-interpolated sections) ranging from 8.5 to 35.9 ft/s and channel Froude values from 0.45 to 1.23, with 83% of the Froude numbers falling within the 0.7 to 1.3 Froude criteria. Hence, this estimate appears to be reasonable for this stream and is considered the most appropriate for this magnitude of event.

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 4.5%, measured from adjacent contours on the USGS quadrangle, was used.


Figure 8: Typical narrow, steep reach and wider less steep reach on Kearney Creek.

Figure 9: Typical wooded floodprone zone on Kearney Creek.

Figure 10: Typical reach in South Piney Canyon.

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. 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 7) 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 but while still appropriately modeling storage and floodwave attenuation.

If more accurate estimates to the most likely extent of the breach flow is required on the alluvial fan, as well as relatively accurate depth and velocity estimates, finely detailed data of the alluvial fan will need to be gathered. A 2-D hydraulic model, such as SMS FESWMS, will also be necessary since flow from the canyon onto the alluvial fan is a 2-D situation and is only roughly approximated by a 1-D model. 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.

Seventeen cross-sections were developed for this reach. 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 968 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 0.045 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 most of the reaches. Overbank roughness was maintained at 0.15 and calibrated channel roughness ranged from 0.1 in upper sections of the reach to 0.07 in lower sections.

Two bridges exist on this reach and are shown in figure 13. However, these bridges are 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 "n" is used for simplicity.

Three bridges exist on this reach and are shown in figures 16 through 18. Neither the WY-193 nor US-87 bridges are modeled since they will likely be insignificant to flow conveyance due to debris. However, the I-90 bridge, with much greater possible conveyance, is modeled. For simplicity in this analysis, the true vertical alignment of the highway is 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 a "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) is used.

The one county road bridge and the several private bridges within this reach are not modeled due to 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) is used.

Two US-14 bridges and several private bridges cross Piney Creek within this reach. These bridges were not modeled due to the 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 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 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) is 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) is 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

As discussed at the beginning of this report, this analysis provides a prediction of the extent and timing of flooding from a catastrophic breach of Kearney 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 is approximate. 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 these results are approximate. The nature and limitations of these predictions must be kept in mind when using these results.

A catastrophic breach of Kearney Dam, with an initial peak flow of about 94,400 cfs, will inundate the floodplains of 84 miles of stream valley of Kearney Creek, South Piney Creek, Piney Creek, and Clear Creek before finally attenuating to about 14,100 cfs in the Powder River near the Clear Creek confluence. This is a 10-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, several highways and one interstate will be inundated, bridges may be damaged, and many lives could be lost. This potential for harm is why the Wyoming State Engineer's office rates this structure as a "1", a high hazard, and requires the development of an Emergency Action Plan (Wyoming State Engineer's Office, 2000). Identification of the extent and timing of a floodwave is the first part of an Emergency Action Plan.

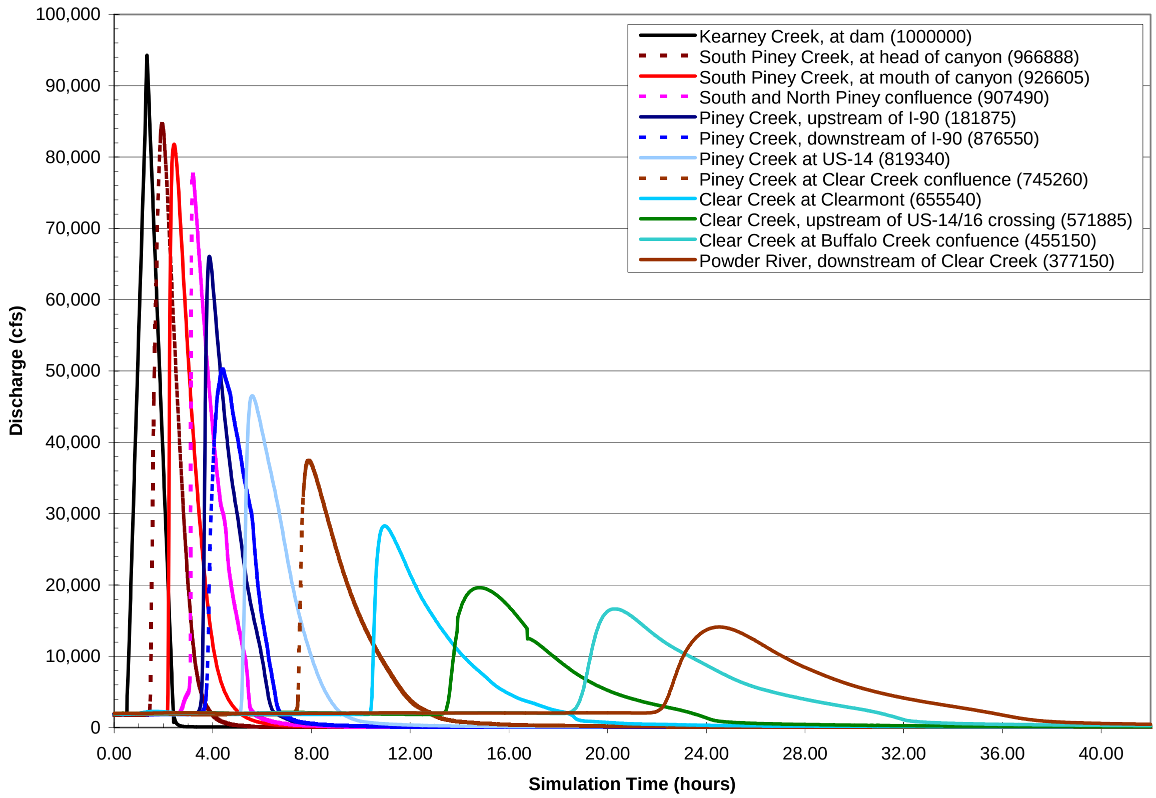


Figure 28: Breach hydrographs.

The probable inundation extent and timing is provided on the inundation maps of Appendix A. These sixteen maps, which were created using ArcMAP 8.2, 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 selected structures that will be threatened by a breach, with the associated times to inundation (from the beginning of the dam breach) provided for convenience. A key to these maps is provided in Figure 29.

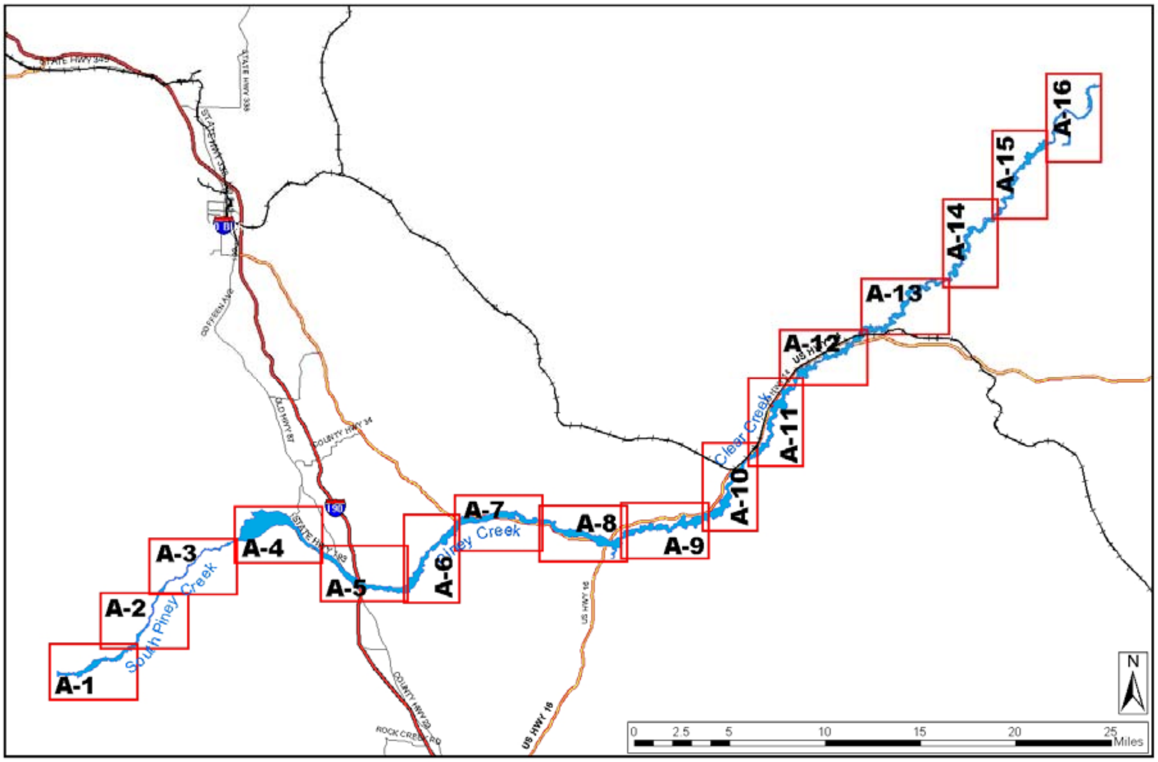


Figure 29: Probable inundation map key.

Based upon the unsteady flow analysis of Kearney Creek, South Piney Creek, Piney Creek, Clear Creek and the Powder River, and combined with observations and analysis performed after the Lawn Lake dam failure in Rocky Mountain National Park and other flow studies performed on steep, moveable-bed streams, the following scenario is presented as the result of a catastrophic breach of Kearney Dam.

A breach of the embankment may occur from either overtopping, piping failure, or embankment sliding or settlement. As a worst case (a completely filled reservoir), a hydrograph with a peak of approximately **94,600 cfs** and a volume of 7405 ac-ft will result. The time-to-peak of this hydrograph is estimated to be 50 minutes. The resulting floodwave will envelope the entire valley bottom of Kearney Creek and South Piney Creek for the entire 13.9 mile reach, to the mouth of South Piney Canyon above Story. At this point peak flow will likely be attenuated to **81,800 cfs**, which is almost 40-times greater than the maximum recorded flow of 2,090 cfs (in 1963) and 35-times greater than the estimated 100-year flow of 2,290 cfs (see Appendix C). Peak flow depths will range from 13 to 35 feet within this reach, with average peak channel velocities ranging from 8 to 37 ft/s and floodplain velocities ranging from 3 to 17 ft/s. The time-to-peak of the floodwave will shorten from 50 minutes at the dam to 18 minutes at the mouth of the canyon. This steep-rising hydrograph may appear as a "wall of water." Due to the steep, wooded, alluvium-bedded nature of this reach, this extreme flow will cause a great deal of bed scouring, with channel erosion in the tens of feet and the stripping of all vegetation within the flood path. As more of the floodway is inundated and stripped, the resulting debris flow will periodically lose it's 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 flood will proceed in this stair-stepped, debris-dam-forming manner until the mouth of the canyon is reached, with the leading edge of the floodwave taking approximately **1.6 hours** to reach the canyon mouth.

This canyon reach is on public land so no residences will 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 **81,800 cfs** at the canyon mouth, maximum flow depths will be approximately 25 feet deep with channel velocities of 32 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 is still steep and vegetation and alluvium is still prevalent - significant scouring and vegetation stripping in certain areas is expected. Cascading debris dams will also be likely forming, creating unpredictable flow paths throughout the width of the alluvial fan. Hence all of the community of Story will be threatened in the event of a breach. Peak flow depths will range from 12 to 25 ft within the Story reach, with average peak channel velocities of 14 to 32 ft/s and floodplain velocities ranging from 4 to 17 ft/s. The time-to-peak of floodwave will lengthen from 18 to 34 minutes. Within this 3.6 mile reach the peak flow is expected to attenuate to **77,500 cfs**, with leading edge of the floodwave taking approximately **2.1 hours** (from the time of failure) to reach the Piney confluence. All 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 **77,500 cfs** to **66,100 cfs** just above I-90 within this 4.1 miles reach. The leading edge of the floodwave will take **2.8 hours** to reach section 881,875, the limit of backwater from the highway embankment at peak flow. Peak flow depths in this reach will range from 15 to 21 feet, with average peak velocities of 15 to 21 ft/s and floodplain velocities ranging from 2 to 9 ft/s. Time-to-peak will be consistently 34 minutes within this reach. Numerous roads, structures, and lives will be threatened.

Due to the extreme magnitude of this event, flow over the I-90 bridge and embankment is expected. Storage from backwater behind the embankment is expected to reduce the peak flow to **50,400 cfs**, with at least 10,000 cfs passing over the highway. Bridge failure due to abutment or pier scour is also a possibility. Danger exists to any vehicles (and occupants) caught in this overflow or failure.

Downstream of the bridge, at section 876,540, flow will be attenuated to **50,200 cfs**, with the leading edge of the floodwave arriving at **2.9 hours**. Within this next reach, from I-90 to US-14 at section 819,340, peak flow will attenuate to **46,500 cfs** with the leading edge of the floodwave arriving at **4.6 hours**. Peak flow depths within this reach will range from 9 to 19 ft, with average peak channel velocities of 9 to 17 ft/s and floodplain velocities ranging from 3 to 8 ft/s. Time-to-peak will stay at a consistent 32 minutes. 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 **46,500 cfs** to **37,400 cfs**, with the leading edge of the floodwave arriving at **6.6 hours**. Peak flow depths will range from 9 to 19 ft, with average peak channel velocities of 6 to 18 ft/s and floodplain velocities ranging from 2 to 9 ft/s. Time-to-peak will decrease from 32 to 28 minutes, then increase to 47 minutes at Ucross. US-14&16, various structures, and lives will be threatened. The 37,400 cfs flow at Ucross is more than 10-times the maximum recorded flow of 3570 cfs (in 1963) and the estimated 100-year flow of 3,620 cfs. (see Appendix C).

Within the next reach, from Ucross to Clearmont, flow will attenuate from **37,400 cfs** to **28,200 cfs**, with the leading edge of the floodwave arriving at Clearmont in **9.8 hours**. 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 initially be at 47 minutes at Ucross, decrease to 33 minutes, then increase to 41 minutes at Clearmont. 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 **28,200 cfs** to **14,900 cfs**, with the leading edge of the floodwave arriving at the Powder River at **20.3 hours**. Peak flow depths will range from 8 to 17 ft, with average peak channel velocities of 3 to 28 ft/s and floodplain velocities ranging from 1 to 11 ft/s. Time-to-peak will increase from 41 minutes to 151 minutes. The 14,900 cfs flow in Clear Creek near it's mouth is greater than the maximum recorded flow of 9600 cfs (in 1954). This flow is less than the estimated 200-year flow of 15,700 cfs and is approximately equivalent to a 150-year event. 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 **14,100 cfs**, with the leading edge of the floodwave arriving at section 377,150 at **21.4 hours** with a 170 minute time-to-peak. Peak channel flow velocities will range from 5 to 9 ft/s with a little bit of floodplain flow providing velocities of 1 to 3 ft/s. According the Powder River streamgage this 14,100 cfs flow is a 10-year event and has been surpassed in 9 of the last 72 years. This flow will have minimal potential for danger to structures and lives within the sparsely-populated Powder River valley.

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