Willow Park Reservoir PMP Analysis

Johnson County, Wyoming

January 2006

Upper Watershed

Story

Willow Park Embankment

NRCS

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Willow Park Reservoir: PMP Analysis

Job Number: WY0505
Short Job Description: Willow Park probable maximum precipitation analysis.
Location: Johnson County, Wyoming above the community of Story, Wyoming.

Summary: Predictions have been made of the conveyance capability of Willow Park Reservoir's outlet works to convey the flood response to a maximum probable precipitation event.

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INTRODUCTION

This report details the methods and results of a Probable Maximum Precipitation (PMP) analysis for the Willow Park Reservoir of Johnson County, Wyoming. The analysis consists of a hydrologic model that simulates a synthetic PMP event, produces runoff from sub-basins within the watershed, and routes the storm flow through channels, natural lakes, and reservoirs to the outflows of Willow Park Reservoir. This report is intended for use by the Natural Resources Conservation Service (NRCS), Wyoming regulators, and the Willow Park Reservoir Company.

The Willow Park Reservoir is located on South Piney Creek at an elevation of 8600 feet in the Bighorn Mountains above the community of Story. Average precipitation within the reservoir's 33.8 square mile watershed varies from 25 to 39 inches, according to PRISM (Figure 1). The embankment 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 reservoir storage is 5123 ac-ft. These volumes do not account for sediment accumulation since construction in the late 1950's.

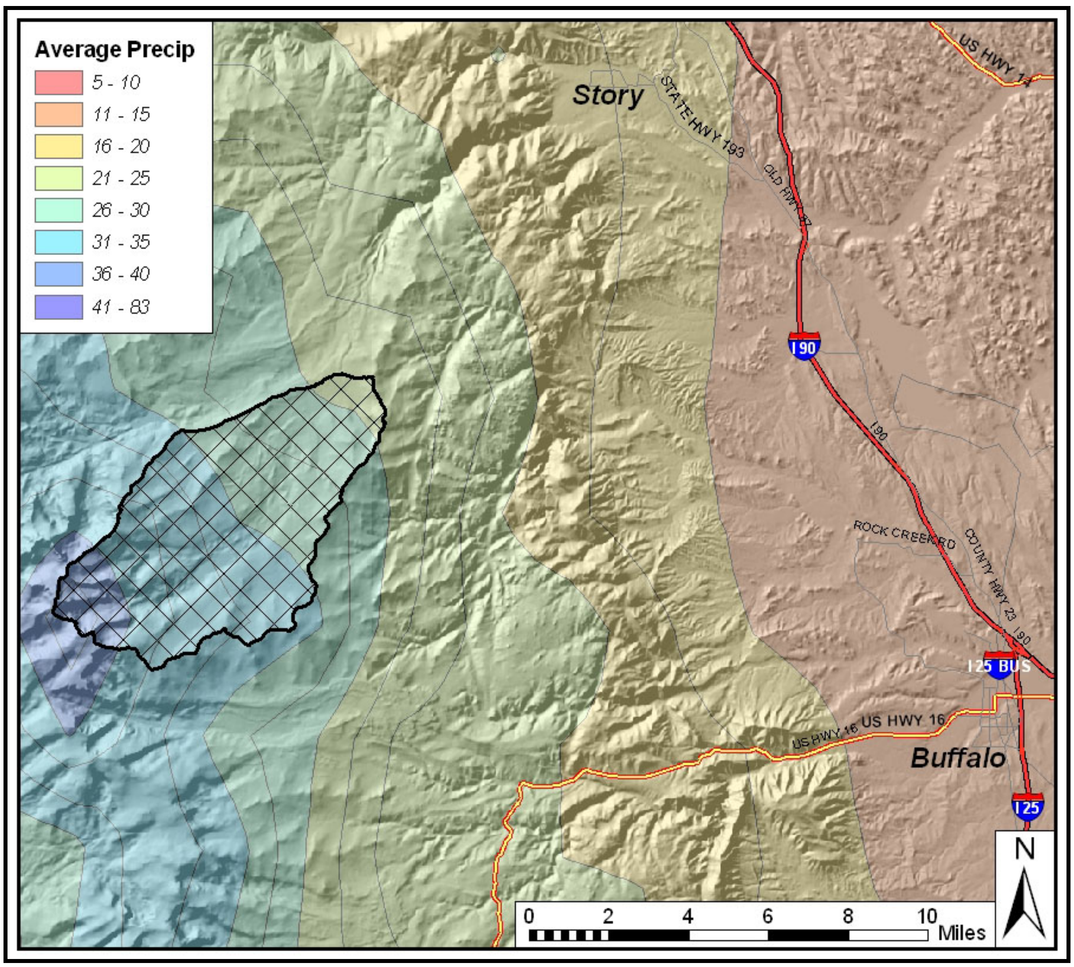

Figure 1: Region of analysis, mountainous portion. Shaded relief and average annual precipitation (PRISM) estimates are shown. The Willow Park watershed is shown cross hatched.

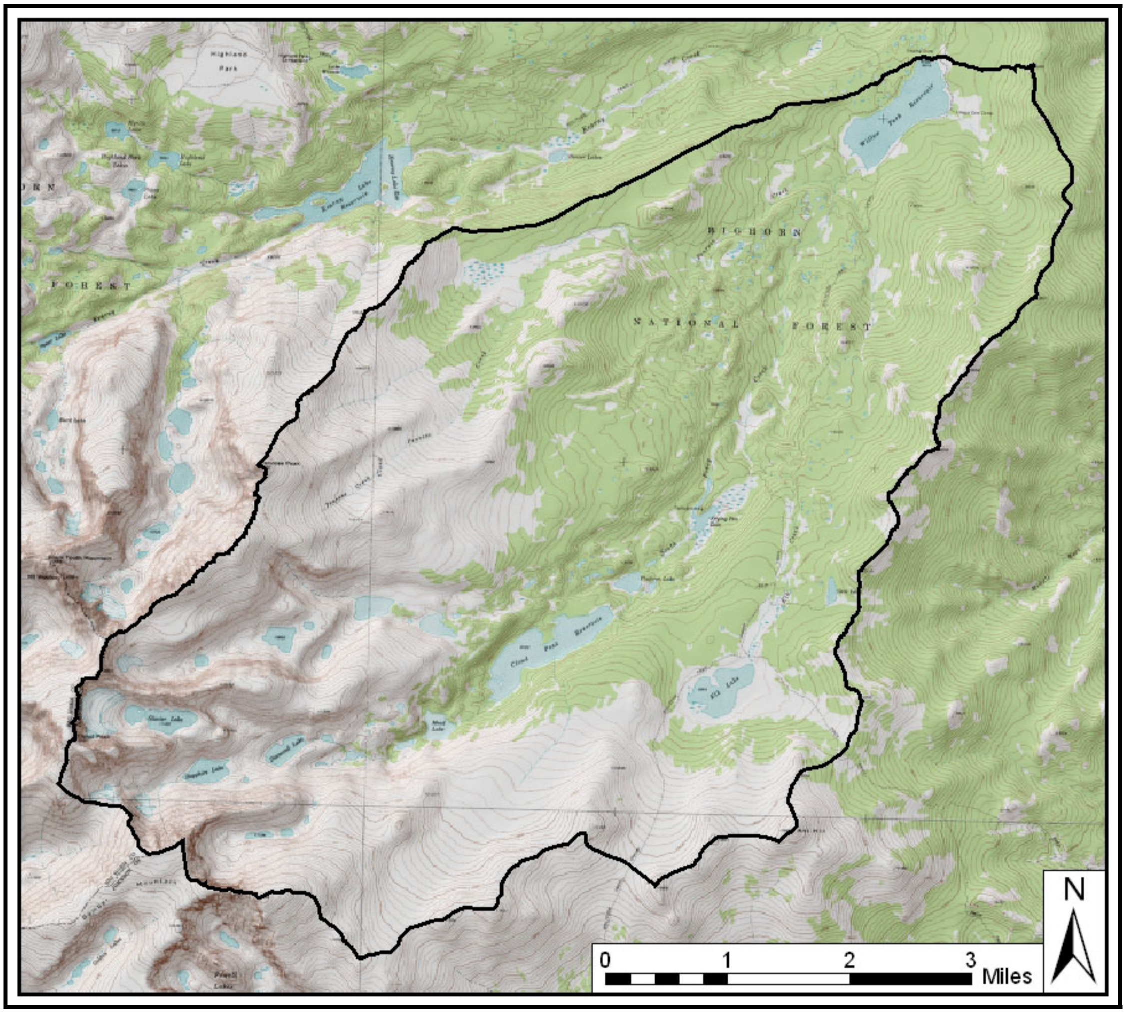

Figure 2: Reservoir watershed, superimposed on a USGS 7.5-minute quadrangle.

This analysis is a dams-in-series situation – 7.9 square miles of the watershed is controlled by Cloud Peak Reservoir.

The Willow Park watershed is a high-elevation watershed with steep, above treeline, upper zones that have the potential for rapid response to large summer rain events. Lower elevation zones are forested and have less potential for runoff for typical events. However, these forested areas will also produce a great deal of runoff in the extreme rainfalls modeled in this analysis. Figures 2 and 4 provide USGS topography and color infrared photography of the watershed. Figure 3 is a photograph of the upper Cloud Peak watershed.

This analysis assumes that the PMP event occurs after the snow-melt season has ended and that there is no significant rain-on-snow component to the PMP response. Additionally, the NRCS Curve Number method was used in this analysis. This method has a number of limitations in forested and alpine tundra watersheds – the results of this analysis need to be considered approximate.

This report details the methods used to determine the probable maximum flood (PMF) response to the PMP. Results are provided and conclusions are drawn from these results.

Figure 3: Upper Cloud Peak watershed.

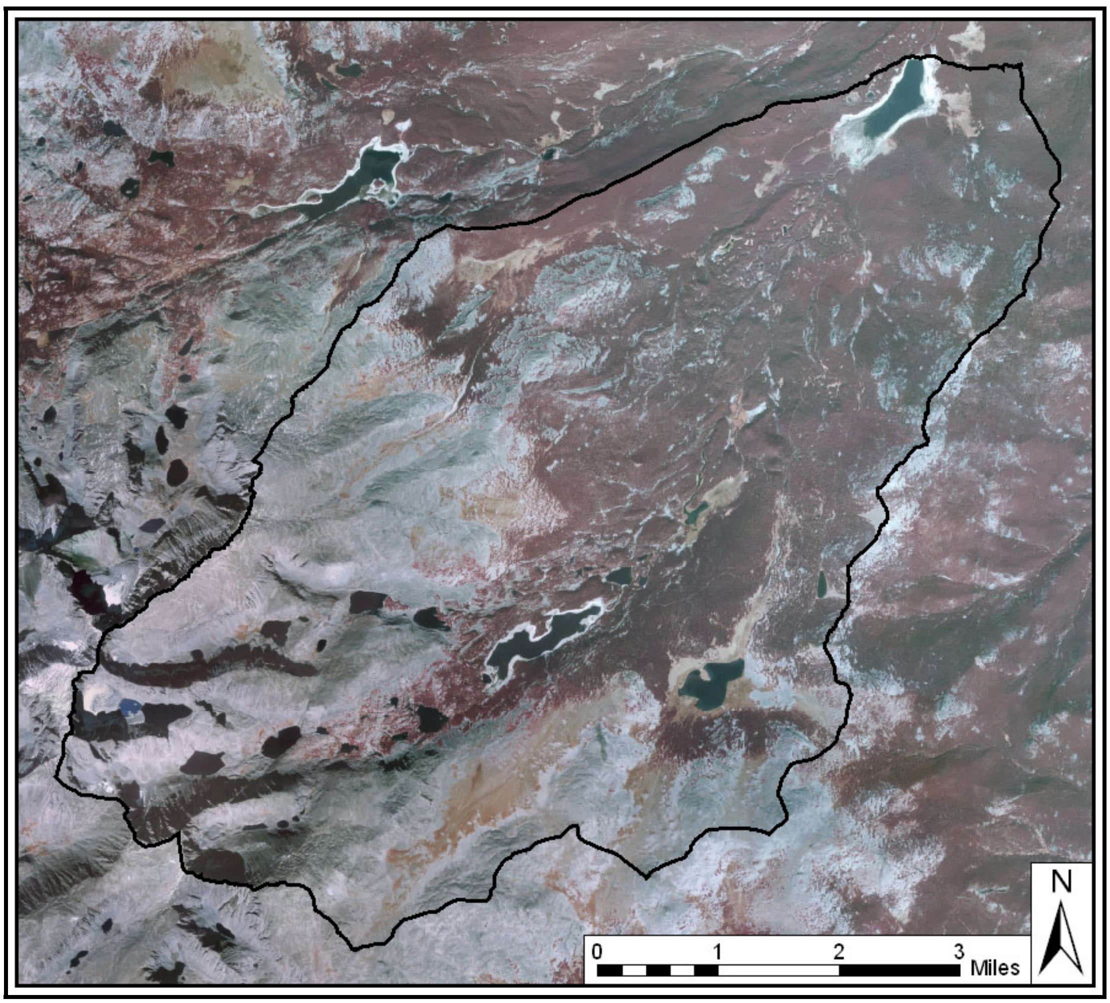

Figure 4: Willow Park Reservoir watershed, superimposed on 1-meter color-infrared ortho photography.

PROBABLE MAXIMUM PRECIPITATION

The National Oceanic and Atmospheric Administration (NOAA) has responsibility for providing Probable Maximum Precipitation (PMP) estimates. A PMP is the theoretical greatest depth of precipitation for a given duration and area that is physically possible (Hansen et. al. 1988). HMR-55A, Probable Maximum Precipitation Estimates - United States Between the Continental Divide and the 103rd Meridian (Hansen et. al. 1988), is the applicable publication detailing the recommended PMP estimate for the Willow Park watershed.

In the HMR-55A study, as well as other PMP studies, two storm types are assessed: the short-duration local storm (intense, small area, short duration) and longer, more general storms. HMR-55A assigns PMP values for local storms, a storm restricted in time and area to less that 500 mi^2 and less than or equal to six hours in length. General storms, that is, a storm event which produces precipitation over larger areas and duration of longer than six hours and is associated with a major synoptic weather feature (Hansen et. al. 1988), provide PMP values for events longer than 6 hours. Due to this local/intense versus longer/generalized differentiation in this PMP study, two storm lengths are used in this analysis: a 6 hour and 24 hour storm. This is also needed for current NRCS TR-60 criteria (NRCS 2005a).

The generalized PMP for a 10-square mile watershed area, as determined from HMR-55A for application to the Willow Park watershed, is 19 inches for the 6-hour, 10 mi^2 event and 31 inches for the 24-hour, 10 mi^2 event. This 24-hour value is an approximate aerial average.

The 10 mi^2 events were then adjusted for the watershed area of 33.9 mi^2. This area falls within subregion A, the Missouri River basin in the orographic subunit (Hansen et. al. 1988). From the A orographic subunit, the percent reduction of the 10 mi^2 PMP is 94% for the 6-hour and 97% for the 24-hour event. This corresponds to 17.9 inches for the 6-hour event and 30.1 inches for the 24-hour event. To reflect the accuracy of the original values, **18 inches** and **30 inches** were used in this analysis for the **6-hour** and **24-hour** PMP values (respectively). Cumulative precipitation using a TR-60 distribution (NRCS 2005a), for the 6-hour and 24-hour simulations, has been provided in Figure 5.

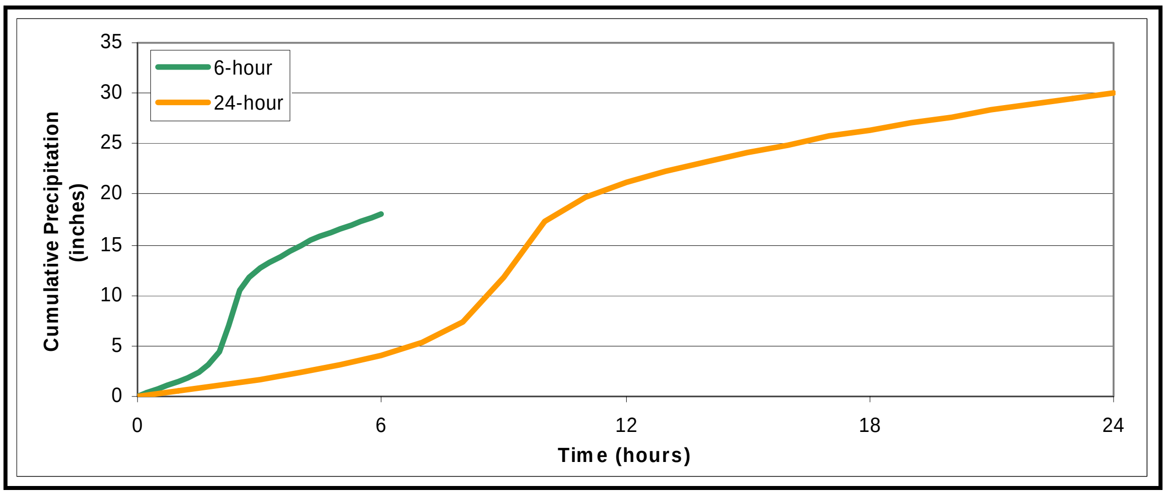

Figure 5: Cumulative precipitation (inches) for the 6-hour and 24-hour PMP events.

HYDROLOGIC MODELING

Hydrologic modeling was performed using the program HEC-HMS (version 2.2.2), a model developed by the U.S. Army Corps of Engineers' Hydrologic Engineering Center. The NRCS curve number (CN) technique for estimating direct runoff from rain events in ungaged watersheds was used in this analysis.

Model Form

As documented in NRCS (2004b), the NRCS method for estimating direct runoff from individual storm rainfall events follows the following form:

$$Q = \frac{(P - I_a)^2}{(P - I_a) + S} \qquad \text{if } P > I_a$$

$$Q = 0 \qquad \text{if } P \leq I_a$$

Where Q is the depth of runoff (inches), P is the depth of rainfall (inches), I_a is the initial abstraction (inches), and S is the maximum potential retention (inches). The derivation of this equation is not physically based but does respect conservation of mass (NRCS 2004b).

The Curve Number is defined as:

$$CN = \frac{1000}{10 + S}$$

The initial abstraction was initially described and has traditionally been used as:

$$I_a = 0.2S$$

This relationship is fairly poor, as Figure 10-1 in NRCS (2004b) illustrates.

A schematic of the model used in the Willow Park analysis is provided in Figure 6.

CN Development

The CN method is a simple and widely used technique for estimating a stream hydrograph at the outlet of a watershed. Documentation is provided on the method in the NRCS National Engineering Handbook, Section 4, Hydrology, Chapters 9 and 10 (NRCS 2004a, NRCS 2004b), in Rallison (1980), as well as numerous other publications. However, little quantitative information has been published of the data base on which it was developed (Maidment 1992) and many of the curves used in the development have been misplaced (Woodward 2005). The method was developed for rural non-mountainous watersheds in various parts of the United States, within 24 states; was developed for single storms, not continuous or partial storm simulation; and was not intended to recreate a specific response from an actual storm (Rallison, 1980). This latter point is disconcerting but understandable considering that typical condition CNs are being applied to the real-world variability of soil moisture, spatial precipitation variability, variation in precipitation intensity, and interception. Most fundamentally, the conceptual foundation of the CN technique is disconnected with actual streamflow generating processes during more-frequent small to moderate rain events. The CN is a simple watershed-scale method that gives simplified results at a watershed outlet for larger events. For a theoretical extreme storm such as a PMP, the method is appropriate and thought to give good results (Woodward 2005).

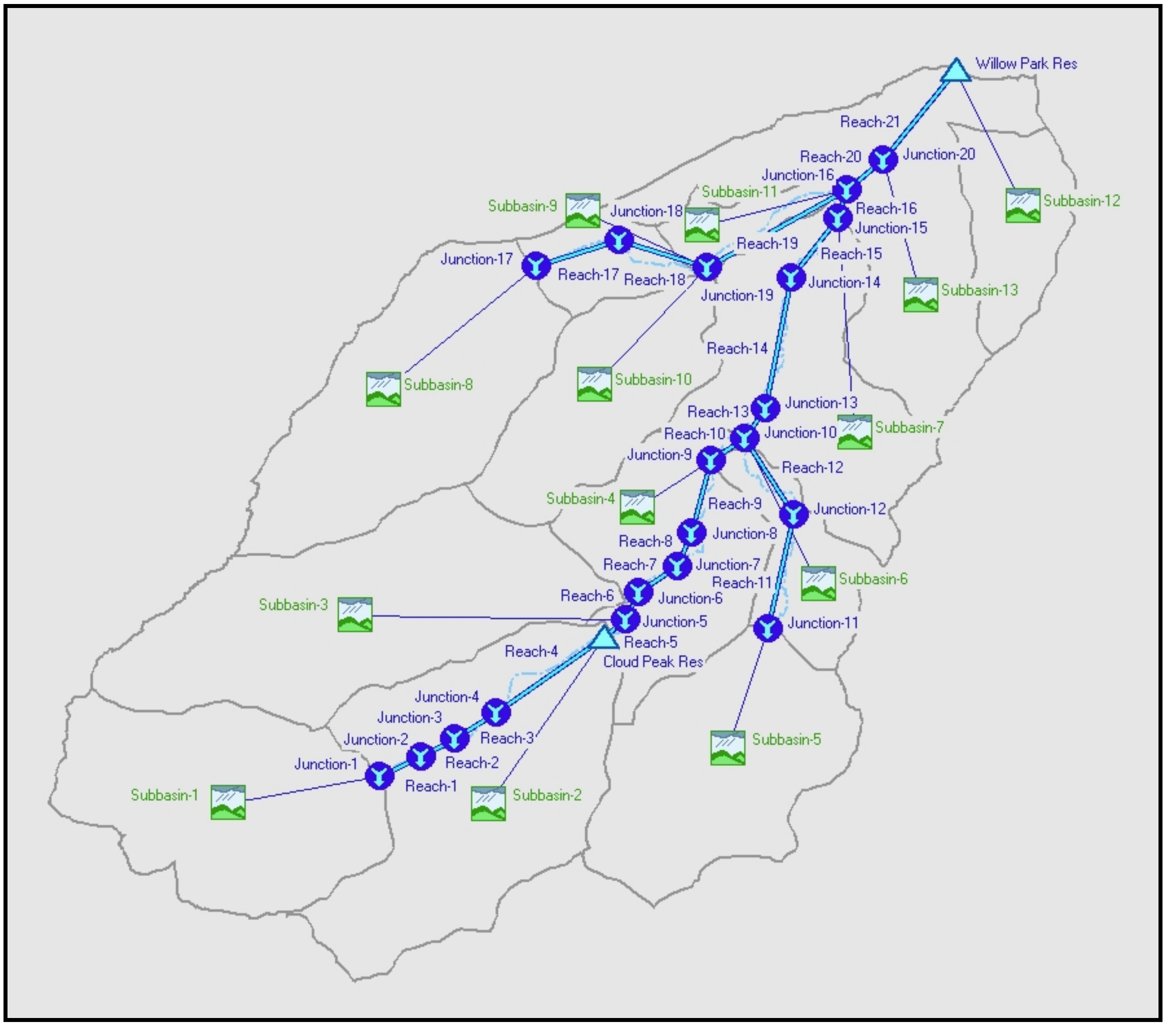

Figure 6: Schematic of hydrologic model.

CN Limitations

It is important to recognize the limited applicability of the NRCS curve number technique to forested and alpine-tundra mountainous watersheds, such as Willow Park. But this is not just a problem with the CN method – forested watersheds are, in general, extremely difficult to model. Existing technology and data availability does not provide accurate methods of developing estimates of runoff from forested watersheds such as Willow Park's. But hydrologists must still make an estimate of the flow that results from such an event as a PMP – designers and regulators need to make informed decisions of the adequacy and safety of a high hazard structure such as the Willow Park embankment dam.

According to NRCS (2004a), the "combination of a hydrologic soil group (soil) and a land use and treatment class (cover) is a hydrologic soil-cover complex". Through catchment-scale empirical studies, each with one complex of hydraulic soil group and cover, runoff curve numbers have been assigned to complexes (Mochus, 1964). Through work done by the USFS almost 50 years ago, it is stated in NRCS (2004a) that in the forest-range regions of the Western United States, soil group, soil type and cover density are the principal factors in estimating CN. Graphs are provided to estimate runoff curve numbers given soil groups and cover density. However, it may not be this simple. One of the key issues to understand in forested watersheds is when and how a well-vegetated forest produces runoff. The forested catchments in the Willow Park watershed have a distinct lack of swales. Smaller, more frequent, rain events do not produce runoff from much of the surface areas of the watershed since the forest will intercept much of the rainfall, while the remainder gets infiltrated into the ground litter. Large events will produce runoff from the forested areas. But where is the line drawn between smaller events that don't produce runoff from forested catchments and events that do? Assuming a global parameter is at all appropriate, what is the proper "initial abstraction" for such watersheds?

Very large precipitation events that are a significant proportion of the PMP have occurred in forested mountainous terrain and produced very large flows. The Big Thompson flood in Colorado is one such event, where up to 12 inches of rain fell in 4 hours (Hansen et. al. 1988), flooding a canyon section of the Big Thompson River and killing 139 people. Such examples encourage the assumption that much of the forested land can produce overland flow in such events and should thus be assigned a CN that would produce a large response. This is the key assumption of this analysis – that the forested watersheds will produce significant overland flow for large to extreme rain events and that the uncertainty in our understanding of initial abstraction is less important.

CN Application

To develop appropriate CN for this mountainous watershed, soils and vegetation mapping was obtained from the U.S. forest Service (USFS). A soil survey from the USFS was also obtained and, with the assistance of an ongoing NRCS soil survey of this area, hydraulic soil group classifications were assigned. Vegetation mapping of the watershed is provided in Figure 7, soil mapping in Figure 8, and hydraulic soil group classification in Figure 9. Vegetation in the watershed includes water, forbs, grasses, lodgepole, ice, rock, rock/soil, spruce/fir, and willow.

Watershed Soil Descriptions

Descriptions of the soils in the Willow Park reservoir watershed, as reported by the USFS, are provided below. When available, short NRCS descriptions are also provided.

- **(10/993)** PIC0/VASC **Agneston-Granite-Rock** outcrop associated on montane and subalpine mountain slopes, 5 to 50 percent slopes. Alluvium and/or colluvium derived from granite.
- **(11)** PIEN/VASC **Agneston-Leighcan** association on montane and subalpine mountain slopes, 5 to 30 percent slopes.
- **(13)** ALPINE **Cirque Land**, 10 to 130 percent slopes.
- **(37)** ALPINE **Rubble land** on subalpine and alpine mountain slopes, 5 -50 percent slopes.
- **(16/981)** SALIX/JUCO **Cryaquolls** on montane and subalpine mountain slopes, 0 to 5 percent slopes. Alluvium derived from igneous and sedimentary rock.
- **(25/997)** FEID/CAREX **Lucky-Burgess-Hazton** association on montane and subalpine mountain slopes, 2 – 30 percent slopes.

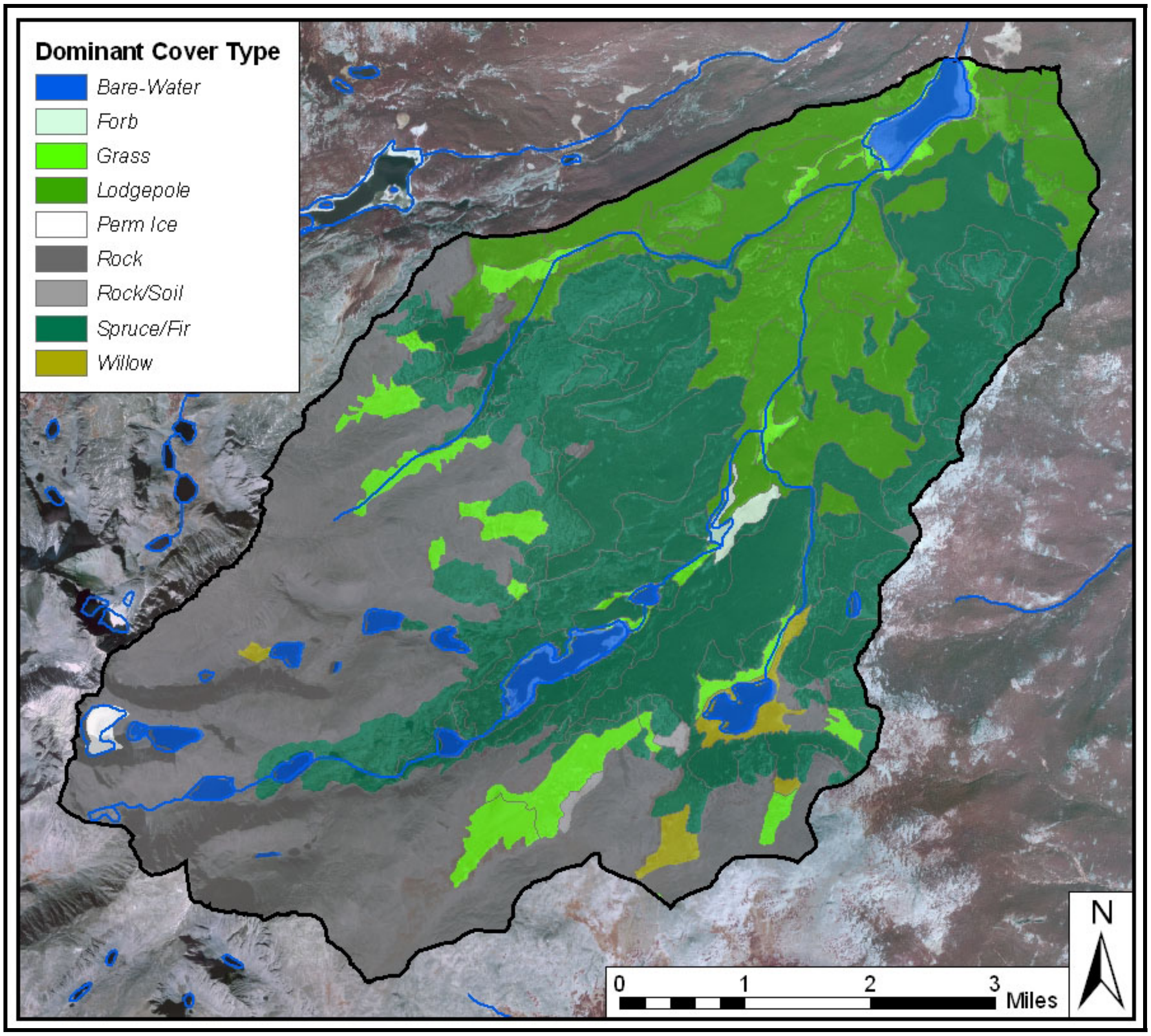

Figure 7: Dominant vegetation of the Willow Park Reservoir watershed, courtesy of the USFS.

- **(26)** ALPINE **Mirror-Teewinot-Bross** association on subalpine and alpine mountain slopes, 2 – 40 percent slopes.
- **(31/994)** PICO/VASC **Rock outcrop-Agneston-Rubble** association on montane and subalpine mountain slopes, 5 to 60 percent slopes. Alluvium and/or colluvium derived from granite.
- **(33)** ALPINE **Rock outcrop-Mirror-Teewinot** association on subalpine and alpine mountain slopes, 5 – 35 percent slopes.
- **(36)** ALPINE/PEIN/VASC **Rock outcrop-Teewinot-Agneston** association on subalpine mountain slopes, 5 to 35 percent slopes.
- **(19B)** PICO/PIEN/VASC **Frisco - Troutville** association on montane and subalpine glacial moraines, 2 to 40 percent slopes.
- **(19A)** PICO/VASC **Frisco - Troutville** association on montane and subalpine glacial till, 2 to 40 percent slopes.

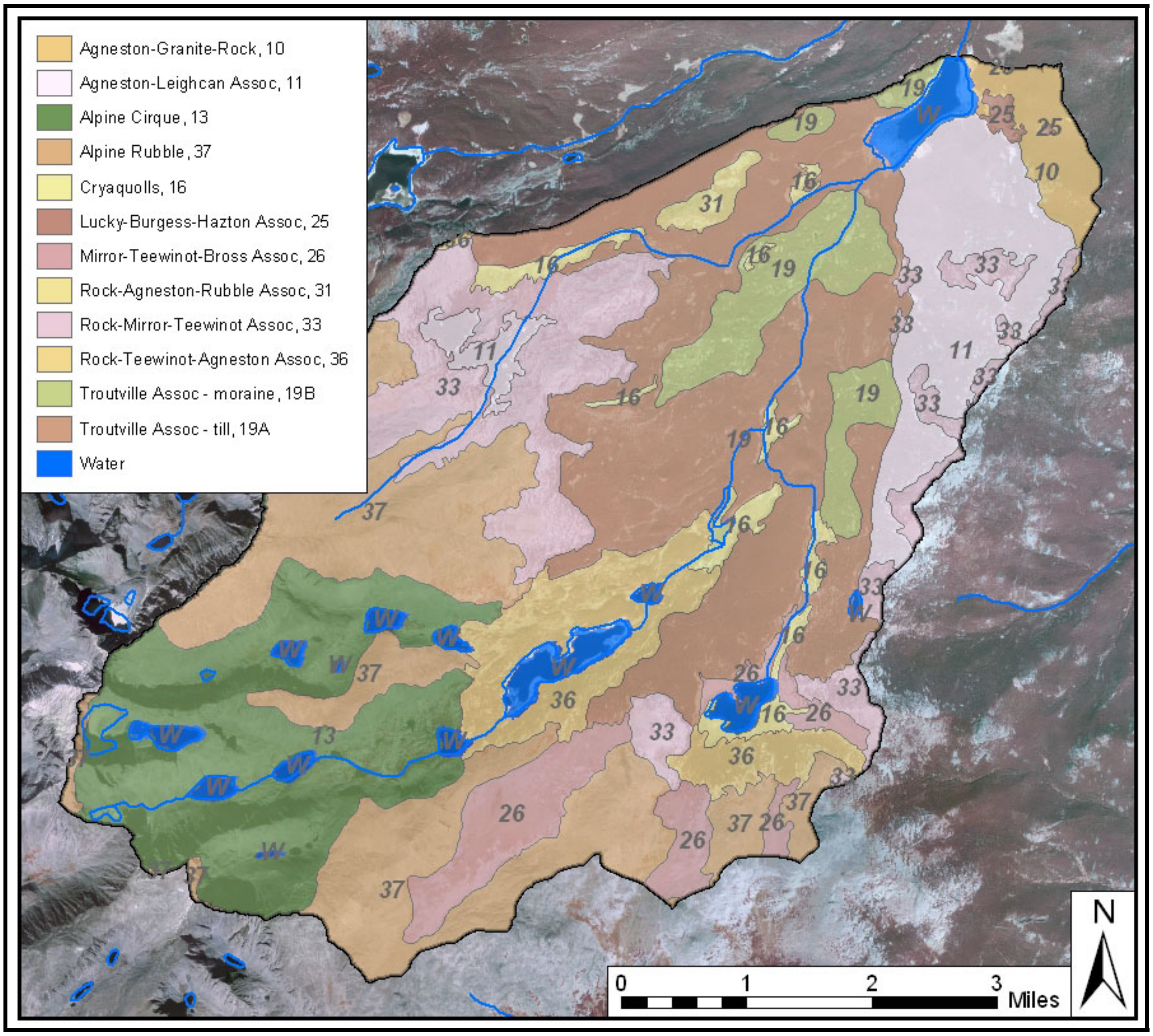

Figure 8: Soils types, courtesy of the USFS.

Hydrologic Soil Group Classification

Hydrologic groups for the soils of the watershed, as provided by an ongoing NRCS soil survey, are provided below. The dominant classification that is used in this hydrologic analysis is in **bold.** Since the NRCS soil survey is incomplete, a number of the hydrologic group classifications needed to be estimated from USFS permeability descriptions or assumed from general descriptions.

- **(10/993)** Agneston, Granile, Rock Outcrop: **C**, B, D.
- **(11)** Agneston, Leighcan: **C**, B (from USFS description).
- **(13)** Cirque Land: **C** (assumed).
- **(37)** Rubble land: **C** (assumed)
- **(16/981)** Cryaquolls: **D**.
- **(25/997)** Lucky, Burgess, Hazton: **C**, **C**, D.
- **(26)** Mirror, Teewinot, Bross: **B, B**, C (from USFS description).
- **(31/994)** Rock outcrop, Agneston, Rubble: D, **C**, **C**.

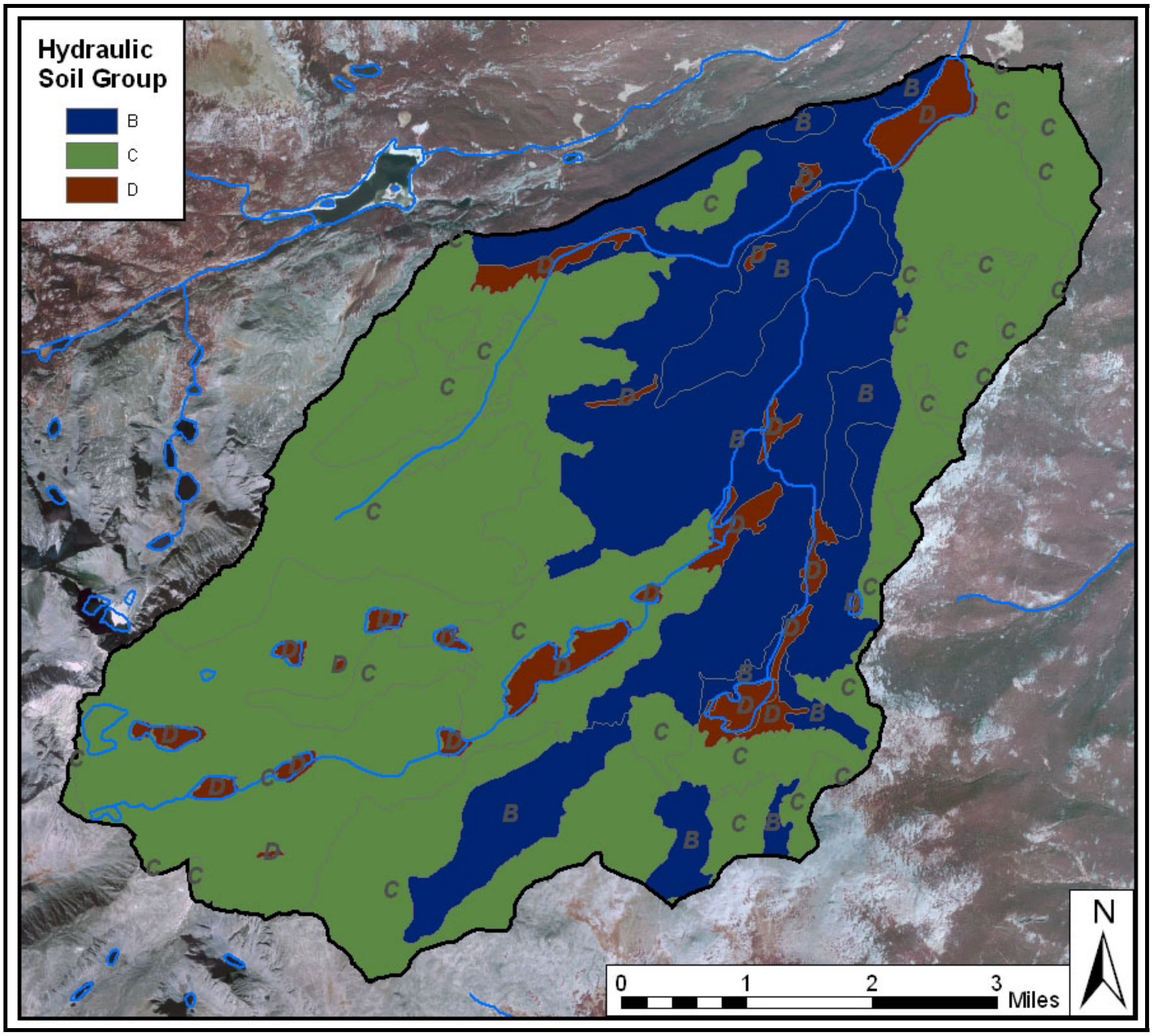

Figure 9: Hydrologic soil group classification.

- **(33)** Rock outcrop, Mirror, Teewinot: D, **C**, B (from USFS description).
- **(36)** Rock outcrop, Teewinot, Agneston: D, B (from USFS description), **C**.
- **(19B)** Frisco, Troutville moraine**: B**, **B** (from USFS description).
- **(19A)** Frisco, Troutville till**: B**, **B** (from USFS description).

CN Assignments

Table 1 provides a list of landuse type and the associated estimated CN values for various conditions.

A number of notes should be made regarding these CN selections: (1) Water surfaces are subject to direct runoff – CN = 98 (NRCS 2004); (2) The Forb cover area was visited and found to have wetland characteristics, CN taken from pasture/grassland/range cover type with poor hydrologic condition; (3) For grass, CN taken from meadow-continuous grass cover type; (4) For Lodgepole and Spruce/Fir, “woods” in good condition was used; (5) Under warmer, rain condition, the snow/ice field is assumed to allow infiltration; (6) The rock and rock/soil land conditions are alpine tundra – herbaceous (grass, weeds, lowgrowing brush) type with fair and good hydraulic condition assumed for CN; (7) For Willows, a "brush" cover type was used.

Table 1: CN Assignments.

Value	Description	Hydrologic Condition	A	B	C	D
B	Bare-Water	----	98	98	98	98
F	Forb	poor	68	79	86	89
G	Grass	good	30	58	71	78
T218	Lodgepole	good	30	55	70	77
BIC	Perm Ice	----	60	60	60	60
BRO	Rock	fair	----	71	81	89
BRS	Rock/Soil	good	----	62	74	85
T206	Spruce/Fir	good	30	55	70	77
S921	Willow	good	30	48	65	73

With application in non-agriculture forested watersheds, the CN method is being used outside the watershed landuse types that the method was developed for. These CN values are approximate.

Catchment Composite CNs

The soils and vegetation shapefiles were merged, with the Table 1 CN assignments applied to the merged file to provide a CN for each of the 755 resulting polygons. Table 2 list the composite CNs, as well as other characteristics, for the 13 sub-basins illustrated in Figure 6.

Table 2: Composite CNs and other watershed characteristics for the Willow Park sub-basins illustrated in Figure 6.

Sub-Basin ID	Area (mi^2)	Composite CN	Initial Abstraction (inches)	Lag Time (minutes)
1	3.495	80.4	0.49	23
2	4.420	75.6	0.65	29
3	4.378	78.5	0.55	39
4	2.402	61.0	1.28	28
5	3.084	71.0	0.82	42
6	1.089	60.3	1.32	21
7	3.095	58.2	1.44	31
8	3.644	75.4	0.65	32
9	0.757	62.2	1.22	12
10	2.113	61.1	1.27	24
11	0.525	59.1	1.39	13
12	1.196	70.0	0.86	19
13	3.625	67.6	0.96	21

Initial Abstraction

Recently, it has been suggested that the use of an initial abstraction, I_a, of 0.2S, where S is the maximum potential retention after runoff begins, is too high. Instead, it has been found that the use of 0.05S is more appropriate (NRCS 2005b). To make use of the most-recently available information, it would have been preferred to use an I_a of 0.05S. However, since changing the I_a assumption would change the CNs listed in NRCS (2004a), an I_a of 0.2S was used in this analysis. This initial abstraction estimate is very approximate for the forested portions of the Willow Park watershed.

Lag-Time Estimates

Using the physically-simplified CN methodology, precipitation that is not initially abstracted or infiltrated becomes excess precipitation that flows down-gradient to the sub-basin outlet, which is modeled using a transform method. Actual hydrologic processes are more complicated than this but the use of CN concept is a necessary simplification to model this watershed. HEC-HMS allows the use of transform methods to route excess flow to the mouth of each sub-basin, but this method is not preferred since the CN technique is typically used with a time-of-concentration estimate. This latter method was used in this analysis.

The methods documented in SCS 1972, NEH Section 4, Chapter 15, were used to compute lag estimates for each sub basin.

Stream Reach Network

Stream reaches within the Willow Park watershed are typically steep boulder-bed streams with interspersed small lakes and wetlands, as shown in Figure 10.

Figure 10: Typical stream reaches in the Willow Park watershed.

To model travel time and attenuation, stream reaches were developed to route the flow from each sub-basin through the watershed to Willow Park Reservoir. The Muskingum-Cunge method was used in the model. Due to model requirements, the stream network was designed so that each reach had a consistent slope. For each reach, simple trapezoidal cross-sections were developed and energy slopes and a Manning's n was designated.

Water bodies along the reach network were simulated by using the average bottom width, with a mild assumed energy slope of 0.001 feet/feet and a Manning's n of 0.07 for a sluggish reach with weeds and deep pools (Brunner and Goodwell 2002).

Manning's n Estimates for Steep Reaches

It has been shown in hydraulic research that supercritical flow in steep sloped mountainous streams occurs only for short lengths and duration and, instead, critical flow may be much more dominant. In practice, this situation impacts the appropriate selection of Manning's *n* in a hydraulic model. The selection of Manning's *n* for lag-time and hydraulic modeling in this analysis were based on this philosophy, as described in this section.

Dam breaches and other extreme flow events such as the Probable Maximum Flood 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 extreme flows 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 cfs. 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 necessitated the calibration of *n* to 0.20.

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

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

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.

Manning's n Selection Using Froude Number

This issue of the selection of the appropriate steep-channel *n* values is relevant for the selection of the time of concentration and lag time in each of the subcatchments, in the routing of hydrographs within the HEC-HMS model, and to route the Cloud Peak dam failure to Willow Park reservoir. These reaches are all alluvial-bed streams. The Cloud Peak Reservoir dam breach analysis (Yochum 2005) has previously identified initial velocities that were predicted to be as high as 48 ft/s, with Froude numbers as high as 2.59 that were then calibrated to attain more reasonable velocities and Froude numbers. The calibrated Manning's *n* values were used in the HEC-HMS PMP model, both for the identical reaches and for application to other similar reaches.

Cloud Peak Reservoir

Storage, attenuation and outflow from Cloud Peak Reservoir were modeled in this PMP analysis using as-built drawing dimensions. A reservoir was included in the model, emergency spillway dimensions were entered, and a table of elevation and storage was coded. The gated principal spillway was assumed to be closed in the model – the only flow through the reservoir was the emergency spillway (Figure 11). Also, the initial elevation of the reservoir was assumed to be 9715 feet, at a volume of 1784 acre-feet. This is the volume of the reservoir half full, to the crest of the emergency spillway. Photos of the Cloud Peak embankment are provided in Figure 12.

Figure 11: Emergency Spillway, Cloud Peak Reservoir. Capacity $\approx$ 1900 cfs.

Cloud Peak Reservoir will overtop if a PMP event occurs in the watershed. In the case of significant overtopping, the embankment will likely fail. The reservoir was modeled to simulate attenuation that it will provide to Willow Park Reservoir. The embankment was simulated to both not fail and fail in different simulations - both scenarios were modeled within HEC-HMS.

For the case of failure, geometry of the failure is similar to those used in the dam breach analysis (Yochum 2005). The breach bottom elevation was 9701.5 feet with a bottom width of 40 feet and side slope of 1.5:1, horizontal to vertical. Both the bottom width and side slopes are the limits of what embankment section allows. The trigger elevation was 9732.5 (0.5 feet above the crest of the embankment) and the development time was 1.14

hours. The development time estimate was created using Froehlich's regression equation (Froehlich 1995). This method uses the equation

$$t_f = 3.84 V_w^{0.53} h_b^{-0.90}$$

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).

Figure 12: Cloud Peak reservoir embankment.

Willow Park Reservoir

Storage, attenuation and outflow from Willow Park Reservoir (Figure 13) were also modeled using as-built drawing dimensions. A reservoir was included in the model with a table of elevation and storage included. The uncontrolled principal spillway was modeled as well as the emergency spillway (Figure 14). The principal spillway has a crest elevation of 8616.5 feet and the emergency spillway has a crest elevation of 8619.5 feet. The top of the embankment is at 8625.5 feet. The overtopping weir width includes the dike areas on the north side of the reservoir. All of these elevations are from the as-built drawings and have not been field verified. Also, the initial elevation of the reservoir was assumed to be 8605.5 feet, at a volume of 2562 acre-feet. This is the volume of the reservoir half full, to the crest of the emergency spillway.

Figure 13: Willow Park reservoir embankment.

Figure 14: Emergency Spillway, Willow Park Reservoir. Capacity ≈ 8100 cfs.

MODELING RESULTS

Four scenarios were simulated in this analysis: an 18-inch 6-hour PMP event, both without and with a simulated Cloud Peak Reservoir failure and a 30-inch 24-hour PMP event, both with and without a simulated Cloud Peak Reservoir failure. Hydrographs at the outlet are provided in Figure 15 and tabular results of each simulation are provided in Tables 3 through 6. The limitations of this modeling, discussed above, should be noted. Accordingly, these results need to be considered approximate.

Hydrologic modeling of the Willow Park watershed indicates that if a PMP event occurs, between 20,400 to 24,000 acre-feet of water will flow out of the watershed for the 6-hour event, with 95 percent of the volume exiting in 18 hours. For the 24-hour event, between 41,300 to 45,000 acre-feet of water will flow out of the watershed, with 97 percent of the volume exiting in 30 hours. These volumes are a great deal more than the combined empty storage capacity of the reservoirs, 11,200 acre-feet, measured to the tops of their embankments. In all of these analyses, both with and without the Cloud Peak failure, the Willow Park reservoir embankment will be substantially overtopped - the emergency spillway capacity of about 8100 cfs is insufficient.

The effect of the Cloud Peak failure on peak flow at Willow Park interestingly varies according to the length of the storm. For the 6- hour event, the Willow Park failure will be delayed by attenuation with the Cloud Peak reservoir for sufficient time so that the breach wave, when the reservoir does fail, arrives later than the rainfall peak (using the TR-60 rainfall distribution). The breach wave is not additive. However, for the 24-hour event the breach wave arrives early enough for a portion of the wave to be significantly additive to the rainfall-runoff peak. This phenomenon is illustrated in Figure 15.

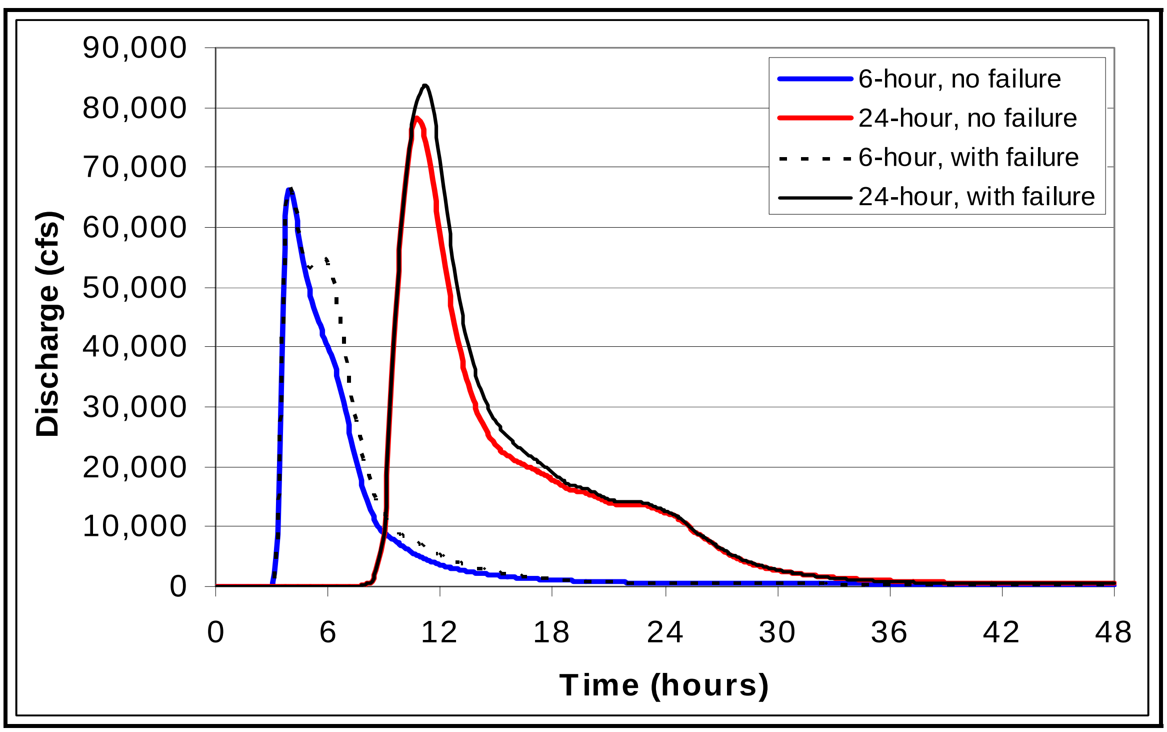

Figure 15: Hydrographs at the Willow Park Reservoir outflow, for the 6- and 24-hour storms, both with and without a Cloud Peak Reservoir failure.

Results from the 6-hour PMP analysis, without a Cloud Peak failure, are shown in Table 3. This analysis indicates that the peak flow at the outlet of Willow Park Reservoir will be 66,300 cfs, which represents a peak flow yield of 1960 cfs/mi^2. The emergency spillway for Willow Park has a conveyance capacity of about 8100 cfs, with a total conveyance of about 9000 cfs with the inclusion of the principal spillway – the spillways can only convey about **14 percent** of the PMP. At the 66,300 cfs peak flow, the embankment would be overtopped by a maximum of 5 feet. The duration of overtopping will be about 5 ½ hours. The peak flow exiting Cloud Peak reservoir will be 13,400 cfs. The emergency spillway for Cloud Peak has a conveyance capacity of about 1900 cfs. At the 13,400 cfs peak flow, the embankment would be overtopped by a maximum of about 3.8 feet. The duration of overtopping will also be about 5 ½ hours. Considering the material of the embankment and the lack of armor at the embankment crests (Figures 12 and 13), both embankments will most likely fail in the case of a 6-hour PMP in the Willow Park watershed, with the peak breach flow exiting the canyon just above the community of Story two hours after the initiation of the Willow Park failure (Yochum 2005). Most of Story would be inundated, threatening the loss of many lives and causing extensive property damage both within Story and downstream throughout the Piney Creek and Clear Creek river valleys (Yochum 2005).

Table 3: Hydrologic model results, 18 inch 6-hour PMP event without Cloud Peak Embankment failure.

Hydrologic Element	Peak Discharge (cfs)	Time of Peak	Total Volume (acre-feet)	Depth of Runoff (inches)	Contributing Area (square miles)
Subbasin-1	20,900	30 Sep 05 0245	2,870	15.37	3.50
Junction-4	18,700	30 Sep 05 0255	2,850	----	3.50
Subbasin-2	23,100	30 Sep 05 0250	3,450	14.63	4.42
Cloud Peak Res	13,400	30 Sep 05 0435	4,370	----	7.92
Subbasin-3	20,600	30 Sep 05 0300	3,500	15.08	4.38
Junction-6	19,900	30 Sep 05 0430	7,870	----	12.29
Subbasin-4	10,500	30 Sep 05 0250	1,550	12.10	2.40
Junction-9	24,200	30 Sep 05 0310	9,380	----	14.70
Subbasin-5	12,900	30 Sep 05 0305	2,280	13.88	3.08
Subbasin-6	5,260	30 Sep 05 0245	695	11.96	1.09
Junction-10	40,000	30 Sep 05 0310	12,400	----	18.87
Subbasin-7	12,200	30 Sep 05 0255	1,900	11.55	3.10
Junction-15	49,300	30 Sep 05 0315	14,300	----	21.96
Subbasin-8	18,200	30 Sep 05 0255	2,840	14.60	3.64
Junction-19	29,200	30 Sep 05 0255	4,700	----	6.52
Subbasin-9	4,540	30 Sep 05 0235	499	12.32	0.76
Subbasin-10	9,810	30 Sep 05 0245	1,370	12.12	2.11
Subbasin-11	2,900	30 Sep 05 0235	328	11.72	0.53
Junction-16	77,400	30 Sep 05 0305	19,300	----	29.01
Subbasin-13	19,500	30 Sep 05 0245	2,570	13.30	3.63
Junction-20	90,000	30 Sep 05 0305	21,900	----	32.63
Subbasin-12	6,900	30 Sep 05 0240	875	13.71	1.20
Willow Park Res	66,300	30 Sep 05 0400	20,400	----	33.83

Results from the 24-hour PMP analysis, without a Cloud Peak failure, are shown in Table 4. This analysis indicates that the peak flow at the outlet of Willow Park Reservoir will be 78,200 cfs, which represents a peak flow yield of 2310 cfs/mi^2. The emergency spillway for Willow Park has a conveyance capacity of about 8100 cfs, with a total conveyance of about 9000 cfs with the inclusion of the principal spillway – the spillways can only convey about **12 percent** of the PMP. At the 78,200 cfs peak flow, the embankment would be overtopped by a maximum of about 5.7 feet. The duration of overtopping will be about 18 ½ hours. The peak flow exiting Cloud Peak reservoir will be 20,900 cfs. The emergency spillway for Cloud Peak has a conveyance capacity of about 1900 cfs. At the 20,900 cfs peak flow, the embankment would be overtopped by a maximum of about 5.3 feet. The duration of overtopping will also be about 18 ½ hours. Considering the material of the embankment and the lack of armor at the embankment crests (Figures 12 and 13), both embankments will most likely fail in the case of a 24-hour PMP in the Willow Park watershed, with the peak breach flow exiting the canyon just above the community of Story two hours after the initiation of the Willow Park failure (Yochum 2005). Most of Story would be inundated, threatening the loss of many lives and causing extensive property damage both within Story and downstream throughout the Piney Creek and Clear Creek river valleys (Yochum 2005).

Table 4: Hydrologic model results, 30 inch 24-hour PMP event without Cloud Peak Embankment failure.

Hydrologic Element	Peak Discharge (cfs)	Time of Peak	Total Volume (acre-feet)	Depth of Runoff (inches)	Contributing Area (square miles)
Subbasin-1	12,400	30 Sep 05 1000	5,080	27.26	3.50
Junction-4	12,100	30 Sep 05 1010	5,070	----	3.50
Subbasin-2	15,100	30 Sep 05 1005	6,230	26.44	4.42
Cloud Peak Res	20,900	30 Sep 05 1040	9,360	----	7.92
Subbasin-3	14,600	30 Sep 05 1010	6,290	26.94	4.38
Junction-6	32,600	30 Sep 05 1035	15,600	----	12.29
Subbasin-4	7,630	30 Sep 05 1005	30,100	23.49	2.40
Junction-9	34,800	30 Sep 05 1045	18,600	----	14.70
Subbasin-5	9,840	30 Sep 05 1015	4,210	25.60	3.08
Subbasin-6	3,520	30 Sep 05 1000	1,350	23.33	1.09
Junction-10	46,000	30 Sep 05 1035	24,200	----	18.87
Subbasin-7	9,490	30 Sep 05 1010	3,770	22.82	3.10
Junction-15	52,600	30 Sep 05 1035	28,000	----	21.96
Subbasin-8	12,300	30 Sep 05 1005	5,130	26.41	3.64
Junction-19	21,400	30 Sep 05 1005	8,750	----	6.52
Subbasin-9	2,550	30 Sep 05 1000	963	23.76	0.76
Subbasin-10	6,820	30 Sep 05 1005	2,650	23.52	2.11
Subbasin-11	1,720	30 Sep 05 1000	645	23.04	0.53
Junction-16	71,900	30 Sep 05 1025	37,300	----	29.01
Subbasin-13	12,300	30 Sep 05 1000	4,820	24.93	3.63
Junction-20	81,300	30 Sep 05 1020	42,200	----	32.63
Subbasin-12	4,120	30 Sep 05 1000	1,620	25.40	1.20
Willow Park Res	78,200	30 Sep 05 1045	41,300	----	33.83

Results from the 6-hour PMP analysis, with a Cloud Peak failure, are shown in Table 5. This analysis indicates that the peak flow at the outlet of Willow Park Reservoir will be 66,500 cfs, which represents a peak flow yield of 1970 cfs/mi^2. The emergency spillway for Willow Park has a conveyance capacity of about 8100 cfs, with a total conveyance of about 9000 cfs with the inclusion of the principal spillway – the spillways can only convey about **14 percent** of the PMP. At the 66,500 cfs peak flow, the embankment would be overtopped by a maximum of 5 feet. The duration of overtopping will be about 6 ½ hours. The peak flow exiting Cloud Peak reservoir with the failure will be roughly 37,600 cfs. The Cloud Peak breach wave will arrive at Willow Park reservoir lagged behind the rainfall peak, so the peak flow at Willow Park will not significantly change. However, the Willow Park reservoir embankment will most-likely fail in the case of a 6-hour PMP, with the peak breach flow exiting the canyon just above the community of Story two hours after the initiation of the Willow Park failure (Yochum 2005). Most of Story would be inundated, threatening the loss of many lives and causing extensive property damage both within Story and downstream throughout the Piney Creek and Clear Creek river valleys (Yochum 2005).

Table 5: Hydrologic model results, 18 inch 6-hour PMP event with Cloud Peak Embankment failure.

Hydrologic Element	Peak Discharge (cfs)	Time of Peak	Total Volume (acre-feet)	Depth of Runoff (inches)	Contributing Area (square miles)
Subbasin-1	20,900	30 Sep 05 0245	2,870	15.37	3.50
Junction-4	18,700	30 Sep 05 0255	2,850	----	3.50
Subbasin-2	23,100	30 Sep 05 0250	3,450	14.63	4.42
Cloud Peak Res	37,600	30 Sep 05 0455	7,900	----	7.92
Subbasin-3	20,600	30 Sep 05 0300	3,520	15.08	4.38
Junction-6	40,500	30 Sep 05 0455	11,400	----	12.29
Subbasin-4	10,500	30 Sep 05 0250	1,550	12.10	2.40
Junction-9	38,800	30 Sep 05 0505	12,900	----	14.70
Subbasin-5	12,900	30 Sep 05 0305	2,280	13.88	3.08
Subbasin-6	5,260	30 Sep 05 0245	695	11.96	1.09
Junction-10	43,700	30 Sep 05 0505	15,900	----	18.87
Subbasin-7	12,200	30 Sep 05 0255	1,910	11.55	3.10
Junction-15	49,400	30 Sep 05 0310	17,800	----	21.96
Subbasin-8	18,200	30 Sep 05 0255	2,840	14.60	3.64
Junction-19	29,200	30 Sep 05 0255	4,700	----	6.52
Subbasin-9	4,540	30 Sep 05 0235	499	12.32	0.76
Subbasin-10	9,810	30 Sep 05 0245	1,370	12.12	2.11
Subbasin-11	2,900	30 Sep 05 0235	328	11.72	0.53
Junction-16	77,500	30 Sep 05 0305	22,800	----	29.01
Subbasin-13	19,500	30 Sep 05 0245	2,570	13.30	3.63
Junction-20	90,100	30 Sep 05 0305	25,400	----	32.63
Subbasin-12	6,900	30 Sep 05 0240	875	13.71	1.20
Willow Park Res	66,500	30 Sep 05 0400	24,000	----	33.83

Results from the 24-hour PMP analysis, with a Cloud Peak failure, are shown in Table 6. This analysis indicates that the peak flow at the outlet of Willow Park Reservoir will be 83,800 cfs, which represents a peak flow yield of 2480 cfs/mi^2. The emergency spillway for Willow Park has a conveyance capacity of about 8100 cfs, with a total conveyance of about 9000 cfs with the inclusion of the principal spillway – the spillways can only convey about **11 percent** of the PMP. At the 83,800 cfs peak flow, the embankment would be overtopped by a maximum of 6 feet. The duration of overtopping will be about 18 ½ hours. The peak flow exiting Cloud Peak reservoir with the failure will be roughly 43,100 cfs. The Cloud Peak breach wave will arrive at Willow Park reservoir slightly lagged behind the rainfall peak, but peak flow at Willow Park will add 5600 cfs to the rainfall peak. The Willow Park reservoir embankment will most-likely fail in the case of a 24-hour PMP, with the peak breach flow exiting the canyon just above the community of Story two hours after the initiation of the Willow Park failure (Yochum 2005). Most of Story would be inundated, threatening the loss of many lives and causing extensive property damage both within Story and downstream throughout the Piney Creek and Clear Creek river valleys (Yochum 2005).

Table 6: Hydrologic model results, 30 inch 24-hour PMP event with Cloud Peak Embankment failure.

Hydrologic Element	Peak Discharge (cfs)	Time of Peak	Total Volume (acre-feet)	Depth of Runoff (inches)	Contributing Area (square miles)
Subbasin-1	12,400	30 Sep 05 1000	5,080	27.26	3.50
Junction-4	12,100	30 Sep 05 1010	5,070	----	3.50
Subbasin-2	15,100	30 Sep 05 1005	6,230	26.44	4.42
Cloud Peak Res	43,100	30 Sep 05 1050	13,000	----	7.92
Subbasin-3	14,600	30 Sep 05 1010	6,290	26.94	4.38
Junction-6	51,800	30 Sep 05 1050	19,200	----	12.29
Subbasin-4	7,630	30 Sep 05 1005	3,010	23.49	2.40
Junction-9	50,900	30 Sep 05 1055	22,200	----	14.70
Subbasin-5	9,840	30 Sep 05 1015	4,210	25.60	3.08
Subbasin-6	3,520	30 Sep 05 1000	1,350	23.33	1.09
Junction-10	60,400	30 Sep 05 1055	27,800	----	18.87
Subbasin-7	9,490	30 Sep 05 1010	3,770	22.82	3.10
Junction-15	65,000	30 Sep 05 1100	31,500	----	21.96
Subbasin-8	12,300	30 Sep 05 1005	5,130	26.41	3.64
Junction-19	21,400	30 Sep 05 1005	8,750	----	6.52
Subbasin-9	2,550	30 Sep 05 1000	963	23.76	0.76
Subbasin-10	6,820	30 Sep 05 1005	2,650	23.52	2.11
Subbasin-11	1,720	30 Sep 05 1000	645	23.04	0.53
Junction-16	78,200	30 Sep 05 1100	40,900	----	29.01
Subbasin-13	12,300	30 Sep 05 1000	4,820	24.93	3.63
Junction-20	83,800	30 Sep 05 1055	45,700	----	32.63
Subbasin-12	4,120	30 Sep 05 1000	1,620	25.40	1.20
Willow Park Res	83,800	30 Sep 05 1110	45,000	----	33.83

At first glance, the extreme runoff and low conveyance capacity of the existing emergency spillway seems unreasonable. Comparing this simulated runoff with the runoff response from actual extreme events used in the PMP computations can help judge the reasonableness of these predictions.

On May 30 and 31, 1935 a series of convective storms (Cherry Creek storm) broke out in Colorado east of Colorado Springs between the Front Range and the Kansas border. These storms were small in aerial extent but extreme in intensity. Within the Kiowa Creek watershed, a non-mountainous watershed flowing off of elevated forest and lower range land, an extreme localized cell dropped up to 24 inches of rain in 6-hours (Hansen et. al. 1988) within or adjacent to the Kiowa Creek watershed. The resulting flood had a peak flow of 43,500 cfs on 5/30/1935 at USGS streamgage *Kiowa Creek at Elbert* (ID 06758000, elevation 6740 feet), a 28.6 square mile watershed. This flow represents a peak flow yield of 1520 cfs/mi^2. For reference, the 6-hour 10 square mile PMP for this location is 26 inches.

From June 13 through 20, 1965, heavy convective rainstorms (Plum Creek storm) occurred in the same vicinity as the Cherry Creek storm. During the most intense period, on June 16 and 17, up to 18.1 inches of rain fell within a 24-hour period, with rainfall depths over 5 inches common (Hansen et. al. 1988). Up to 14 inches of precipitation fell just south of the Kiowa Creek watershed. The 28.6 square mile Kiowa Creek at Elbert gage recorded a peak flow of 41,500 cfs from this event. This flow represents a peak flow yield of 1450 cfs/mi^2. It is quite interesting that two such large rainfall-runoff events occurred (and were recorded) in the same watershed.

Numerous other extreme precipitation events in Colorado and Wyoming, events used in the computation of PMP estimates for these areas, have occurred between the Continental Divide and the 103rd meridian. However such extreme events, though they occur regularly, occur infrequently over a streamgaged watershed of appropriate size to measure the flood response and occur very infrequently over any particular watershed.

Kiowa Creek is in a different hydrologic area than the Willow Park watershed. The publication of HMR-55A had the specific purpose of extrapolating actual measured storms, such as the Cherry Creek, Plum Creek and numerous others, throughout the area of analysis, from the Continental divide to the 103rd meridian. These are the PMP numbers used in this hydrologic analysis. The important point to draw from the Kiowa Creek runoff events is the extreme floods that occurred in response to actual extreme rain events. This indicates that, if an extreme rainfall event occurs on the much more mountainous Willow Park watershed, the very high peak flows predicted in this hydrologic analysis are reasonable.

Finally, it is interesting and useful to assign a return period of a storm that would be at the capacity of the Willow Park and Cloud Peak reservoirs emergency spillways. A number of potential events were modeled in a trial-and-error approach for finding the precipitation frequency of an overflow event. The initial elevation of the reservoirs proved to be a sensitive assumption to the results of this analysis, hence two initial conditions were used, specifically: (1) the same elevations used for the PMP analysis, that is, an elevation of 9715 feet for Cloud Peak and 8605.5 feet for Willow Park, reflecting both reservoirs half-filled (by volume) to the crest of their emergency spillways; and (2) at the crest of Cloud Peak's emergency spillway (9726 feet) and at the crest of Willow Park's principal spillway (8619.5 feet). NOAA Atlas 2, Volume II (Miller et. al. 1973) for 24-hour storms was used in the analysis. The rainfall distribution for the Willow Park watershed is provided in Table 7. NOAA Atlas 2 only provides a rainfall distribution for the 2 through 100-year events – these values were plotted on probability paper and a line through these points was used to extrapolate to the 200-, 500- and 1000-year events. Results of the hydrologic analyses are provided in Table 8.

Table 7: 24-hour rainfall frequencies for the Willow Park reservoir watershed, from NOAA Atlas 2.

Storm Frequency (years)	Rain Amount (inches)
2	2.6
5	3.2
10	3.6
25	4.2
50	5.0
100	5.5
200	*6.5
500	*7.7
1000	*8.8

*extrapolated estimate

Table 8: Spillway capacity hydrologic analysis results.

Precipitation	High or Low	24-hour	Cloud Peak		Willow Park	
	Initial Stage	Frequency	Max. Stage	Peak Outflow	Max. Stage	Peak Outflow
(inches)		(year)	(feet)	(cfs)	(feet)	(cfs)
5.50	low	100	9723.2	0	8620.0	750
5.50	high	100	9729.2	750	8622.0	2850
*6.5	low	200	9725.4	0	8621.0	1660
*6.5	high	200	9730.0	1050	8623.0	4410
*7.7	low	500	9727.5	230	8621.9	2700
*7.7	high	500	9731.0	1470	8624.3	6650
*8.8	low	1000	9728.7	570	8622.7	3940
*8.8	high	1000	9731.9	1890	8025.5	8860

*extrapolated estimate

With an initial water surface at the crest of the emergency spillway, this analysis indicates that Cloud Peak reservoir's spillway can pass about an 8.8 inch **1000-year** rain event. With an initial water surface at the crest of the principal spillway, Willow Park reservoir can also pass approximately the 8.8 inch **1000-year** event. At about the 1000-year event the spillway will be at capacity with overtopping and likely failure being imminent.

The Willow Park and Cloud Peak reservoirs are nearing the end of their initial 50-years of design life. Using design life and the spillway capacity return period of 1000-years, the risk of overtopping can be computed. With a 50-year design life the statistical **risk of an overtopping event was 5 percent**. This is the risk of the structure to have already been overtopped. If rehabilitation on the structures is performed, an additional 50 years can be added to their design lives. With a rehabilitated 100-year design life the statistical **risk of an overtopping event is 10 percent** over the next additional 50 years.

CONCLUSIONS

This hydrologic analysis was performed using the NRCS curve number technique to simulate the runoff. This method has limitations that impact its use in forested watersheds such as Willow Park. These limitations were discussed in the body of this report. An alternate procedure, such as paleoflood hydrology, may be warranted in this situation. However, limitations with other methods also exist and would need to be acknowledged if additional analysis is performed.

The probable maximum precipitation (PMP) values were computed for the 33.8 square mile watershed and were found to be 18 inches for the 6-hour storm and 30 inches for the 24-hour storm.

Four hydrologic analyses were performed to assess the capability of the existing Willow Park principal and emergency spillways in passing the Probable Maximum Flood response to the PMP event. It is common practice through the United States for a high hazard dam, such as Willow Park, to be able to pass the PMF safely. The analyses included simulations for both the 6- and 24-hour storms, with and without a simulated Cloud Peak reservoir embankment failure. The results of the simulations are provided in Figure 15 and Tables 3 through 6. The analyses indicate that the emergency spillway for Cloud Peak reservoir is significantly undersized – it can only pass 14 to 11 percent of the PMF. **If a PMP occurs, both the Cloud Peak and Willow Park reservoir embankments will be overtopped and most likely fail.**

Additional hydrologic analysis indicated that the existing spillways for both Cloud Peak and Willow Park Reservoir, as shown on as-built drawings, can pass a 24-hour rainfall event of 8.8 inches, which is about a 1000-year return-period rain event. At first glance, such conveyance capacity appears to be substantial but **this frequency over a 100-year rehabilitated design life indicates an overtopping risk of 10 percent**. A 10-percent chance of overtopping and failure of the high-hazard Willow Park Reservoir embankment is significant. This probability of a breach, with the resultant floodwave likely causing substantial loss of life and extensive property damage in the community of Story and throughout the Piney Creek and Clear Creek stream valleys, is a significant threat that, in the least, strongly argues for the installation of an automated emergency alert system.

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