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PMF HYDROLOGY, WITH RODEO-CHEDISKI FIRE IMPACTS, AND SPILLWAY HYDRAULICS FOR BLACK CANYON LAKE AND DAM

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Abstract

A significant portion of the 5.6 mi^2 watershed that is tributary to the Arizona Game and Fish Department's Black Canyon reservoir near Heber, Arizona, was variably burned (no impact to severe) by the Rodeo-Chediski fire, which involved nearly 460,000 acres above and below Arizona's Mogollon Rim during the summer of 2002. The reservoir and spillway at Black Canyon were originally designed to accommodate a 100-*yr* inflow design flood. With impacts on the downstream community of Heber in mind, the Arizona Department of Water Resources has classified the dam as high hazard, and therefore the reservoir/spillway system must safely pass the applicable hydrologic design event, in this case the full PMF. We report on PMP development and on PMF hydrology simulations using both simple (single basin) and complex (multi-basin) HMS watershed models that take into account the degree of burn and probable recovery over the 5-year period likely to elapse between the present time and the construction of improvements. Additionally, we provide results of a hydraulic evaluation of the existing spillway.

Introduction

Communities downstream of earthen dams are at increased risk of disastrous flooding. For some years, legislatures and regulators have focused on worst-case scenarios. Though such scenarios have an exceedingly low probability of occurring, all involved are unwilling to suffer the consequences. In addition to flood hazards posed by dams, many areas experience infrequent high-magnitude storm events with the potential to cause severe flooding. 'Flashy' ephemeral watercourses in arid and semi-arid regions convey storm flows that rapidly peak following intense precipitation of relatively brief duration. This is counter-intuitive to many unfamiliar with hydrologic processes. Finally, the effects of fire on watershed condition can magnify the 'flashiness' of these systems and greatly increase peak flows – particularly for low-recurrence-interval events. Flood hazard management for a burned watershed with both a dam and flash-flood potential demands a strategy that considers: watershed and stream channel conditions; and performance and reliability of engineered structures.

The Black Canyon reservoir, earthen dam and tributary watershed (5.6 mi^2), are located in remote semi-arid ponderosa pine country above the Mogollon Rim in north central Arizona. The dam is nearly 80 *ft* tall, was built in the early 1960's and has a capacity of 1,580 *ac-ft*. See Figures 1 & 2. The Arizona Game & Fish Department (AGFD) owns and operates the facility, located on U.S. Forest Service land, primarily for recreation. A failure of this dam could catastrophically flood the downstream community of Heber, Arizona, (population of nearly 2,500), which, like many other communities, has grown or expanded in a flood plain.

Figure 1. View of existing Black Canyon dam and spillway.

Figure 2. Upstream view of Black Canyon dam spillway approach channel.

In the summer of 2002, the Rodeo-Chediski fire burned nearly 460,000 acres in the area. Close to 24% of the Black Canyon Lake (BCL) watershed was severely burned; 13% was moderately burned; 47% was slightly burned; and, 16% went untouched by fire. See Figures 3 and 4. Note that northern Arizona ponderosa pine forests vary from never-burned 'dog-hair' thickets of small diameter trees (up to 5000 stems per hectare) to much more sparsely populated mature 'old-growth' stands (several hundred stems per hectare) that have experienced fire in the past, or have been better managed.

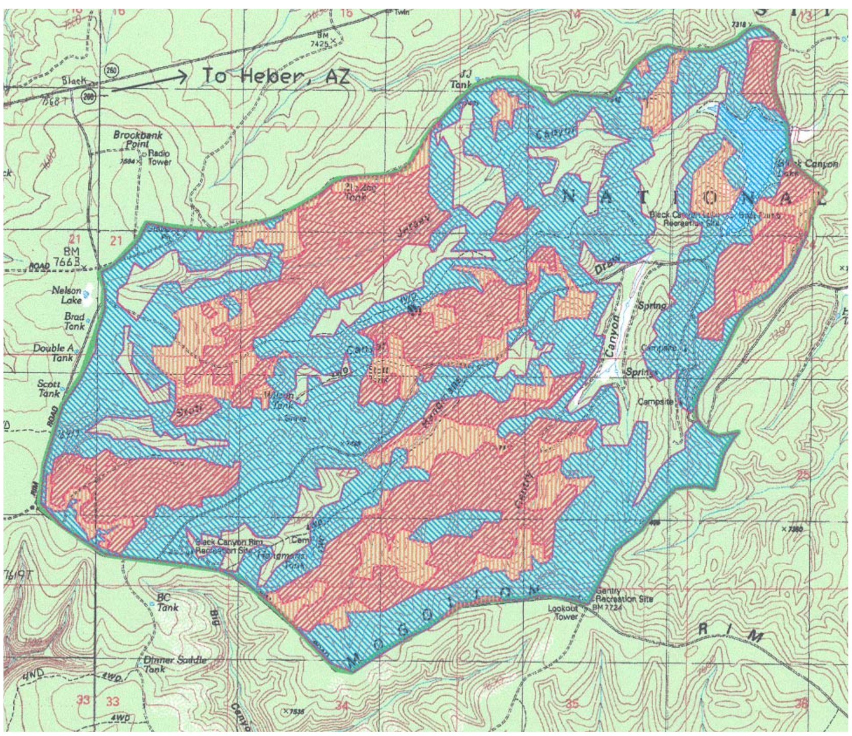

Figure 3. BCL watershed (delineated) burn severity (after BAER Team, 2002b) draped over USGS 7.5-minute topographic map. North is up. NE-SW hatch is severely burned, vertical hatch is moderately burned, NW-SE hatch is slightly burned; no hatch indicates unburned areas.

Figure 4. Recovery of under-story vegetation in moderately to severely-burned areas. October of 2003.

BCL dam, due to its height and capacity, is regulated by the Arizona State Department of Water Resources (ADWR). In 2001, ADWR (2001) reported BCL spillway adequacy concerns. Rodeo-Chediski fire impacts added urgency to the need for adequacy and safety review. A study performed soon after the fire (USACE, 2002a) validated ADWR's (2002a) 'high' hazard classification. The USACE report stated that, following the Rodeo-Chediski fire, there would be a 53% increase in peak discharge to BCL for a 100-*yr* storm event. In the event of an extreme dam failure due to spillway inadequacy and dam overtopping, a resulting peak discharge of 50,000 *cfs* at the dam would yield an attenuated peak flow of 33,000 *cfs* at Heber within 1.25 *hr*, resulting in flood damage and possible fatalities due to lack of time for evacuation (USACE, 2002a).

In addition, ADWR (2002b) raised concerns about probable maximum flood (PMF) capacity, and subsequently required that the BCL spillway be able to pass 100% of the PMF without overtopping the dam. However, the existing spillway was designed to pass only the peak discharge resulting from a 100-*yr* 6-*hr* storm event (Earle V. Miller Engineers, 1963). To address these issues, AGFD requested an evaluation of:

- Probable Maximum Precipitation (PMP);
- The reservoir inflow design flood (IDF) for the 100-*yr* and PMP events, considering each of the existing burned (2003) and predicted recovered (5-year outlook – 2008) conditions.
- Existing spillway capacity and assessment of improvement alternatives.

PMP Development

Helton (2003) estimated the PMP for the BCL watershed using methods provided in Hydro-Meteorological Report (HMR) 49 (Hansen et al., 1984). HMR 49 identifies two types of storms that result in a maximum amount of precipitation. The general storm has numerous extreme rainfalls, which cover small areas but are all contained within a general rainfall that covers a wide span of area over an extended period of time (24 to 72 *hr*). Local storms are rainfalls covering small areas in shorter time periods (e.g., 6 *hr*) and are most notable in intermountain regions (Hansen et al., 1984). The PMP was estimated for the 6-*hr* local storm as well as the 72-*hr* general storm for August, which was selected since the ending months of summer typically produce the greatest depth of orographic precipitation (Hansen et al., 1984).

The local storm PMP is 10.43 *in*. The corresponding time distribution of precipitation, or hyetograph (Figure 5) was developed following the procedure provided by HMR 5 (U.S. Weather Bureau, 1947). Following HMR 36 (U.S. Weather Bureau, 1961) the general storm PMP is 22.0 *in*.

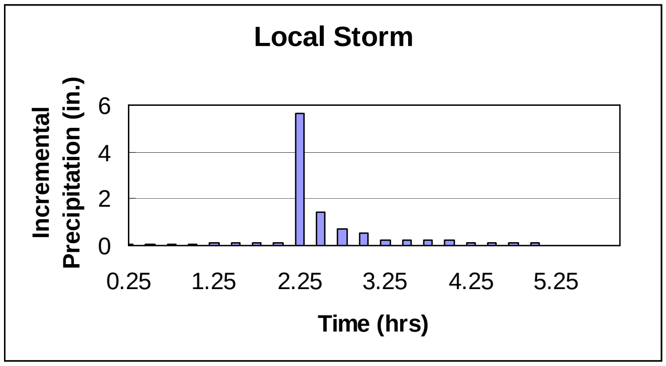

Figure 5. Time distribution of precipitation for the local storm 6-hour PMP.

There are two primary factors that make the local storm most relevant to the BCL watershed. While the total rainfall of the general storm is greater than the local storm, the local storm has much higher rainfall intensity and occurs over a small area. Large watersheds and/or large reservoirs will attenuate the effect of the high-intensity short-duration rainfall 'spike' characteristic of the local storm. The contributing watershed for BCL is small enough that the high local storm rainfall intensities are effective throughout most of the watershed. The reservoir capacity, at 1,580 ac-*ft*, is relatively small and will allow PMF to proceed through the reservoir without significant attenuation. In this case, the local storm yields the PMP of greatest interest for a PMF evaluation.

A comparison may be made between our PMP result and that for the nearby Blue Ridge Dam site (71.1 mi^2 watershed; storage of nearly 20,000 ac-*ft*). For Blue Ridge, an average 6-*hr*. PMP of 8.1 *in* was determined by URS Greiner Woodward Clyde (1999). This is considerably less than the 10.43 *in* for the local storm PMP at BCL. However, since the watershed area tributary to Blue Ridge is much larger than BCL (approx. 5.6 mi^2), the areal reductions applied as part of the local storm PMP procedure are considerably greater than for BCL. Thus, the Blue Ridge local storm PMP is understandably smaller.

PMF Model Development and Simulation Results

Here we present a synopsis of the PMF development and results. Please refer to Helton (2004) for a full presentation of the PMF evaluation.

Approach and Parameters

Flood record data are unavailable for the ungaged BCL watershed. Therefore, our approach in estimating the PMF was to create a hydrologic model for which 100-*yr* and PMF simulation results were validated using simulation results obtained by other means, or from other studies. We simulated the watershed's hydrological response for these two events by considering the watershed:

- as both a single catchment and multiple-catchment system;
- both in its existing condition (2003, or 1 year post burn), and, after 'hydrologic' recovery –forecast to be 5 years out (2008, or 6 years post burn).

A total of 8 hydrologic models were thus developed. Subdividing the watershed, in this case, into 19 catchments, was explored because of the potential for underestimation of floods by using a single-catchment spatial average of watershed condition, e.g. the Curve Number (CN) and hydraulic parameters. Predicting the watershed's future response is also important, as a partially-recovered watershed condition will likely exist when spillway improvements are implemented.

The models were built using U.S. Army Corps of Engineers HMS software (USACE, 2001). The simulations utilized the NRCS/SCS unit hydrograph method for developing flows from rainfall distributions. That method entails calculating runoff excess and then transforming it into a flood hydrograph using a dimensionless unit hydrograph (USACE, 1999). Watershed parameters of interest included: catchment area(s), CNs, Time of Concentration (T_C), and, channel geometry and roughness.

The local storm PMP rainfall amount, originally developed with a 15-*min* increment, was modified to yield a time series with a 3-*min* increment, necessary for convergence of the 19-sub-catchment simulations (Helton, 2003).

A 100-*yr* 6-*hr* rainfall amount of 4.5 *in* was determined using NOAA Atlas 2, (Miller et al., 1973). This amount was used by Earle V. Miller Engineers (1963) for the then-proposed dam. NOAA Atlas 14 (hdsc.nws.noaa.gov/hdsc/pfds/sa/az_pfds.html) provides more up-to-date information than NOAA Atlas 2. The NOAA atlas 14 100-*yr* 6-*hr* storm precipitation value of 4.2 in was not used, because a key application of the 100-*yr* 6-*hr* simulation was model validation using the original Earl V. Miller Engineers (1963) report; thus, the NOAA Atlas 2 amount of 4.5 *in* was preferred for the purposes of that comparison. A rainfall distribution for the 6-*hr* storm event was developed following Technical Release (TR) 60 (NRCS, 1985a).

Though the NRCS (1985b) provides four 24-*hr* unit rainfall hyetographs, we chose to proceed with the standard 6-*hr* storm hyetograph because peak flows developed using this hyetograph have a better match to predictions developed using USGS regression equations.

The runoff characteristics of the watershed depend on topography, soil, vegetation, and the antecedent moisture condition (AMC). Our models are based upon convective type storms, which generally occur during the mid- to late summer monsoon season (Hansen et al., 1984). We assumed that the length of time between storm events as well as the brevity of the storm events will allow the soils to move out of the saturated condition between storm events. For this reason, an AMC of II was adopted.

Modeling the BCL watershed as one catchment involves averaging watershed attributes that can, and do, vary significantly. Subdividing the watershed into 19 catchments was an attempt to explore model sensitivity to watershed spatial discretization. The 19-catchment delineation is somewhat subjective because burn severity (Figure 3) varied complexly. The 19-catchment model required more mathematical routines, watershed information, and assumptions, which potentially provide additional errors. Therefore it was important to verify that the routing calculations within our 19-catchment models converged. This was done by addressing HMS convergence warnings and re-evaluating problem reaches until no warnings were received.

A Terrestrial Ecosystem Survey (TES) with soil types was available for the Apache-Sitgreaves National Forest (Laing et al., 1987). Also, based upon an identification performed by an Apache-Sitgreaves soil scientist, the Burned Area Emergency Response (BAER) Team report (BAER 2002a; 2002b) classified the watershed soil as type C (NRCS classification scheme – NRCS, 1986). Based on our field observations and soil sampling we concurred with and adopted this classification.

Before the watershed burned the vegetation, per the TES (Laing et al., 1987), was fairly uniform, and the stand of woody plants there consisted primarily of Ponderosa pine (Pinus ponderosa scopulorum), with some White fir (Ables concolor) and Douglas fir (Pseudotsuga menziesii); the ground cover consisted mainly of grass and variable pine needle litter thickness. The Rodeo-Chediski fire dramatically impacted watershed vegetation, litter, and soil. Severely-burned areas consisted only of dead Ponderosa pine stands and scorched earth. By October, 2003, moderately-burned stands (Figure 4) had some grass beginning to be replenished among the few live Ponderosa pines. At the same time, slightly burned areas appeared to have substantially recuperated, with abundant new grass growth.

Curve Numbers (CNs) determined by the BAER team (C. Lovely, personal communication, June, 2003) and by ourselves for the four burned condition categories are provided below:

	Existing Condition	Predicted Future Condition	BAER Team
Severely Burned	90	74	90
Moderately Burned	82	72	85
Slightly Burned	76	65	80
Unburned	65	65	75

The BAER team selected CNs for their hydrological analysis mostly through experience. Ideally, the CNs would be back-calculated through calibration from observed events. We are unaware of any published back-calculated CNs for post-fire forested watersheds. CNs for the various burned conditions of the BCL watershed were developed from existing information (NRCS, 1986). CNs for the unburned condition were chosen from the Oak Creek study (ADWR, 1990), whose authors attempted to develop calibrated CNs for various watersheds, including unburned Ponderosa pine forest systems.

Weighted CNs for the present and predicted future conditions were determined for each catchment of the 19-catchment and 1-catchment models. The lake was given a CN of 99; this is the largest value allowable by HMS and it is assumed that the lake bottom is fully saturated and therefore impermeable.

Channel geometry and roughness are required to calculate time of concentration (T_C), and to estimate hydrograph attenuation due to stream reach routing. T_C is the time it takes for water to travel from the hydraulically most distant point on a watershed, in this case, the Mogollon Rim, to the reservoir. For models that treated the watershed as one catchment, routing was not required. The USACE (1999) recommends either of the Muskingum-Cunge or Kinematic Wave routing methods for ungaged stream channels. The former was used, as it allows for more detailed channel geometries, rather than generic channel shapes, and permits changes in lateral channel roughness. We surveyed seven stream channel cross-sections, believed to represent most of the channel geometries found within the watershed, and data for each cross-section were prepared in the required HMS format. Slope and reach lengths were estimated from USGS 7.5' topographic maps.

Manning's roughness coefficient, *n*, was required for stream routing. We applied the Cowan (1956) method, as modified by Aldridge et al. (1973), for larger rock material, to estimate *n*. The *n* values selected were utilized in two segments of the HMS models:

- ❑ Reach routing;
- ❑ Time of Concentration estimation.

T_C was calculated using TR 55 procedures (NRCS, 1986). T_C is required in HMS to apply the NRCS dimensionless unit hydrograph methodology to calculate the unit hydrograph peak. An initial estimate of flow is required to get the channel velocities and subsequently the open channel travel time component of T_C. An iterative procedure was used for all 8 models to obtain convergence between the initial channel flow estimates and that provided by HMS.

Simulation Results

Peak flows obtained from HMS simulations are tabulated below. The full PMF hydrograph for the 19-catchment predicted future condition appears in Figure 6. The forecast peak PMF flow of 22,300 *cfs* is considerably less than the 27,100 *cfs* predicted for the existing multi-catchment watershed condition 1 year post-fire, but of similar magnitude.

	1 catchment				19 catchments	
Storm Event	Existing Condition Peak Flow (*cfs*)	Predicted Future Condition Peak Flow (*cfs*)	Existing Condition Excess Precipitation (*in*)	Predicted Future Condition Excess Precipitation (*in*)	Existing Condition Peak Flow (*cfs*)	Predicted Future Condition Peak Flow (*cfs*)
100-*yr* 6-*hr*	4,058	2,064	2.34	1.47	4,709	2,652
PMP	24,507	18,664	7.68	6.33	27,119	22,300

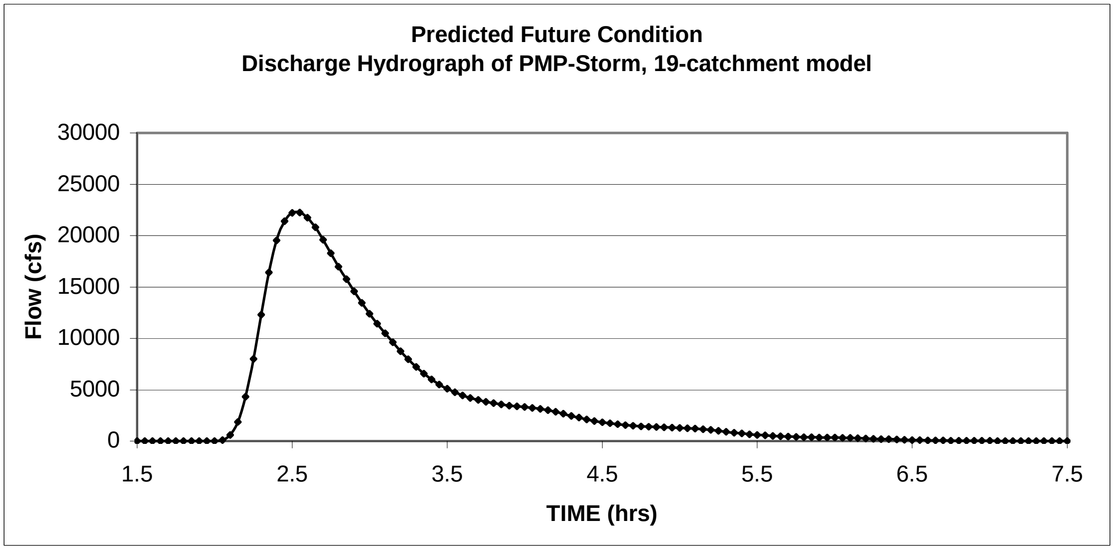

Figure 6. PMF hydrograph for the watershed condition 6-years (2008) post fire for the 19-catchment model.

Model Validation

A numerical hydrology model, if it cannot be calibrated using known precipitation-flow response for the watershed under consideration, or for a nearby similar shed, should at least be validated, for example, by comparing results with predictions from USGS regional regression equations, earlier hydrology analyses conducted as part of other investigations, or with simulation results, preferably calibrated, for another watershed with similar land use, vegetative cover, soils, and storm meteorology/climate.

Though the BCL watershed is ungaged, a summer monsoon flood did occur on September 6, 2002, soon after the Rodeo-Chediski Fire. The Arizona Game and Fish Department surveyed a high water mark, at the location of a road crossing, upstream of BCL. Given the water stage, channel cross-section, channel slope, and some limited rain gage data, calibration for the low-flow post-burn condition was possible—at least in principle. The available information was not used for calibration because:

- watershed condition improved substantially between the time of the monsoon flood (Sept. 6, 2002), shortly after the fire, and the time (fall 2003) considered by our simulation of the 'existing condition'; and,
- the September 6, 2002 storm recurrence interval is less than 2-*yr*, whereas the storms of interest are 100-*yr* and PMP events.

A PMF study (George V. Sabol Consulting Engineers, 1996) was completed as part of the evaluation of Lynx Lake dam near Prescott, Arizona. The BCL and Lynx Lake watersheds have similar vegetation, soil, and elevation. The Lynx Lake shed has an area of approximately 18.5 mi^2, nearly 3 times larger than the watershed area tributary to BCL. Therefore, comparisons presented here use peak discharge normalized by watershed area (cfs/mi^2). Lynx Lake had a local storm PMP value of 11.8 *in*, developed using HMR 49 methodology, and a 100-*yr* 24-*hr* storm precipitation of 4.92 *in*. As discussed above, BCL had a PMP value of 10.43 in and 100-*yr* 6-*hr* precipitation of 4.5 *in*. The Lynx Lake 100-*yr* 24-*hr* storm event models considered both the Green and Ampt (G&A) method and a prescribed initial loss combined with a uniform loss rate (IL+ULR) method. The normalized peak flow comparison is below.

Normalized Peak Flows (*cfs*/mi^2) for BCL and Lynx Lake

	BCL				Lynx Lake			
	1 catchment		19 catchments		1 catchment		10 catchments	
Storm Event	Existing Condition	Predicted Future Condition	Existing Condition	Predicted Future Condition	IL+ULR Method	G&A Method	IL+ULR Method	G&A Method
100-*yr* 24-*hr*	n/a	n/a	n/a	n/a	570	440	630	430
100-*yr* 6-*hr*	720	366	835	470	n/a	n/a	n/a	n/a
PMP	4,345	3,309	4,808	3,776	3,000	n/a	3,600	n/a

The Lynx Lake watershed is not in a burned condition, so our predicted future condition (2008) for BCL should be the basis for comparison. With this in mind, the PMF's are very similar. The PMF simulation (19-catchment model) for BCL resulted in a normalized peak flow of 3,776 *cfs*/mi^2 while the Lynx Lake PMF simulation (10-catchment model) resulted in 3,600 *cfs*/mi^2, despite the Lynx Lake PMP being greater than that for BCL: 11.8 vs. 10.4 *in*. An explanation is that the BCL PMP has a maximum of 5.67 in of precipitation in any given 15-*min* interval while the Lynx Lake PMP has a 15-*min* maximum precipitation of 5.2 *in*.

For the 100-*yr* events, BCL 19-catchment simulation yielded a normalized peak flow of 470 *cfs*/mi^2 while the Lynx Lake 10-catchment simulations were 630 *cfs*/mi^2 (IL+ULR method) and 430 *cfs*/mi^2 (G&A method). The three estimates are similar in magnitude.

The USGS provides regression-based equations for estimating storm discharge (U.S. Geological Survey, 2002). These are incorporated into the National Flood Frequency (NFF) computer program (http://water.usgs.gov/software/nff.html). NFF can be used to estimate flood magnitude versus recurrence interval for any site and was applied in an independent assessment of our simulation results. The NFF rural technique was utilized due to BCL's remote location. The BCL watershed falls on the boundary of two NFF Arizona flood regions: 11, and 12 (Thomas et al., 1994). Regions 11, NE Arizona, and 12, more centrally located, are separated in part by the Mogollon Rim. NFF procedures do permit a watershed that spans two different regions; however, the BCL watershed is not large enough to justify this.

Required NFF parameters consisted of watershed mean elevation, mean annual evaporation (44.1 in, from Thomas et al., 1994) and mean annual precipitation (30 in, from U.S. Weather Bureau, 1963, as reproduced by Arizona Department of Transportation [ADOT, 1994, Fig. 10.10]).

For several reasons, Region 11 was not considered further. The BCL watershed's area and mean annual evaporation each fall on the border of acceptable ranges for region 11 (Thomas et al., 1994) and the application of the region 11 equation is thus believed to be less reliable. Also, the region 11 equation is based on data less robust than those for region 12 and has a higher standard error of regression (Thomas et al., 1994). The Mogollon Rim acts as a natural barrier of precipitation for land to the northeast of the rim; moisture coming from the south is able to reach the Mogollon Rim but begins to dissipate to the northeast of the Rim (Hansen et al., 1984). Therefore, we believe that the precipitation on the Mogollon Rim itself is only affected by the elevation.

For region 12, the NFF forecast of 100-*yr* flow is 3,470 *cfs*, with an associated 39% standard error value. This falls in between our 19-catchment 100-*yr* peak flow HMS simulations for existing and predicted future conditions: 4,709 *cfs* and 2,650 *cfs*, respectively. The nearly 4,700 *cfs* for the existing condition is approximately 1,200 *cfs* higher than the NFF value. Probably, this is because the NFF regression equation is based on gaging of unburned watersheds, which produce lower flows.

Region 1 is an NFF region that does not appear on the Arizona flood region map of Thomas et al. (1994). It represents areas above 7,500 *ft* elevation. A weighted average method (Thomas et al., 1994) is available to address watershed areas near or above this elevation threshold. The procedure is not incorporated into the current version of NFF. The weighted peak discharge value for regions 1 and 12 is nearly 1,625 *cfs*, with estimated standard error of 40 to 45%. The forecasted low value is clearly driven by the low discharge for region 1.

A 100-*yr* IDF was developed in the initial BCL design report (Earle V. Miller Engineers, 1963). The IDF was a constant flow rate of 1,150 *cfs*, which was based on a 100-*yr* 6-*hr* precipitation event of 4.5 *in*. The designers chose a runoff curve number of 70 resulting in a constant direct runoff value of 1.8 *in*, based on USBR (1960) procedures. In order to compare our model with the 1963 results, we modified our 1-catchment future (2008) condition model to use the 1963 uniform rain distribution. Also, we modified the loss parameters from the SCS-CN basis to the 1963 constant loss rate basis, with steady 1.8 *in* of runoff. With these modifications the resulting peak flow rate that we obtained with an HMS simulation was 1,092 *cfs*, just under the 1963 IDF peak of 1,150 *cfs*.

After the Rodeo-Chediski fire, the USACE (2002a) evaluated changes in potential storm water runoff from BCL and other nearby watersheds. Their report provides 100-*yr* 1-catchment hydrographs for unburned and burned watershed conditions. Their 'calibration' target for the 100-*yr* unburned condition was based on a USGS regression equation (Jennings et al., 1994). The USACE 'target' for the burned condition utilized these same equations, with modifications based on CN changes provided in the BAER report (BAER Team, 2002b). Below is a comparison of the 100-*yr* event peak discharges from the USACE report and this evaluation.

USACE evaluation	Peak Discharge (*cfs*)	this evaluation: 1-catchment 100-*yr* 6-*hr* storm	Peak Discharge (*cfs*)
unburned condition	2,190	predicted future (2008) condition	2,064
burned condition	2,990	present (2003) burned condition	4,058

Our predicted watershed response is very similar to the USACE unburned condition result because of our assumption of hydrologic recovery. While an increase in peak discharge is observed for each party's evaluation for the burned condition, the estimates are more divergent, but are still similar in magnitude.

Hydraulics

As mentioned above, the existing BCL spillway (Figure 1), having been designed in 1963 to convey an (attenuated) 100-*yr* 880 *cfs* peak flow, has been deemed inadequate by ADWR. As a prelude to an alternatives evaluation (ongoing), verification of existing spillway capacity was necessary. Due to the steep slope of the spillway chute, downstream of the crest, the capacity is inlet controlled. Our evaluation consisted of:

- Verify the original spillway design flow by routing the original IDF (1150 *cfs* for 6 hours) through the reservoir.
- Completing a 'back-of-the-envelope' analysis, assuming a reasonable range of discharge coefficients.
- Estimating the spillway discharge coefficient following Bradley (1952).
- Assessing the impact of the spillway approach channel conditions (Figure 2), by developing a backwater curve, or water surface profile, from the spillway crest into the reservoir using a HEC-RAS simulation.

Haestad Methods PONDPACK software was used to route the original design hydrograph through the BCL reservoir using the original spillway discharge rating and reservoir stage-volume curves (Earl V. Miller Engineers, 1963). The initial pool elevation for both of these routings was 160 *ft* – the spillway crest elevation. The reservoir area and volume, as a function of elevation, were as provided by the designer, but were verified. The reservoir routing simulation yielded an outflow hydrograph with a peak of nearly 860 *cfs*, which is very close to the 880 *cfs* developed originally.

An upper bound on spillway capacity, for the design reservoir water surface of 164 *ft*, or a design reservoir head, H, of 4 *ft*, can be estimated by applying the equation for discharge over a sharp-crested weir – the control structure that will yield the maximum discharge – the basis for design of the 'Ogee' crest spillway. Note that the design reservoir head, H, is that water surface elevation in a location where velocity is negligible. Using an English-units discharge coefficient, C, of 4, and a reservoir head of 4 *ft*, yields a discharge of 1280 *cfs*. Clearly, the BCL spillway cannot pass a flow much greater than approximately 1300 *cfs* for a design reservoir head of 4 *ft*. For the lower bound, the discharge coefficient for a broad-crested weir with vertical upstream face can be as low as 2.6. A lower bound on spillway capacity is nearly 830 *cfs* for H of 4 *ft*.

Bradley (1952) provided a compilation of model-based discharge coefficients for a wide variety of irregular shapes for both the free overfall (ogee type) and suppressed overfall – typical of the BCL spillway, conditions. Bradley's Alcova (C of 2.85) and Agency Valley (C of 2.73) spillway profiles match the BCL spillway profile fairly well. Our application of Bradley's procedure yields C of 2.79 – for the design reservoir head of 4 *ft*. This C agrees well with the design value of 2.85 (Earl V. Miller Engineers, 1963, 1967), and provides for a discharge capacity of nearly 900 *cfs*. Note that while we refer here to C values with 3 significant digits, accuracy to better than 2 digits is unlikely!

We assessed several assumptions of the original design by developing a backwater curve, or water surface profile from the spillway crest into reservoir, by applying a numerical open-channel flow model to determine the water surface profiles upstream and downstream of the BCL spillway crest. The numerical modeling, while not totally free of its own assumptions, such as 1-dimensional flow, is advantageous in that it dispenses with discharge coefficient assumptions. We used the U.S. Army Corps of Engineers River Analysis System HEC-RAS Version 3.1.1 (USACE, 2002b, 2002c).

Steady-state mixed flow simulation is covered in the HEC-RAS reference manual (USACE, 2002c). Figure 7 illustrates the water surface profile results of the HEC-RAS simulation for a uniform discharge of 880 *cfs*. The water surface profile (backwater curve) from the spillway crest into the reservoir indicates a reservoir water surface elevation of 164.0 *ft* – consistent with the original design assumption. Approach losses are approximately 0.2 *ft*.

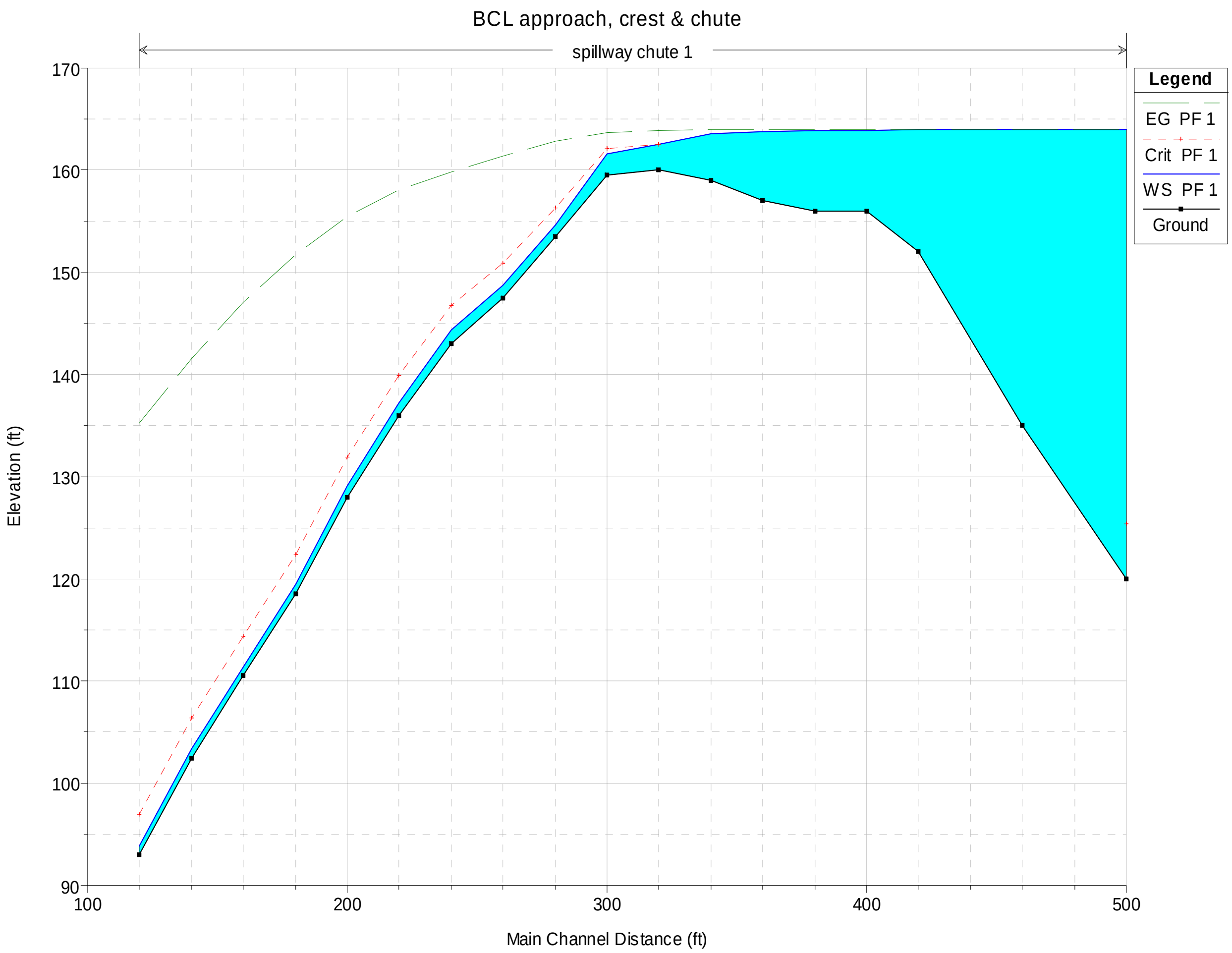

Figure 7. HEC-RAS water surface profile simulation results for steady-state spillway flow of 880 *cfs*. The hydraulic grade line is the water surface. EG is energy grade line, CRIT is water surface profile at critical depth, WS is water surface profile, and GROUND is the elevation along the channel bottom centerline. The spillway crest is at main channel distance of 320 *ft*.

The following particulars may be of interest.

- Expansion coefficient of 0.3 and contraction coefficient of 0.1 for cross-sections upstream of the 20-*ft*-wide chute;
- Expansion and contraction coefficients of 0.03 and 0.01 (respectively) in the chute (justification per USACE, 2002c, pages 3-19 and 3-20 and based on personal communication with Cameron Ackerman, USACE-HEC, February 9, 2004);
- *n* of 0.035 (Chow, 1959) for the rock portion of the approach channel;
- *n* of 0.014 (Chow, 1959) for concrete spillway crest and chute sections;
- Downstream boundary condition of normal depth, upstream (adverse slope) boundary condition of critical depth for the mixed flow steady state simulation.

For comparison with the HEC-RAS simulation results, approach losses were estimated following guidelines provided by LaBoon (2001). The approach losses consist of approach channel friction loss, h_f, and head loss due to constriction from the reservoir to the approach channel, h_c. The combined losses are nearly 0.1 *ft*, which is the same order of magnitude as the approach losses (0.2 *ft*) identified with the HEC-RAS simulation of 880 *cfs* flow.

Conclusions

- The multiple-basin model forecasts PMF flows 10% (present burned condition) to 14% (future recovered condition) greater than PMF flows for a single basin.
- The effects of watershed model subdivision (discretization) on simulation results can be pronounced for small events, such as the 10-year or 25-year storm, but the consequences are less important for the PMF.
- Considering the increase in flood discharges created by the burned condition of the watershed, the PMF is much less sensitive to watershed condition when compared to the sensitivity of shorter-recurrence-interval storm flows to condition. In the case of a PMF the runoff will be extreme regardless of soil or vegetation condition.
- The impact of fire on a watershed can be extreme – especially for small sheds. Yet, recovery of under-story vegetation and soil condition does occur over a period of several years to as many as 5 to 10 years. In some situations (drought, steep slopes, etc.), recovery may take longer or may be incomplete.
- The existing spillway crest at BCL functions similarly to a broad-crested weir, with discharge coefficient of 2.8 for a reservoir water surface elevation of 164 *ft*, with associated flow of 880 *cfs*. A variety of classical and numerical methods yield self-consistent estimates for the overflow spillway discharge coefficient.
- Energy losses (0.2 *ft*) associated with the curving shallow approach channel are minor.

Improvements alternatives being considered for Black Canyon include: increasing storage by raising the dam crest elevation; increasing discharge capacity by widening the spillway channel; hardening the earthen embankment so that it can pass an overtopping flow without significant damage, or some combination of these. Finally, while PMF evaluations tend to focus on structural remedies, watershed condition is an area that deserves serious consideration for design storms of smaller magnitude.

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