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Bonasa Breaks Ranch Dam

Dam Failure and Hydrologic Report

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For more information contact:

Water Resources Program
P.O. Box 47600
Olympia, WA 98504-7600

Phone: 360-407-6872

Washington State Department of Ecology - www.ecy.wa.gov

- Headquarters, Olympia 360-407-6000
- Northwest Regional Office, Bellevue 425-649-7000
- Southwest Regional Office, Olympia 360-407-6300
- Central Regional Office, Yakima 509-575-2490
- Eastern Regional Office, Spokane 509-329-3400

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by

Guy Hoyle-Dodson, P.E.
Hydrology and Hydraulics Specialist

Gus Ordonez, P.E.
Geotechnical Specialist

Dam Safety Office
Water Resources Program
Washington State Department of Ecology
Olympia, Washington

Dam Failure and Hydrologic Report

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This dam failure and hydrologic analysis of the Bonasa Breaks Ranch Dam and the engineering analyses and technical material presented in this report were prepared by the undersigned professional engineers.

8/21/2017

Guy Hoyle-Dodson, P.E.
Hydrology and Hydraulics Specialist
Dam Safety Office
Water Resources Program

August 21, 2017

8/21/17

Gus Ordonez, P.E.
Geotechnical Specialist
Dam Safety Office
Water Resources Program

August 21, 2017

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Executive Summary

This report summarizes the Dam Safety Office's observations and findings of our May 3, 2017, inspection of the breached Bonasa Breaks Ranch Dam, and our subsequent engineering analyses. Our primary objective was to determine the causes of the April 13, 2017, dam breach failure of the Bonasa Breaks Ranch Dam by evaluating the hydrologic response of the reservoir and spillway to potential and design hydrologic and hydraulic operating conditions.

Bonasa Breaks Ranch Dam is located approximately 3.8 miles southwest of Anatone in Asotin County, Washington. The dam is a privately owned, unpermitted homogeneous earthfill embankment. Before the breach, the dam was capable of impounding approximately 36 acre-feet of water (maximum estimated storage). The breach failure released approximately 29.1 acre-feet of water. The embankment has a structural height of approximately 18.6 feet, a crest length of 414 feet, and a crest width varying between approximately 4 and 7 feet. The upstream and downstream faces have approximate slopes (Horizontal:Vertical) of 1.75H:1V and 2H:1V, respectively.

The dam was provided with a single spillway structure, a 3-foot diameter corrugated metal pipe through the upper section of the embankment section, approximately 120 feet from the southwest abutment. A low level outlet was not provided for the impoundment. A shallow trapezoidal channel located at the southwest abutment area also exists, but it does not appear to act as a functional emergency outlet for the impoundment, and may have been drainage for a borrow site at that location.

According to the dam owner and our research there are no plans, specifications or construction inspection reports for the original dam or the later embankment raises. The owner reports that the original dam was built in the 1960s or earlier. In 2008, the Dam Safety Office (DSO) conducted a statewide search for jurisdictional impoundments (impounding 10 acre-feet or more) not included in our Dam Safety inventory. Using 2006 orthophotos (uniform-scale aerial photographs) for that search, it was estimated that the dam impounded less than 10 acre-feet. It was therefore deemed non-jurisdictional and no further action was taken. The dam was modified to its current configuration (with the raised embankment) in 2006 and 2007 based on statements by the owner and as depicted in 2009 orthophotos of the site.

As part of the engineering review of the failure, we inspected and surveyed the embankment crest, breached section, and the upstream and downstream slopes adjacent to the breached section. Hydrologic and hydraulic analyses were performed to examine the adequacy of the spillway to accommodate the runoff from both the assumed inflow at the time of the breach and the dam safety inflow design flood.

Based on the information collected at the site and on the engineering analyses summarized in this report, it is our opinion that that Bonasa Breaks Ranch Dam most likely failed due to inadequate hydraulic capacity of its overflow spillway, resulting in overtopping of and subsequent breach of a section of the embankment.

Other failure scenarios such as a breach caused by “piping” of embankment materials, or overtopping at a point of slope failure, cannot be completely dismissed. Piping is a feature of the foundation that can lead to cracking of embankment materials and resulting leakage problems. We observed a “tube-like” feature in the dam along the breach path that may be associated with piping. Aerial photographs from 2006 and earlier depict a possible tree or large bushes growing on the dam near the breach point. The remnant root system may have decayed and created a piping pathway for failure. We also observed an organic layer (mostly grass) 1-2 feet below the surface of the downstream slope on the northern sidewall of the breach. This organic layer may be considered a weak plane which adversely affected the stability of the downstream slope and potentially led to a loss of freeboard.

Nonetheless, the visual inspection of the embankment revealed signs such as vegetation on the crest and downstream face that was probably bent by water flow; and, spotty accumulations of sediment across the crest and downstream slope that were likely deposited by water flowing over the embankment. Furthermore, there were no visible signs of piping, cracking, and/or slope failure on the downstream face along the remaining sections of the embankment.

In summary, our observations and analysis indicate that the as-modified dam impounded enough water to be jurisdictional under dam safety laws and regulations. Accordingly, the modifications to the dam in 2006 and 2007 should have been completed under review and permitting by the Dam Safety Office.

The unpermitted dam did not meet several of the requirements outlined in the *Dam Safety Guidelines*, most noticeably resulting in:

- Inadequate spillway capacity;
- Inadequate freeboard;
- Inadequate cross section geometry; and,
- Inadequate construction procedures.

Given these inadequacies, a Dam Safety construction permit would not have been issued for the modifications made to the original embankment.

A summary of the pertinent elements of this analysis are included in this report.

Introduction

In the early morning of April 13, 2017, severe flooding and a debris flow were reported on Rattlesnake Creek along State Highway 129 in Asotin County, Washington. Initially thought to be a landslide, it was subsequently identified as a dam breach flood, traced by helicopter flight to its origin at the Bonasa Breaks Ranch (BBR) Dam.

In response to the reported dam failure, the Department of Ecology's Dam Safety Office inspected and surveyed the BBR Dam site on May 3, 2017. The primary objective of the site visit was to inspect the dam and obtain measurements of the embankment and reservoir. The inspection team also visited the downstream areas to observe damage that resulted from the breach.

After returning to the office, the site visit information was used to conduct a number of hydrologic analyses. Additional data and information was gathered from a number of outside sources to support the analyses and conclusions presented in this report.

Background

The BBR Dam is located in southeastern Asotin County, approximately 3.8 miles southwest of the unincorporated community of Anatone, within Township 07 N, Range 45 E, Section 08, at Latitude: 46.0947 and Longitude: -117.1894 (Figure 1). The reservoir is located on Rattlesnake Creek (Washington State DNR, 03/01/2006), which is a tributary of the Grande Ronde River. The property is owned and operated by a Florida Limited Liability Corporation (LLC) named Bonasa Breaks Ranch, LLC. The LLC manager is Mr. Stephen E. Croskrey. The property was purchased by the LLC in 2004 and included a small dam and impoundment.

It is not known when the original dam was built or what function it originally served. The current owner's representative said it has existed at the site since at least the 1960s and is now used for fish rearing and recreation. DSO records contains only cursory information about the impoundment, derived from GIS orthophotos established before the mid-2000s and there has been no DSO involvement or oversight prior to the breach. Orthophotographic records indicate that a rudimentary structure has existed at this location for a number of years, although its impoundment of water appears to be seasonal and, up to at least 2006, on a much smaller scale than the volume impounded before the 2017 breach (Figure 2).

A 2008 DSO survey of unpermitted impoundments in Washington State identified this structure from 2006 orthophotos, but the surface area was estimated to be only 1.2 acres and the volume was estimated to be less than the 10 acre-feet DSO jurisdictional threshold. Accordingly, the dam was not considered jurisdictional and therefore was not included in the DSO dam inventory.

In 2006 and 2007, the owner enlarged the embankment size which increased the reservoir volume. A culvert was also added. The work was conducted by Mr. Ken Thorton (BBR property manager) and Mr. Tom Mullins (a local resident who worked on ponds). The owner reports that no design, construction, or permitting documents exist for the original dam or for the modifications.

The reservoir normal pool surface area increased from approximately 1.2 acres to approximately 4.3 acres. The embankment modifications increased the volume of the reservoir above the 10 acre-feet jurisdictional threshold. Accordingly, this would have made the impoundment jurisdictional, and would have required that a Dam Safety construction permit be issued before construction of the modifications took place. A permit is typically issued after the construction plans, specifications, and engineering reports are reviewed by engineers from the DSO to ensure that the proposed structure complies with the requirements of RCW 90.03.350. Because the modification was never reported to the DSO, the dam design and construction were not reviewed by the DSO. As a result, the impoundment was never included in the DSO's inventory. This modified impoundment continued to operate with an unpermitted status for approximately ten years.

In the early morning of April 13, 2017, severe flooding and a debris flow were reported on Rattlesnake Creek, impacting State Highway 129. Initially thought to be a landslide, it was subsequently identified as a breach flood, traced by helicopter flight to its origin at the BBR Dam (Figure 3). The helicopter was hired by the Asotin County Conservation District and provided continuous video footage from the Grand Ronde River to the BBR Dam site.

It is our initial assessment that the dam breach was the primary forerunner to the observed channel scouring and debris flows in Rattlesnake Creek. Additional hydrologic analyses would be required to evaluate the likelihood that contributing flows off adjacent watersheds to Rattlesnake Creek might have enhanced the scouring of the main channel. A cursory examination of the post-breach photographic record found no signs that adjacent flows were significant contributors.

Dam Breach Inspection and Surveys

On May 3, 2017, the DSO performed an inspection at the site to ascertain dam embankment and breach characteristics, with the goal of determining the impact of watershed runoff on the dam failure. DSO representatives included Joe Witczak, Gus Ordonez, and Guy Hoyle-Dodson. The DSO was accompanied to the BBR Dam site by representatives of the owner: Mr. William Spenser (attorney) and Mr. Ken Thorton (property manager). Also in attendance were representatives of Asotin County (Mr. Karst Riggers), the Asotin County Conservation District (Ms. Megan Stewart), Washington Department of Fish and Wildlife (Mr. Tom Schirm), and other Ecology staff (Mr. Stephen Hummel and Ms. Hallie Ladd). The inspection included a photographic record of the embankment, the remnant reservoir, the breach aperture (Figure 4), a survey of the crest profile, and a cross section at either side of the breached area.

We initially met in Anatone, Washington and, after introductions, briefly discussed the reservoir's history, property access, and the purpose of our visit. We explained the primary objective of the site visit was to inspect the dam and obtain measurements of the embankment and reservoir. We noted that we intended to visit the downstream areas to observe damage that resulted from the breach and that we would prepare a report of our inspection. Mr. Spencer agreed to the inspection and any activities that would explain the cause of the failure. We then caravanned to the dam site.

The breached section of the dam is located approximately 125 feet from the north abutment. A cross section of the breach has approximate dimensions of 42.7 feet across the top and 23.1 feet across the bottom, a maximum breach depth at the center of the embankment of 19.8 feet, with approximate left and right side slopes (looking downstream) of 0.60 H:1V and 0.65 H:1V, respectively. A narrow remnant of the embankment, approximately 3.5 feet in height, continues to impound a remnant reservoir, estimated to have a volume of between 4.9 and 7.1 acre-feet, depending on the remnant surface area assumed.

DSO engineers performed a survey of the embankment crest, breach aperture, and embankment cross section profile (Figure 5 & Figure 16). The BBR Dam is a homogeneous earthfill embankment rising 18.6 feet above original grade (approximate structural elevation 3941.1 feet based on the approximate normal pool rim (estimated from GIS: 10-m DEM) to the lowest surveyed crest elevation of 3959.7 feet. A hydraulic height (depth of the maximum lens of water that could be released) was determined from the embankment survey to be 14.5 feet.

A storage volume was estimated using power equations relating normal pool surface area and hydraulic depth to the volume (Walder and O'Connor, October 1997). This equation's dimensionless exponent and the normal pool volume were iteratively adjusted to match both the measured normal pool surface area and estimated bottom surface area (determined in Arc GIS), producing the final normal pool volume. This procedure calculated a normal pool reservoir storage volume of approximate 18.3 acre-feet with a surface area of 4.26 acres, at a spillway invert overflow elevation of 3957.7 feet (normal pool elevation). From the post-breach water surface elevation to the lowest surveyed crest elevation of 3959.7 feet, the reservoir is calculated to have stored an approximate volume of 29.1 acre-feet (lens of water released during the breach) with a surface area of 6.47 acres. The maximum total pool volume measured from the crest to the bottom of the reservoir is estimated to be 36.2 acre-feet.

An inspection of the post breach embankment and reservoir revealed several noticeable features:

- Erosion features, vegetation that appeared to be laid down flat and spotty sediment accumulations on the downstream face likely resulting from water sheeting down the embankment strongly suggest that the dam may have been overtopped (Figure 6). Overtopping could have led to an erosion failure of the embankment and a sudden release of the reservoir's contents.

- A debris line observed along the rim of the reservoir indicating that the reservoir was particularly high prior to the breach event. DSO surveyed a few locations along the north side of the reservoir and estimated that the debris line was at an approximate elevation of 3959.0 feet, i.e. approximately 0.72 feet below the lowest surveyed crest elevation measured along the dam crest. Although the debris line appeared to be below the lowest crest elevation, other observations indicate that the reservoir water appeared to have flowed over the embankment crest and downstream slope before the breach. It should be noted that drawdown action during the breach event may have swept the observed debris line below its highest elevation before the breach and that the elevation on the crest in the vicinity of the breach may have been lower than the surveyed minimum.

- The embankment appears to have been built of a fine silty sand/sandy silt soil, probably excavated from the reservoir area or areas around the site. Some of the embankment materials were moist to wet depending on their location on the sidewalls of the breached section. Visual observation of the breach walls indicated that other than the presence of a small number of pockets/layers of coarse material and of color changes in material, there were no visible signs of layering or of significant material changes on the core of the embankment section.

  On the northern sidewall, there were signs of what appears to be at least one raise of the embankment as noted below. This was not apparent from visually inspecting the southern sidewall of the breach. We were not able to closely inspect some of the areas on the breached section due to their elevation and the unstable state of the upper sections of the sidewalls.

  There were no signs that features to control seepage through the embankment, e.g., chimney and/or toe drains, were provided for the original and/or modified embankments.

- As noted before, the owner reported that there are no records of the construction work; accordingly, it is difficult to evaluate the level of compaction of the embankment materials based only on visual observations. If necessary, the embankment material properties required for further geotechnical stability analyses would need to be determined through a comprehensive soils exploration program.

- Signs of a rapid drawdown failure (slumping) on the upstream slope of the dam were visible along the northern side of the breached section (Figure 7). We also observed some cracking on the upper section of the upstream slope along the embankment section near the overflow spillway pipe (Figure 8).

- A layer of organic material was observed 1-2 feet below the surface of the downstream slope on the northern sidewall of the breach (Figure 9). This layer was visible from the downstream toe up to approximately halfway up the downstream slope, and its alignment was parallel to the slope face.

It appears that during construction of the most recent embankment raise, the layer of vegetation covering the downstream face was not removed prior to placing structural fill. Further, the thickness/width of the uppermost soil layer clearly indicates that proper methods of construction; e.g., benching into existing slope and providing adequate width of placement area to accommodate heavy construction equipment, were not followed during construction of the embankment raise.

This organic layer may be considered a "weak" plane parallel to the slope which may adversely affect the stability of the downstream slope. It is noted that there were no visible signs of movement along the remaining embankment sections.

- The foundation of the breached section appeared to be formed by a hard, rock-like material. The north side of the foundation exposed at the breach was at an elevation lower than the southern section (Figure 10). This specific zone appeared not to have been properly prepared during original construction of the dam in order to prevent stress concentrations and differential settlement within the embankment. This may lead to vertical cracking which may cause leakage related problems. This failure mode may be compounded if the embankment materials were not properly compacted.

  Typically, a cutoff trench would also be required to block or intercept any seepage along the interface between the structural fill and the foundation material. There were no visible signs that such a cutoff trench was provided for the original embankment.

- A "tube-like" feature was observed on the northern sidewall of the breached section (Figure 11). Features like this are typically associated with piping of embankment materials. There were no visible signs on the sidewall that this feature extended towards the downstream toe or that it may have been hydraulically connected to the reservoir. On the other hand, these features may have been washed away by the dam breach flood waters. It may be noted, that no visible signs of piping (seepage, saturated soils, voids, etc.) were observed on the remaining embankment sections, although this may not pertain to the section that did fail.

- A 3-foot diameter corrugated metal pipe was installed on the upper section of the embankment, approximately 120 feet from the southwest abutment. Installation of this type of conduit requires careful construction practices, as it is difficult to achieve acceptable compaction levels of the backfill material, especially under the pipe. It appears that compaction of the backfill material was not adequate as it was observed that a void had developed under the pipe at both the upstream and downstream side of the dam (Figure 12). It is our opinion that this void was likely created by water seeping along the alignment of the pipe.

  A visual inspection of the pipe interior revealed some damage along the crown of the pipe (Figure 13), which the DSO believes was caused by vehicular traffic on the embankment compounded by an inadequate amount of cover material above the pipe. There was also damage on the bottom of the pipe; however, it is not clear what may have caused it. Hence, it is apparent that adequate construction practices were not followed during installation of the overflow pipe. It is estimated that the deformation of the conduit would have reduced its capacity by less than 10 percent.

It was also observed that an energy stilling basin was not provided at the downstream foundation area where the flows from the pipe discharged. Some erosion of embankment material was observed directly below the section of the conduit on the downstream slope (Figure 14). At minimum, some kind of erosion resistant layer should have been provided in this area to prevent erosion of the foundation and embankment materials.

- An unlined trapezoidal channel was excavated on native ground adjacent to the right abutment (Figure 15). This channel is approximately 10 feet wide at the top, 1-2 feet wide at the bottom, and 2 feet deep, with an invert elevation of approximately 3959.7 feet. This channel would not have functioned as an emergency overflow spillway, since its invert elevation is equivalent to the minimum crest elevation. Thus it would have offered no contribution to the ability of the dam to pass the inflow design flood and still maintain a design freeboard. Therefore, DSO considered this feature unrelated to dam design and operation.

Observations of Dam Design Adequacy

The overflow outlet conduit was found to have an inlet invert elevation of 3957.69 feet, which constitutes the impoundment's normal pool elevation. This represents an embankment normal pool freeboard of 2.04 feet, which meets DSO guidelines. For a small dam (hydraulic height less than 15 feet), a minimum pool freeboard of 0.5 feet is required, although a design freeboard that includes the effects of wind/wave action might be slightly greater. From the survey, the minimum freeboard elevation would be at an elevation of 3959.23 feet and it would be required that the water surface not exceed this elevation during the Inflow Design Flood (IDF).

The embankment crest width was extremely narrow, with a width of 4 feet on the embankment section at the north side of the breach; and, 7 feet on the embankment section at the south side of the breach (Figure 16). *Dam Safety Guidelines Part 4, Dam Design and Construction, 3.2.2 Requirements\Minimums for the Embankment Cross-Section* require a minimum crest width of the larger of 8 feet or the value obtained from $2(H)^{1/2} + 3$, where H is the dam height. For a dam height of 18.6 feet, a minimum crest width of approximately 12 feet would be required. This is greater than the measured width of 4-7 feet. In practice, the crest width is usually controlled by the dimensions of the heavy construction equipment, and minimum crest widths of 15 feet are typical.

Embankments designed and constructed following acceptable methods for dams typically have upstream and downstream slopes of 3H:1V and 2H:1V, respectively. However, this is also checked through engineering analyses. For the BBR Dam, the upstream and downstream slopes were measured at approximately 1.78H:1V and 2.09V:H, respectively.

Based on the narrow crest width, steep upstream slope and lack of a chimney/toe drain, the embankment as-modified would not have met the minimum stability requirements for this type of structure as outlined in the DSO guidelines.

The inspection and subsequent analyses also determined that the overflow spillway did not have the adequate capacity required to pass the Inflow Design Flood (IDF) off the watershed to maintain the proper freeboard below the lowest surveyed crest elevation, based on the anticipated downstream hazard. A hazard analysis identified the impoundment as a Significant, Hazard Class 2E, which translates into a Design Step 3, requiring a Step 3 Design Storm.

The following hydrologic and hydraulic analyses were performed to determine the extent of the breach flood, the IDF, and whether the spillway had the capacity to pass the design storm. Ultimately, the goal was to determine what storm event the conduit could actually pass and still maintain the design freeboard.

Breach Hydraulics

To better characterize the breach flood impacts, a dam breach analysis was conducted using equations developed by MacDonald and Langridge-Monopolis (1984), as described in *Dam Safety Guidelines, Technical Note 1* (Revised 2007). Previously, a stage-storage curve was calculated for the reservoir and a maximum volume estimated (Figure 17). From this volume, a breach formation factor (BFF) was developed and a breach geometry (bottom width and cross sectional area) was calculated. A breach hydrograph with peak discharge, QP, was calculated using an empirical equation based on breach development time and width (Fread, 1981).

A step-wise calculation produced a peak breach discharge of 2507 cfs, reached 10 minutes after the initial breach (Figure 18). The hydrograph demonstrates that the greater part of the breach volume would have been released from the reservoir over a 24-minute period. It is our opinion that this large, compressed peak flood wave, traveling down the very steep gradient of Rattlesnake Creek's channel had the potential to scour a large amount of debris and sediment from the channel. It is assumed that the resulting bulking would have increased the downstream breach volume several fold.

Watershed Climatic and Hydrologic Characteristics

A characterization of design storm hydraulics was performed to determine if the overflow conduit was adequately designed to handle the Inflow Design Flood (IDF) off the watershed. As described in *Dam Safety Guidelines, Technical Note 3* (Revised 2009), the BBR Dam watershed is located within Climatic Region 13, Northeastern Mountains/Blue Mountains of Eastern Washington. Watershed elevations range from 3939 to 4090 feet. The watershed surface area is approximately 0.223594 square miles (approximately 143 acres), with a mean annual precipitation of 21-inches (PRISM).

The following data are representative of typical storms for this watershed:

- The 2-year 24-hour precipitation is 1.275 inches.
- The 2-year 6-hour precipitation is 0.798 inches.
- The 2-year 2-hour precipitation is 0.509 inches.

These data will be used to estimate rainfall depth parameters for the dam safety design storm.

For the purposes of the hydrologic calculations, snowpack was assumed to lie on the impoundment's watershed during the design storm. Daily Climate Summaries for the period of record at station 450184, Anatone show that maximum late-winter snowpack (April & May) is 3.9 inches deep, with a 20 percent water content of 0.78 inches. Short-duration thunderstorms, intermediate storms, and long duration storms are all candidates for this late season period, although the short storm did not include snowmelt.

Selection of Design Storms

As outlined in DSO Guidelines, for the determination of spillway adequacy, a design storm needs to be selected to calculate the IDF that the spillway must be able to pass, while still maintaining the design freeboard below the crest.

Downstream Hazard. The DSO downstream hazard classification considers the potential risk to downstream people, economic loses, and environmental damages as provided in WAC 173-175-130(4)(g). The breach flood from the BBR Dam failure discharged to Rattlesnake Creek, then traveled 6.8 miles to discharge into the Grande Ronde River. The consequences of the actual failure of the dam included damage to a vacation trailer on Rattlesnake Creek and flooding across a public road (SR129). While the damage and road flooding did not result in any casualties or bodily injury, this situation created a potential estimated population at risk of 1 to 3 people. In addition, the dam failure caused severe damage to a number of public and private properties including roads, particularly State Highway 129. The Washington State Department of Transportation (WSDOT) has reported the cost of SR129 repairs to be approximately $730,000. The flooding also damaged a Washington State Department of Fish and Wildlife (DFW) bridge over Rattlesnake Creek near the confluence with the Grand Ronde River. DFW estimates the cost to repair the bridge at approximately $70,000. Finally, significant environmental damage to the riparian habitat along Rattlesnake Creek occurred, including damage to Asotin County Conservation District tree planting projects, damage to a WSDOT salmon restoration project at the SR129 culvert, and sedimentation impacts to Rattlesnake Creek and the Grande Ronde River.

A Design Step Analysis was performed, using methodology from the *Dam Safety Guidelines, Technical Note 2: Selection of Design/Performance Goals for Critical Project Elements*. Based largely on environmental damages and economic losses, this analysis determined a Cumulative Consequence Rating Points of 349.3, which yields an Annual Exceedance Probability (AEP) of $3.16x10^{-4}$. This is equivalent to a Design Step 3. Based on this downstream hazard characterization, the dam is classified as having a **Significant, Hazard Class 2E** downstream hazard setting.

Design Step. The design rainfall event corresponds to Step 3 of an 8 step criteria used by DSO. This event has an annual exceedance probability of 1 in 3,000, and an exceedance probability of 1 in 30 over a 100-year period. Based on the downstream hazard assessment, a Step 3 Design Storm is appropriate for the BBR Dam's hydrologic analysis and spillway design capacity evaluation. This design step is used to derive the rainfall depths associated with the Dam Safety Design Storm.

Design Precipitation. This analysis examined three storm scenarios to identify which one would be most critical for peak water level in the reservoir and peak flow through the spillway. The analysis also applied these storm scenarios to sequentially more frequent design steps (down to the 25-year event) to determine the smallest Inflow Design Flood (IDF) that the overflow spillway conduit would be able to pass and still preserve the reservoir water surface below the dam crest. The analysis used time-distribution hyetographs as given in *Dam Safety Guidelines, Technical Note 3* (2009), based on Schaefer's methodology for extreme storms (Schaefer, 1993, updated 2008).

For the Design Step 3 assessment it was found that the Step 3, short duration storm has a 2-hour rainfall depth of 2.66 inches and a 4-hour rainfall depth of 3.34 inches, assuming that this analysis referenced a new project. The rainfall depths for sequentially less to more frequent storms are as follows:

Table 1. Design Step 3 assessment for short duration rain events (in inches)

Design Storm	2-hour (core)	4-Hour (new project)
Step 3	2.66	3.34
Step 2	2.09	2.63
Step1	1.81	2.27
100-Year	1.28	1.60
25-Year	0.931	1.17

The Step 3, intermediate storm has a 6-hour rainfall depth of 2.75 inches and an 18-hour rainfall depth of 5.15 inches. The design event is a rain-on-snow event, with snowmelt of 0.78 inches in addition to the rainfall, for a total precipitation depth of 5.93 inches. The rainfall depths for sequentially less to more frequent storms was as follows:

Table 2.Step 3 assessment for intermediate length storms and rain on snow events (in inches)

Design Storm	6-hour (core)	18-Hour (new project)	Precipitation + Snowmelt
Step 3	2.75	5.149	5.93
Step 2	2.34	4.381	5.16
Step1	2.12	3.969	4.75
100-Year	1.65	3.089	3.87
25-Year	1.31	2.453	3.23

The Step 3, long duration storm has a 24-hour rainfall depth of 4.05 inches and a 72-hour rainfall depth of 6.74 inches. The design event is a rain-on-snow event, with snowmelt of 0.78 inches in addition to the rainfall, for a total precipitation depth of 7.52 inches. The rainfall depths for sequentially less to more frequent storms was as follows:

Table 3. Step 3 assessment for long duration storms and rain on snow events (in inches)

Design Storm	24-hour (core)	72-Hour (new project)	Precipitation + Snowmelt
Step 3	4.05	6.74	7.52
Step 2	3.52	5.86	6.64
Step1	3.22	5.36	6.14
100-Year	2.58	4.29	5.07
25-Year	2.08	3.46	4.24

<u>Areal adjustments</u>. The drainage area is reasonably small *and* the mean annual precipitation and two-year rainfall depths for the various, *or* long and intermediate, storm durations are reasonably constant across the BBR Dam watershed. Computation of the design precipitation for the geographical center of the watershed is considered adequate for these storms, without further adjustment for watershed area.

Watershed Description and Runoff Characteristics

Drainage area. The BBR Dam watershed has six major soil delineated sub-basins with a total drainage area of approximately 0.2236 square mile. The drainage areas were estimated from an analysis in ArcGIS of USGS 10-meter DEMs using the Texas A&M Hydrologic Analysis Program, in conjunction with SSURGO soils coverage data. Land uses in the watershed are predominantly Woods or Forest Land: Post-Developed Conditions in Poor condition (forest litter, small trees, and brush are destroyed by heavy grazing or regular burning), with typical impervious areas of 3 percent. The water from all sub-basins is combined in a small continuous channel and enters the reservoir at its northwest boundary.

Watershed soils. Soils within the BBR Dam watershed are predominantly Silt Loams with stony intrusions. The runoff potential for these soils places them generally within Hydrologic Soil Group C and D. The total weighted subsurface infiltration rate is approximately 0.287 inches/hour. Deep weighted infiltration rates generally range from 0.0001 to 0.02 inches/hour, with a weighted total for the entire watershed of 0.042 inches/hour. SCS curve numbers assigned to these combinations of soil types and land uses range from CN 74 to CN 79 for typical antecedent moisture conditions (AMC II), with a weighted average value of CN 75 being representative of the overall watershed. Runoff potential for frozen soils is estimated to be similar to wet soil moisture conditions (AMC III), with corresponding weighted SCS curve numbers of CN 88.19. The weighted temporary water absorption capacity of the surficial soil layer is 2.36 inches of water.

Time of concentration. For each sub-basin, the time of concentration for direct surface runoff is computed by adding the travel times for sheet flow, shallow concentrated flow, intermittent channel flow, and continuous channel flow (WSDOT, 1995, chapter 3). The time of concentration is then used to estimate the time lag for the Soil Conservation Service (SCS) unit hydrograph for each sub-basin (SCS, 1972, chapter 15). The time lag for the Bureau of Reclamation (USBR) unit hydrograph is computed based on flow lengths, slopes, and surface roughness within each sub-basin (USBR, 1987, chapter 3; see also Viessman et al, 1977, chapter 4). For this watershed, hydraulic lengths for the various sub-basins range from 655 feet to 2480 feet. The times of concentration for surface flow range from 15 to 48 minutes for the individual sub-basins. The time lag for the SCS unit hydrographs range from 2.4 to 29 minutes, with a weighted average of 21 minutes. The SCS curve number method give a basin lag of 17 minutes. The time lag for the USBR unit hydrographs is 40 minutes.

Interflow. A "time of concentration" for interflow runoff (shallow subsurface flow) is estimated as follows. Interflow velocity is estimated as a percentage of sheet flow velocity, then the interflow travel time is added to the travel times for shallow concentrated flow, intermittent channel flow, and continuous channel flow to find a travel time roughly an order of magnitude larger than the surface runoff travel time (see King County SWM, 1992, pages 15 and 20; see also Barker and Johnson, 1995). This interflow "time of concentration" is then used to estimate the time lag for an SCS unit hydrograph for interflow for each sub-basin. For this watershed, interflow "time of concentration" is 3 hours and 50 minutes.

Inflow to the subsurface system is equal to the losses from the surface system, i.e., the rainfall that did not become direct surface runoff. An infiltration "hyetograph" for each storm scenario is estimated based on the storm hyetograph with peak intensities limited by surface infiltration rates for the soils in the watershed. The infiltration "hyetograph" is then used in conjunction with the interflow unit hydrograph and deep infiltration loss rate to estimate interflow runoff. The storm runoff hydrograph for each sub-basin is computed as the sum of the direct surface runoff and interflow runoff hydrographs.

Reservoir and Spillway Description

Reservoir. Hydraulic data for the reservoir/pond includes stage-surface area-storage volume relationships for the reservoir/pond, and section and profile data for the principal and auxiliary spillways. At a normal pool spillway overflow water surface elevation of 3957.69 feet, the reservoir stores a volume of approximately 18.3 acre-feet with a surface area of 4.26 acres. At a water level equal to the dam lowest surveyed crest elevation of 3959.73 feet, the reservoir/pond would store a volume of 29.1 acre-feet with a surface area of 6.47 acres.

Following the methodology of *Dam Safety Guidelines Part IV, Section 4.6*, the required, minimum freeboard for the BBR Dam during normal pool conditions would be 2.0 feet. Required freeboard during inflow design flood (IDF) condition is 0.5 feet.

Spillways. The principal spillway for BBR Dam is a 3-foot diameter corrugated metal pipe located approximately 120 feet from the south abutment. The inlet invert overflow elevation is 3957.69 feet. The outlet discharges to the downstream toe area of the dam, with an outlet invert elevation of 3954.81 feet. An analysis of the conduit capacity under sequential reservoir elevations (head) produces a defining Stage-Discharge Curve (Figure 19). With the reservoir water level at the dam crest, the principal spillway has a hydraulic capacity of 20.4 cfs. At the minimum freeboard the spillway has a capacity of only 11.9 cfs.

Dam Safety Inflow Design Flood and Spillway Adequacy

Computer model. The hydrologic analysis utilized the US Army Corps of Engineers HEC-HMS hydrology computer program (USACE, 2010). The analysis used the watershed, reservoir, and spillway characteristics described in the previous sections. The analysis computed the rainfall from each of the three design storms, plus snowmelt that would occur during the intermediate and long duration storms, then computed and compared the runoff from each storm scenario. The analysis used the Snyder unit hydrograph transform method, and the Holtan infiltration equation to estimate direct surface runoff, and the SCS unit hydrograph with a longer lag time calculated from the time of concentration to estimate interflow runoff.

Storm and reservoir scenarios. The analysis ultimately considered 2 combinations of storms and reservoir conditions for 5 storm events:

- Step 3, Step 2, Step1, 100-year, and 25-year storms with rainfall on snowmelt, initial lake level at spillway overflow elevation 3957.69 feet, and the dam crest overflow was treated as a weir.

- Step 3, Step 2, Step1, 100-year, and 25-year storms with rainfall only, initial lake level at spillway overflow elevation 3957.69 feet, and the dam crest overflow was treated as a weir.

Inflow design flood. Of the three dam safety design storms, all three storms were found to be critical for peak water level in the reservoir and peak flow through the spillway. For the Design Step 3, all storms produce reservoir water surface elevations above the lowest surveyed crest elevation, even with the overflow spillway fully discharging (Table 4). Accordingly, for the Short, Intermediate, and Long Duration Step 3 Design storms, the spillway has inadequate capacity to pass the IDF and prevent overtopping.

The Short Storm (Thunder Storm) was found to be the controlling storm, producing the highest reservoir elevation of approximately 3960.0 feet, 3.2 inches above the lowest surveyed crest elevation and 9.2 inches above the minimum design freeboard. Based on the watershed parameters described above, the computed Short Storm IDF has a peak inflow volume of 36.4 acre-feet exceeding the peak reservoir storage of 30.3 acre-feet by 6.1 acre-feet. Other storm scenarios produced even greater inflow volumes, overwhelming the reservoir's storage capacity. The Short Storm's peak discharge of 210.7 cfs, exceeds the conduit's maximum discharge capacity required to maintain the design freeboard by almost 199.0 cfs.

This scenario is repeated for both the Intermediate and Long duration Step 3 Design Storms, with the spillway's discharge capacity required to maintain the design freeboard, being significantly less than what would be needed to pass the IDF peak. It is clear that the existing overflow spillway design would not have met DSO design guidelines.

Analysis attempted to determine the storm that the spillway could pass while still maintaining the design freeboard. Sequential reduction in the design storm found that both the Long and Intermediate duration 25-year Design Storms produce reservoir water surface elevations that exceed the required freeboard elevation (Table 5).

Spillway Performance

The spillway as designed did not have adequate hydraulic capacity to safely accommodate the Inflow Design Floods for the Step 3, Step 2, Step 1, 100-year, and 25-Year design storms. Depending on the Step 3 Storm scenarios, the reservoir water surface is 9.6 to 3 inches above the minimum required freeboard as stipulated per *Dam Safety Guidelines, Part IV, Dam Design and Construction, Section 4.6*.

Conclusions

The flowing conclusions can be made about the Bonasa Breaks Ranch Dam April 13, 2017 dam failure:

- The dam, as-modified in 2006 and 2007, would have been considered jurisdictional by the DSO. The owner should have obtained a dam safety permit before enlarging the dam. Use of the dam safety permitting process would have ensured the review by a qualified engineer and use of appropriate design, construction, inspection, maintenance, and emergency response standards.

- It appears that the dam was modified without the input from an experienced engineer and that acceptable practices were not followed during construction. A lack of design rigor both violated the general requirement of the *Dam Safety Guidelines* and accepted dam engineering practices. This lack of engineering oversight led to several deficiencies that each could have contributed to the embankment's failure. The most prominent deficiencies include:

    - Inadequate spillway capacity;
    - Inadequate freeboard;
    - Inadequate cross section geometry; and,
    - Inadequate construction procedures.

- Based upon the information gathered to date, it is DSO's opinion that the flood wave resulting from the failure of the embankment was probably the principle cause of downstream flooding and debris flows. However, at this time we cannot completely dismiss contributions from other watersheds along Rattlesnake Creek. A more complete hydrologic/hydrology analysis will need to be conducted to this end. Further, a field inspection of any creeks that enter Rattlesnake Creek from these watersheds should be conducted to determine if there are signs of erosion, debris flows and other indication that significant runoff from these watersheds may have also contributed to the flood on Rattlesnake Creek at the time of the failure.

- The impoundment still retains appreciable volumes of water and may still exceed DSO jurisdictional limits under extreme precipitation events. Therefore, the impoundment will need to be properly modified to ensure that there are no future releases that may cause downstream damage. This must occur by November 2017 before winter rains begin.

- To date, the property owner has been responsive and professional in providing access and information, as well as addressing follow-up actions needed to stabilize the site. The LLC has retained a licensed engineer (Mr. John North) who has already developed and shared draft plans to stabilize the site.

References and Resources

Barker, B.L., and D.L. Johnson. *Implications of Interflow Modeling on Spillway Design Computations*. In: ***Dam Safety 1995, ASDSO Annual Conference Proceedings***. Association of State Dam Safety Officials. 1995.

King County Surface Water Management Division. ***Modifications to the SBUH Method to Improve Detention Pond Performance***. King County Surface Water Management. April 1992.

National Weather Service. ***Probable Maximum Precipitation – Pacific Northwest States***. Hydrometeorological Report No. 57 (HMR-57). NOAA-NWS. October 1994.

Pilgrim, D.H., and I. Cordery. *Flood Runoff*. In: Maidment, D.R. ***Handbook of Hydrology***. McGraw-Hill. 1993.

Ries, K.G. III, and M.Y. Crouse. ***The National Flood Frequency Program, Version 3: A Computer Program for Estimating Magnitude and Frequency of Floods for Ungaged Sites***. U.S. Geological Survey Water-Resources Investigations Report 02-4168. 2002.

Ries, K.G. III. ***The National Streamflow Statistics Program: A Computer Program for Estimating Streamflow Statistics for Ungaged Sites***. U.S. Geological Survey Techniques and Methods 4-A6. 2007.

Schaefer, M.G. ***Characteristics of Extreme Precipitation Events in Washington State***. Washington State Department of Ecology Publication No. 89-51. October 1989.

Schaefer, M.G. ***Dam Safety Guidelines, Technical Note 2: Selection of Design/Performance Goals for Critical Project Elements***. Washington State Department of Ecology Publication No. 92-55F. July 1992.

Schaefer, M.G. ***Dam Safety Guidelines, Technical Note 3: Design Storm Construction***. Washington State Department of Ecology Publication No. 92-55G. April 1993.

Schaefer, M.G., and B.L. Barker. ***Dam Safety Guidelines, Technical Note 3: Design Storm Construction***. Washington State Department of Ecology Publication No. 92-55G. October 2009.

Schwab, G.O., R.K. Frevert, T.W. Edminster and K.K. Barnes. ***Soil and Water Conservation Engineering***, Second Edition. John Wiley & Sons. 1966.

Sumioka, S.S., D.L. Kresch and K.D. Kasnick. ***Magnitude and Frequency of Floods in Washington***. U.S. Geological Survey Water-Resources Investigations Report 97-4277. 1998.

U.S. Army Corps of Engineers. ***HEC-HMS Hydrologic Modeling System***, version 4.2. USACE Hydrologic Engineering Center (Davis, CA). 2016.

U.S. Bureau of Reclamation. ***Design of Small Dams***, Third Edition. USBR. 1987.

U.S. Geological Survey. ***StreamStats: A Water Resources Web Application*** *(application for Washington State).* USGS – http://water.usgs.gov/osw/streamstats/Washington.html. Reports generated, Beta Release: March 2016.

U.S. Department of Agriculture, Natural Resources Conservation Service, Soil Survey Geographic (SSURGO) database for Asotin County Area, Washington (Asotin and Garfield Counties), Fort Worth, Texas, 2012/06/22

Publisher: U.S. Department of Agriculture, Natural Resources Conservation Service
USDA Soil Conservation Service. ***National Engineering Handbook Section 4 -- Hydrology***. USDA-SCS. 1972.

Viessman, W. Jr., J.W. Knapp, G.L. Lewis and T.E. Harbaugh. ***Introduction to Hydrology***, Second Edition. Harper & Row. 1977.

Washington State Department of Natural Resources, **Hydrography layer, Watercourse (WC, WCHYDRO)**, Personal GeoDatabase Feature Class, vector digital data, Publication date: 03/01/2006.

Washington State Department of Transportation. ***Highway Runoff Manual***. M 31-16. WSDOT. November 2011.

Washington State Department of Transportation. ***Hydraulics Manual***. M 23-03. WSDOT. June 2010.

Water Resources Program, Dam Safety Section. ***Dam Safety Guidelines, Part IV: Dam Design and Construction***. Washington State Department of Ecology Publication No. 92-55D. July 1993.

Western Regional Climate Center. *Period of Record Daily Climate Summary – Station*: *450184 Anatone, Washington,* WRCC – https://wrcc.dri.edu/cgi-bin/cliMAIN.pl?wa0184. Period of Record: 7/1/1984 - 9/30/2004

Appendices

Appendix A. Figures

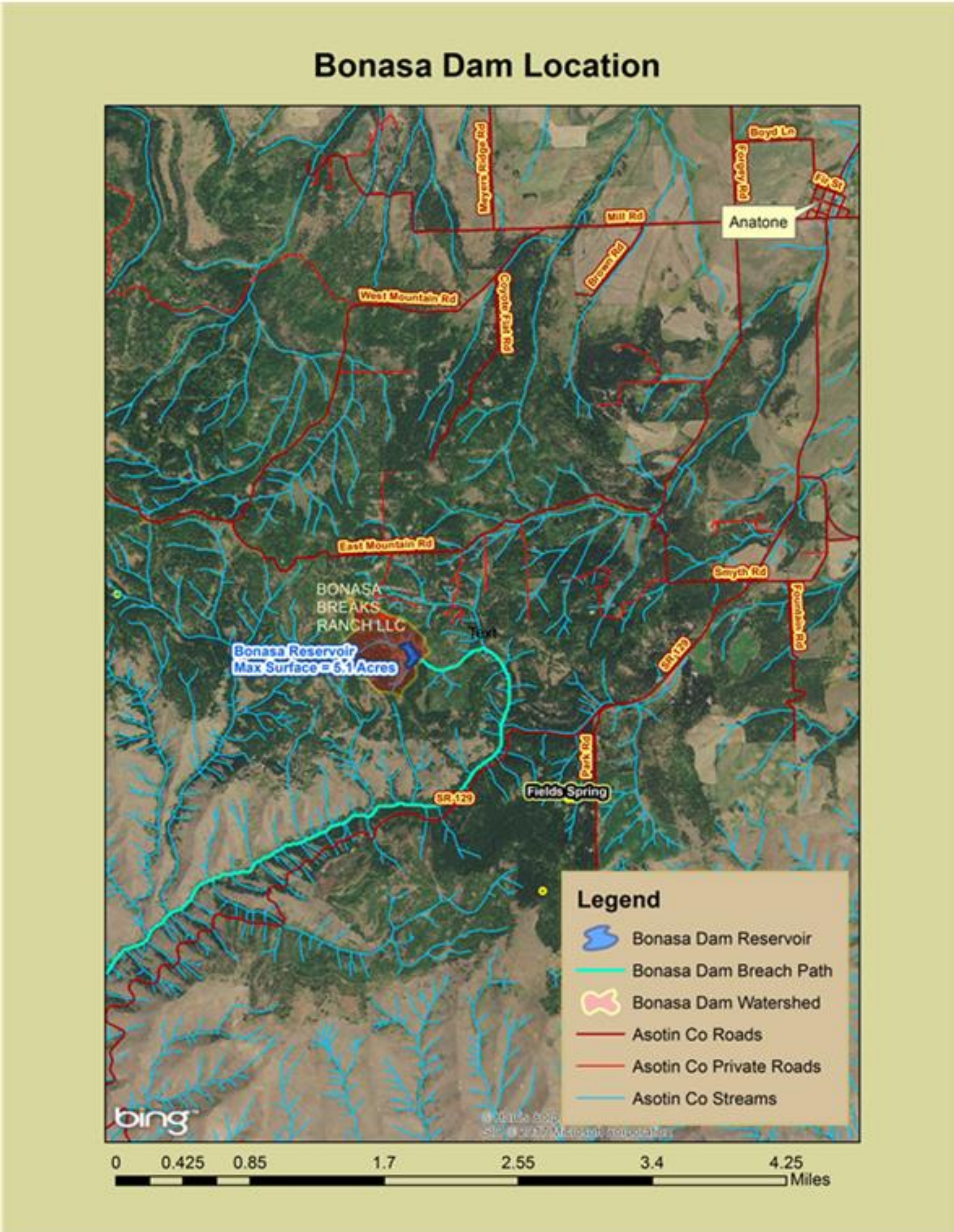

Figure 1. Location map

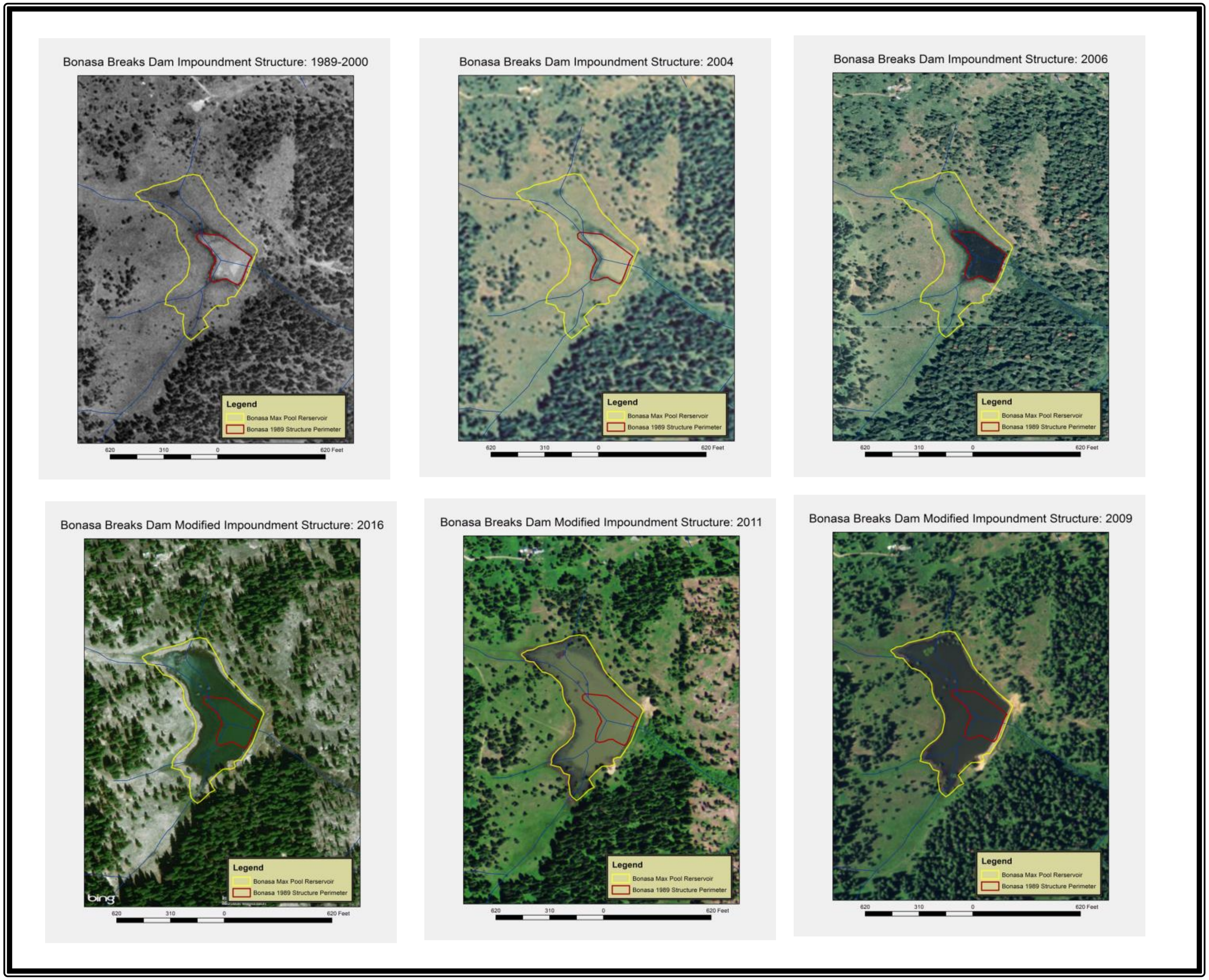

Figure 2. Bonasa Breaks Ranch Dam: Impoundment development (Clockwise from upper left corner)

Figure 3. Remnants of the debris flow on Rattlesnake Creek and State Highway 129 from Bonasa Breaks Ranch Dam breach

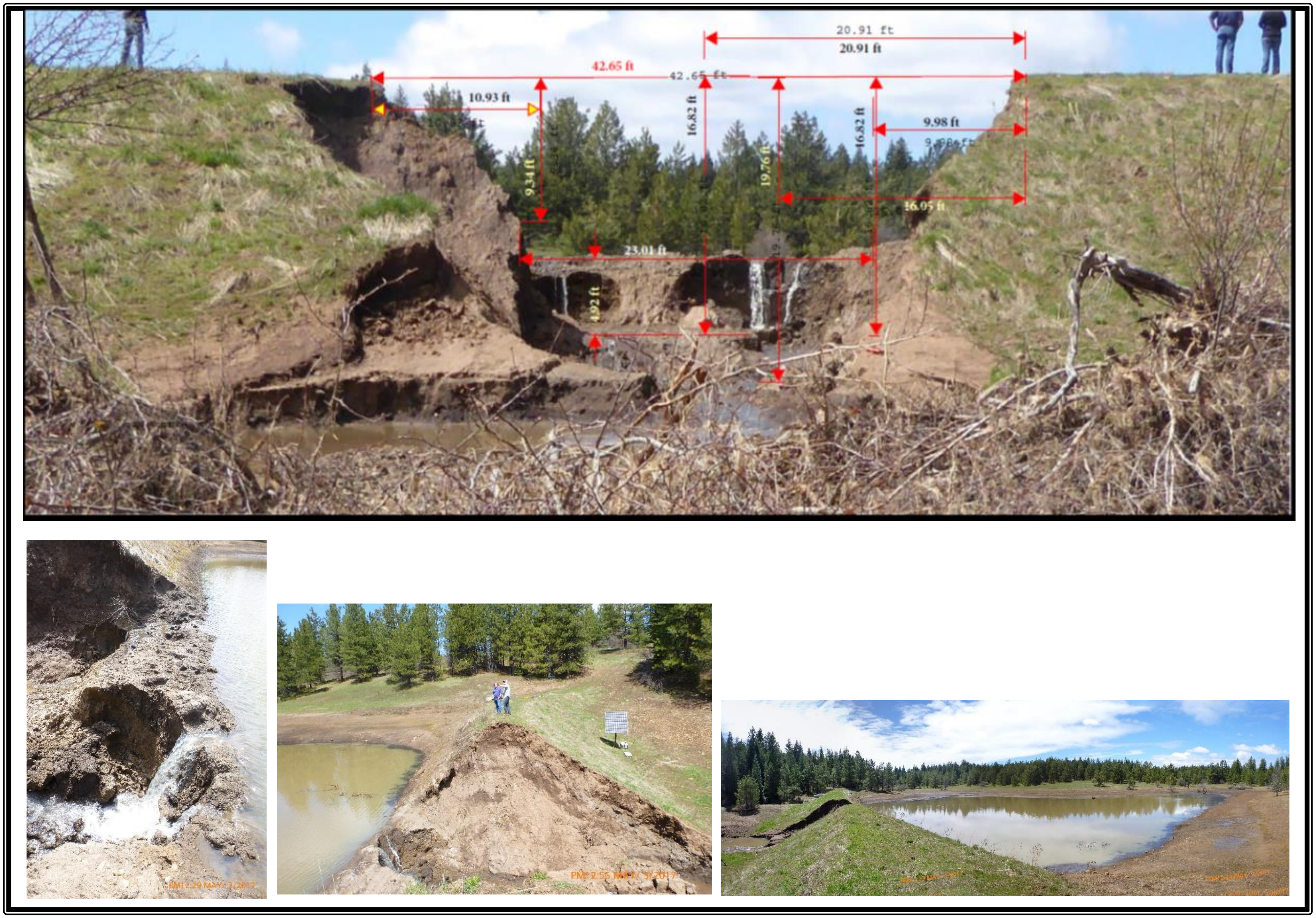

Figure 4. Bonasa Breaks Ranch Dam breach and reservoir remnant characteristics

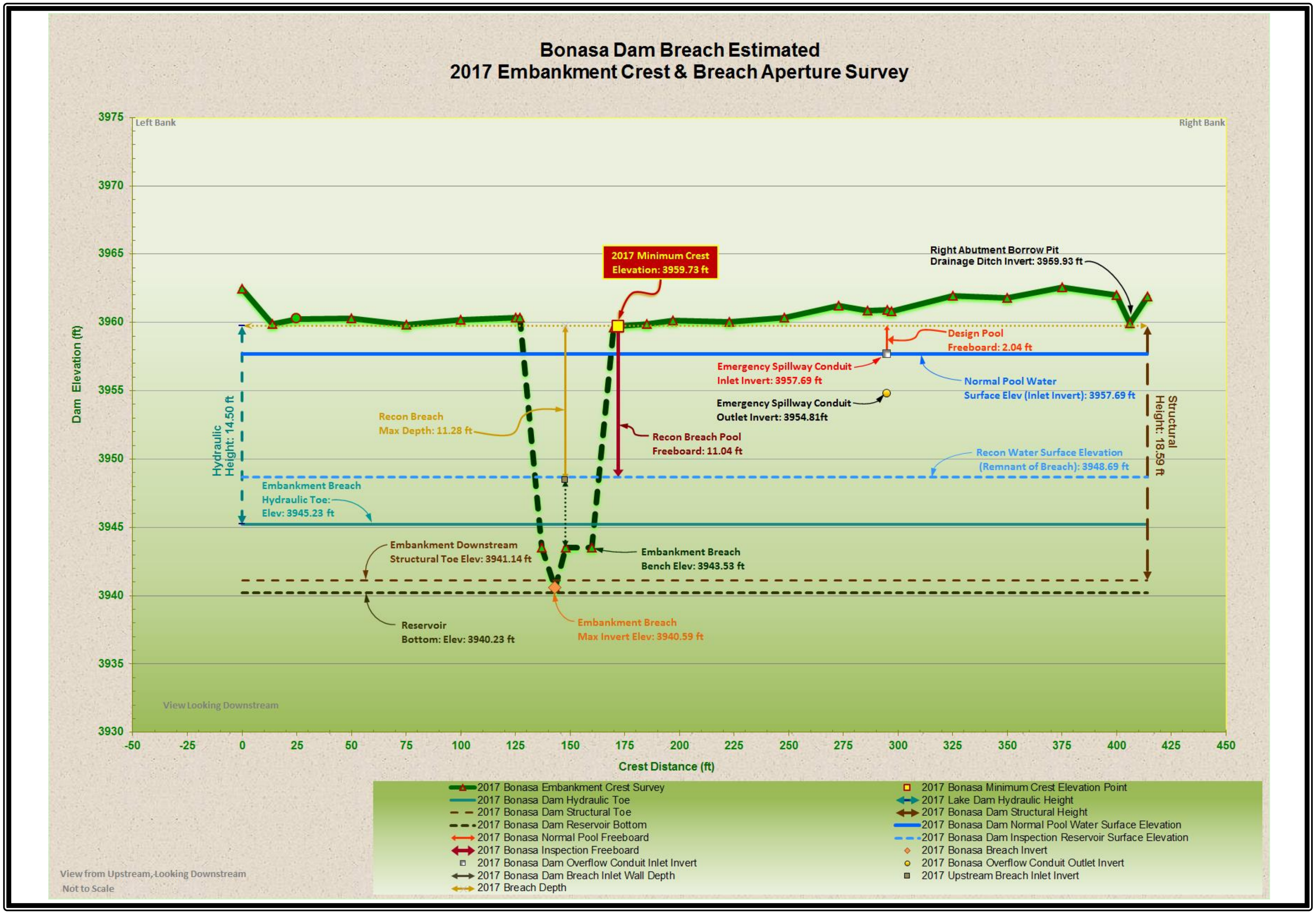

Figure 5. Bonasa Breaks Ranch Dam post breach crest survey

Figure 6. Bonasa Breaks Ranch Dam Inspection: debris line, grasses lying flat on the embankment, and erosion features.

Figure 7. Bonasa Breaks Ranch Dam: soil movement likely induced by rapid drawdown failure on the upstream slope

Figure 8. Bonasa Breaks Ranch breach: cracking on upstream slope along southern embankment section

Figure 9. Bonasa Breaks Ranch downstream toe section. Notice the organic layer below the light brown top layer.

Figure 10. Bonasa Breaks Ranch breach section: Notice the difference in elevation at foundation level.

Figure 11. "Tube-like" feature on southern sidewall of breached section

Figure 12. Downstream side of overflow conduit

Figure 13. Interior of overflow conduit

Figure 14. Downstream toe area at overflow spillway conduit. Notice erosion damage on slope

Figure 15. Channel on right abutment

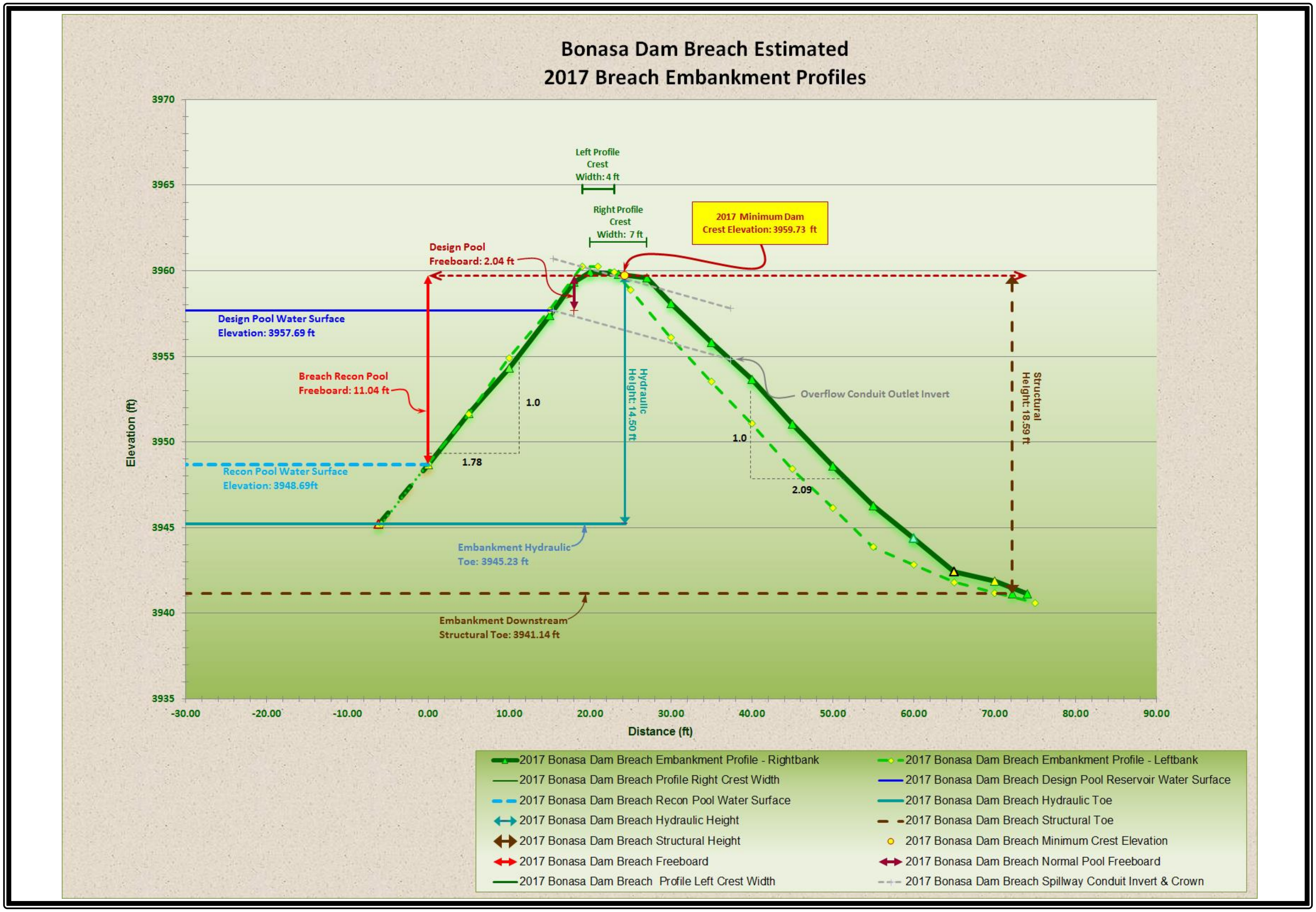

Figure 16. Bonasa Breaks Ranch Dam embankment profile survey (not to scale)

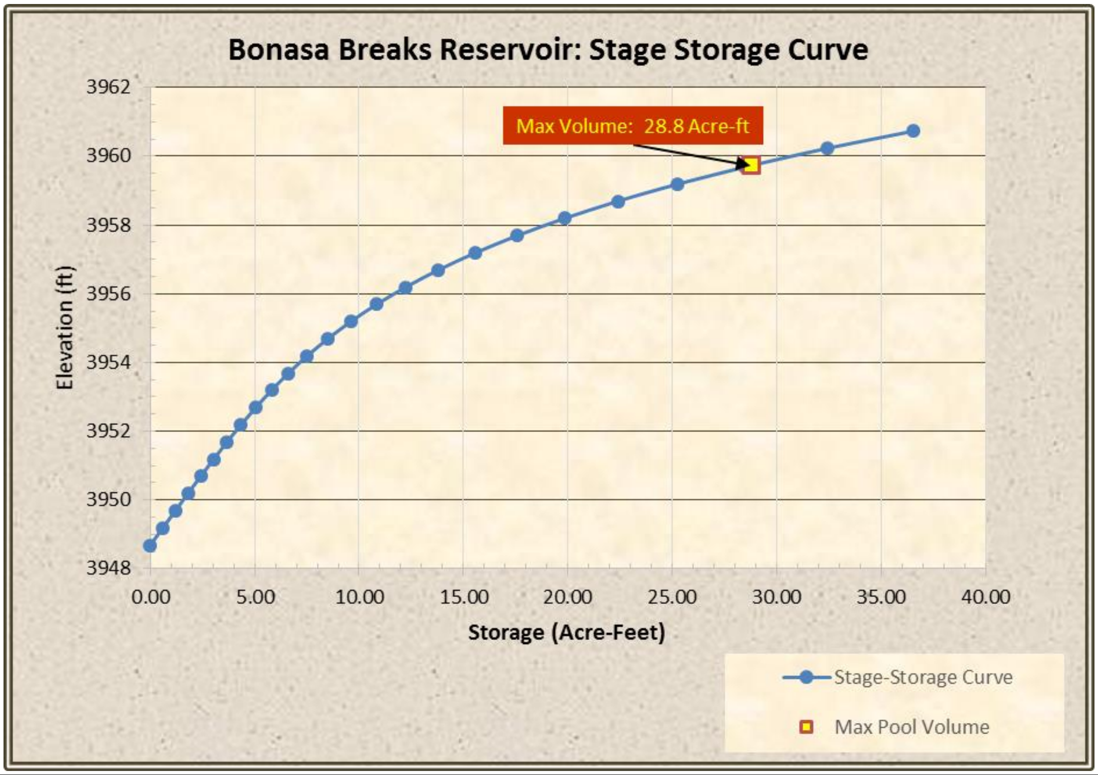

Figure 17. Bonasa Breaks Ranch Dam: Reservoir stage-storage curve with peak volume

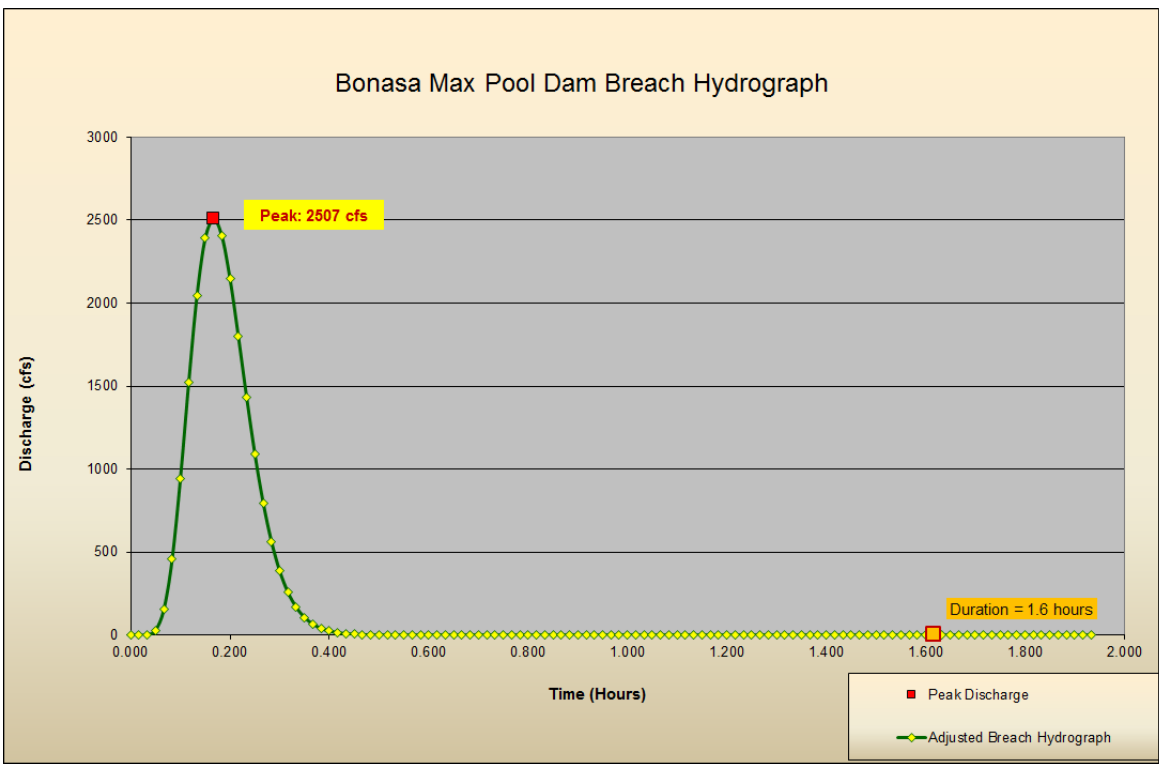

Figure 18. Bonasa Breaks Ranch Dam: Breach hydrograph

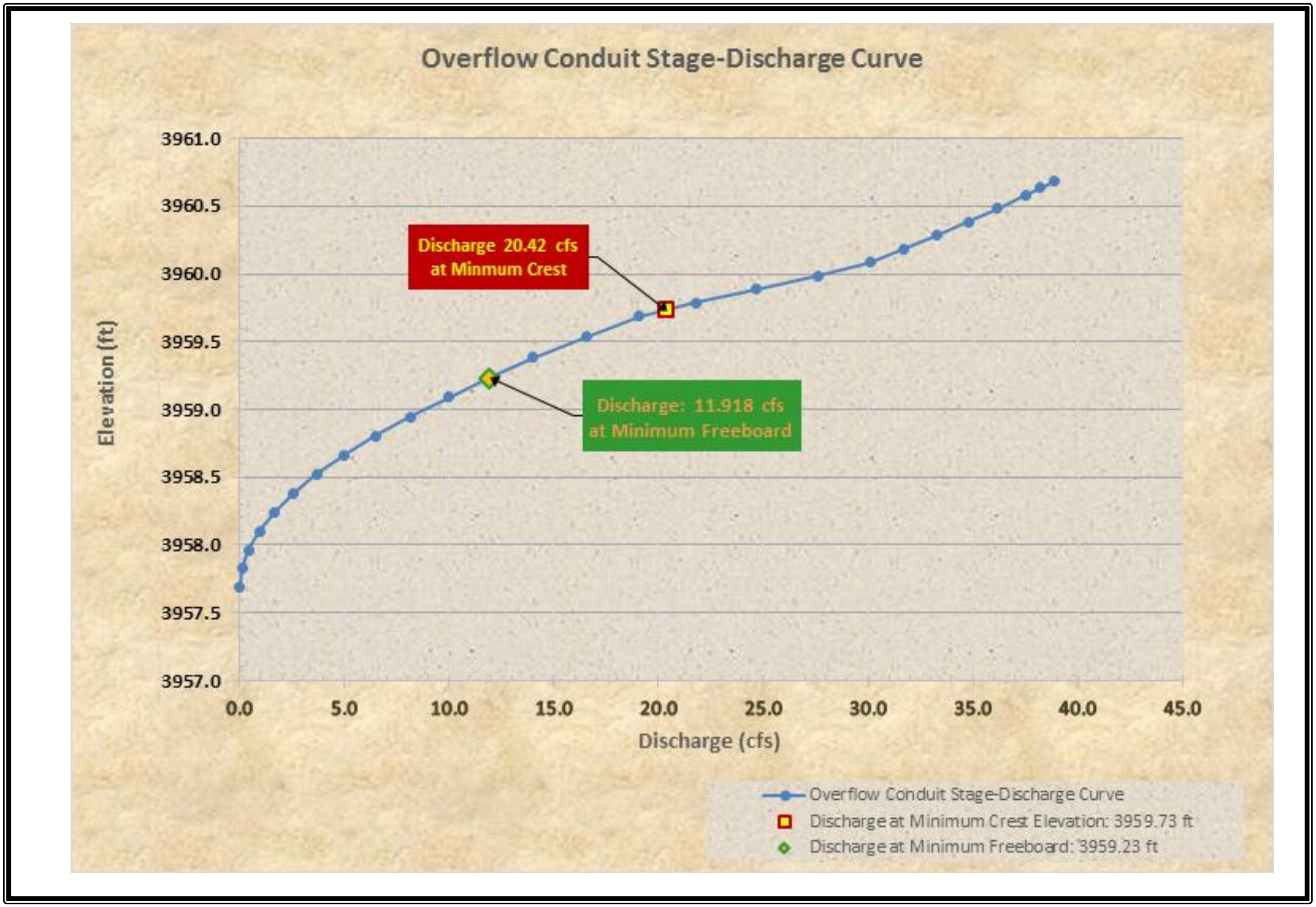

Figure 19. Bonasa Breaks Ranch Dam: Overflow spillway conduit stage discharge curve

Appendix B. Comparison of HEC-HMS Tables

Table 4. Bonasa Breaks Ranch Dam: Step 3 Design Storms - Comparison of the HEC-HMS

Summary Results for Reservoir "Bonasa Impoundment"

Project: Project 4 Simulation Run: ShortStep3Storm-Snow
Reservoir: Bonasa Impoundment

Start of Run: 01May2017, 00:00
End of Run: 02May2017, 00:00
Compute Time: 27Jul2017, 16:07:01

Basin Model: Bonasa Watershed
Meteorologic Model: BonasaStep3Short-Snow
Control Specifications: ControShort

Volume Units: IN AC-FT

Computed Results

Peak Inflow:	213.83 (CFS)	Date/Time of Peak Inflow:	01May2017, 06:45
Peak Discharge:	210.67 (CFS)	Date/Time of Peak Discharge:	01May2017, 07:00
Inflow Volume:	36.40 (AC-FT)	Peak Storage:	30.61 (AC-FT)
Discharge Volume:	32.47 (AC-FT)	Peak Elevation:	3959.95 (FT)

HEC-HMS Step 3 Long Storm - Snow

Crest Minimum Elevation	Reservoir's IDF Peak Elevation	Spillway's Max Discharge At Crest Min	Peak IDF Discharge at Peak Elevation	Sillway's Discharge Deficiency for Required Freeboard Elevation	IDF vs. Crest Elevation Difference	IDF vs. Freeboard Elevation Difference
(ft)	(ft)	(cfs)	(cfs)	(cfs)	(ft)	(ft)
3959.7	3959.8	29.39	57.74	-45.82	0.07	0.57

HEC-HMS Step 3 Long Storm + Snow

Crest Minimum Elevation	Reservoir's IDF Peak Elevation	Spillway's Max Discharge At Crest Min	Peak IDF Discharge at Peak Elevation	Sillway's Discharge Defiiciency for Required Freeboard Elevation	IDF vs. Crest Elevation Difference	IDF vs. Freeboard Elevation Difference
(ft)	(ft)	(cfs)	(cfs)	(cfs)	(ft)	(ft)
3959.7	3959.8	29.39	82.23	-70.31	0.11	0.61

HEC-HMS Step 3 Intermediate Storm - Snow

Crest Minimum Elevation	Reservoir's IDF Peak Elevation	Spillway's Max Discharge At Crest Min	Peak IDF Discharge at Peak Elevation	Sillway's Discharge Defiiciency for Required Freeboard Elevation	IDF vs. Crest Elevation Difference	IDF vs. Freeboard Elevation Difference
(ft)	(ft)	(cfs)	(cfs)	(cfs)	(ft)	(ft)
3959.7	3959.9	29.39	117.01	-105.09	0.15	0.65

HEC-HMS Step 3 Intermediate Storm + Snow

Crest Minimum Elevation	Reservoir's IDF Peak Elevation	Spillway's Max Discharge At Crest Min	Peak IDF Discharge at Peak Elevation	Sillway's Discharge Defiiciency for Required Freeboard Elevation	IDF vs. Crest Elevation Difference	IDF vs. Freeboard Elevation Difference
(ft)	(ft)	(cfs)	(cfs)	(cfs)	(ft)	(ft)
3959.7	3959.9	29.39	143.18	-131.26	0.17	0.67

HEC-HMS Step 3 Short Storm - Snow

Crest Minimum Elevation	Reservoir's IDF Peak Elevation	Spillway's Max Discharge At Crest Min	Peak IDF Discharge at Peak Elevation	Sillway's Discharge Defiiciency for Required Freeboard Elevation	IDF vs. Crest Elevation Difference	IDF vs. Freeboard Elevation Difference
(ft)	(ft)	(cfs)	(cfs)	(cfs)	(ft)	(ft)
3959.7	3960.0	29.39	210.67	-198.75	0.22	0.72

Table 5. Bonasa Breaks Ranch Dam: 25-Year Design Storm – Comparison of the HEC-HMS IDF discharge and the overflow spillway capacity

Summary Results for Reservoir "Bonasa Impoundment"

Project: Project 4 Simulation Run: 25yrLong+Snow
Reservoir: Bonasa Impoundment

Start of Run: 01May2017, 00:00 Basin Model: Bonasa Watershed
End of Run: 04May2017, 00:00 Meteorologic Model: Bonasa25yrLong+Snow
Compute Time: 27Jul2017, 18:00:49 Control Specifications: ControlLong

Volume Units: IN AC-FT

Computed Results

Peak Inflow:	34.75 (CFS)	Date/Time of Peak Inflow:	02May2017, 11:45
Peak Discharge:	15.70 (CFS)	Date/Time of Peak Discharge:	02May2017, 21:15
Inflow Volume:	30.68 (AC-FT)	Peak Storage:	27.66 (AC-FT)
Discharge Volume:	28.67 (AC-FT)	Peak Elevation:	3959.48 (FT)

Summary Results for Reservoir "Bonasa Impoundment"

Project: Project 4 Simulation Run: 25yrInterm+Snow
Reservoir: Bonasa Impoundment

Start of Run: 01May2017, 00:00 Basin Model: Bonasa Watershed
End of Run: 02May2017, 00:00 Meteorologic Model: Bonasa25yrInterm+Snow
Compute Time: 27Jul2017, 18:03:35 Control Specifications: ControlIntermediate

Volume Units: IN AC-FT

Computed Results

Peak Inflow:	60.22 (CFS)	Date/Time of Peak Inflow:	01May2017, 10:45
Peak Discharge:	30.11 (CFS)	Date/Time of Peak Discharge:	01May2017, 14:15
Inflow Volume:	26.69 (AC-FT)	Peak Storage:	29.15 (AC-FT)
Discharge Volume:	20.74 (AC-FT)	Peak Elevation:	3959.73 (FT)

HEC-HMS 25-Year Long Storm + Snow						
Crest Minimum Elevation	Reservoir's IDF Peak Elevation	Spillway's Max Discharge At Crest Min	Peak IDF Discharge at Peak Elevation	Sillway's Discharge Defiiciency for Required Freeboard Elevation	IDF vs. Crest Elevation Difference	IDF vs. Freeboard Elevation Difference
(ft)	(ft)	(cfs)	(cfs)	(cfs)	(ft)	(ft)
3959.7	3959.5	29.39	15.70	-3.78	-0.25	0.25
HEC-HMS 25-Year Intermediate Storm + Snow						
Crest Minimum Elevation	Reservoir's IDF Peak Elevation	Spillway's Max Discharge At Crest Min	Peak IDF Discharge at Peak Elevation	Sillway's Discharge Defiiciency for Required Freeboard Elevation	IDF vs. Crest Elevation Difference	IDF vs. Freeboard Elevation Difference
(ft)	(ft)	(cfs)	(cfs)	(cfs)	(ft)	(ft)
3959.7	3959.7	29.39	30.11	-18.19	0.00	0.50
HEC-HMS 25-Year Short Storm - Snow						
Crest Minimum Elevation	Reservoir's IDF Peak Elevation	Spillway's Max Discharge At Crest Min	Peak IDF Discharge at Peak Elevation	Sillway's Discharge Defiiciency for Required Freeboard Elevation	IDF vs. Crest Elevation Difference	IDF vs. Freeboard Elevation Difference
(ft)	(ft)	(cfs)	(cfs)	(cfs)	(ft)	(ft)
3959.7	3958.9	29.39	7.99	3.93	-0.80	-0.30

Appendix C. Hydrologic Analysis: Supporting Calculations

In recent years, Dam Safety's paper and electronic files have become very integrated such that some documents exist only in electronic form. Consistent with this development, and in the interest of expediting this report, the spreadsheet computations for this hydrologic analysis are not copied here, but are incorporated into this report by reference. Copies of these spreadsheets (either electronic or paper format) are available from the Dam Safety Office.

Spreadsheet calculations were used to develop the input data to a HEC-HMS computer model, with the results from the HEC-HMS model runs copied to other spreadsheets to record them for posterity. The specific spreadsheets used in this hydrologic analysis are listed below. These are all MS Excel 2013 format.

	Spreadsheet file name

Watershed hydrology

Survey	Bonasa Breach Crest & Profile Survey 5-3-2017 #2.xlsm
Breach & Volume Calculations	Bonasa Breach & Volume Analysis #3 1.198AcresBottom m=3.66 5-16-2017.xlsm
Breach Geometry	& Bonasa Breach New Precep & Design Storms Calculations Step 3 Storm with Overtopping.xls
Conduit Calculations	Bonasa Breach New Precep & Design Storms Calculations Step 3 Storm with Overtopping.xls (Plus Step2, Step1, 100-Year & 25-Years Storms)
Soils	Organized Bonasa Surgo Soils Data for Hydrology Analysis.xls
Infiltration and CN computations	Bonasa Breach New Precep & Design Storms Calculations Step 3 Storm with Overtopping.xls

Design storm precipitation	Bonasa Breach New Precep & Design Storms Calculations Step 3 Storm with Overtopping.xls (Plus Step2, Step1, 100-Year & 25-Years Storms)
Snowmelt computations	Bonasa Breach New Precep & Design Storms Calculations Step 3 Storm with Overtopping.xls (Plus Step2, Step1, 100-Year & 25-Years Storms)
Storm, interflow and loss hyetographs	Bonasa Breach New Precep & Design Storms Calculations Step 3 Storm with Overtopping.xls (Plus Step2, Step1, 100-Year & 25-Years Storms