A HYDROLOGIC ENGINEERING ANALYSIS OF A FAILED RANGELAND WATER CONTROL STRUCTURE ON THE BUENOS AIRES NATIONAL WILDLIFE REFUGE

Cameron Dorsett[1]; Donald Slack[2]; Mary Nichols[3]; Kamel Didan[4]

[1]University of Arizona, Biosystems Engineering Graduate

ckdorsett@email.arizona.edu; +1 (979)402.4275

[2]University of Arizona, Department of Biosystems Engineering. 1177 E. 4th St, Shantz Bldg., Rm 627. Tucson, AZ, USA 85721
[3]USDA Agricultural Research Service, Southwest Watershed Research Center. 2000 E. Allen Rd. Tucson, AZ 85719
[4]University of Arizona, Department of Biosystems Engineering. 1177 E. 4th St, Shantz Bldg., Rm 403. Tucson, AZ, USA 85721

Abstract

This report summarizes a hydrologic engineering analysis of a failed concrete drop spillway (broad-crested weir) water control structure at the outlet or "pour point" on the Buenos Aires National Wildlife Refuge (BANWR) watershed southwest of Tucson, AZ. The main objective was to determine the maximum flow the structure could pass without failure (discharge capacity) as well as the rainfall recurrence interval and duration which would result in a flood of a magnitude which would exceed the capacity of the spillway and thus likely lead to failure of the structure. The layout, function, and characterization of the watershed was established using modern software programs and available imagery. The discharge capacity of the concrete drop spillway was found to be 21.1 [m^3s^{-1}]. After evaluating results obtained from the Rational Method and the Curve Number (CN) Method (with assumptions of closed upper-watershed gates and stock pond retention having no effect), the spillway capacity was adequate to withhold runoff volumes generated from 10-yr to 25-yr recurrence interval rainfalls of variable durations and intensities provided the spatial extent of rainfall was limited to one of the two small sub-watersheds (Sub-watersheds A and B). However, if rainfall occurred over the entire watershed or Sub-watershed C then the spillway capacity was exceeded by runoff volumes generated for 10-yr and 25-yr recurrence interval rainfalls of all durations as well as all generated design storms of greater magnitude and intensity.

Keywords: hydraulic structures, design capacity, hydrologic analysis, runoff, curve number

Introduction

The history of land ownership and land use in southeastern Arizona's Altar Valley provides valuable context to the issue of rangeland hydrology, and more specifically, altered runoff pathways. Originally settled in the late 1880s, the area that is presently the U.S. Fish and Wildlife Service (USFWS) Buenos Aires National Wildlife Refuge (BANWR) was once a working cattle ranch that saw several changes in ownership until it was ultimately purchased in 1985 to establish a wildlife reserve, Figure 1. The USFWS subsequently ceased grazing on the range by removing cattle altogether with the aims of restoring floodplain grasslands to increase populations of the masked bobwhite quail (Sayre, 2002). The U.S. Soil Conservation Service (SCS) cost-shared the construction of conservation structures in cooperation with ranch owners at the time, Fred Gill and family (Sayre, 2002). However, there appears to have been other structures that may have been constructed with little or no hydraulic design considerations.

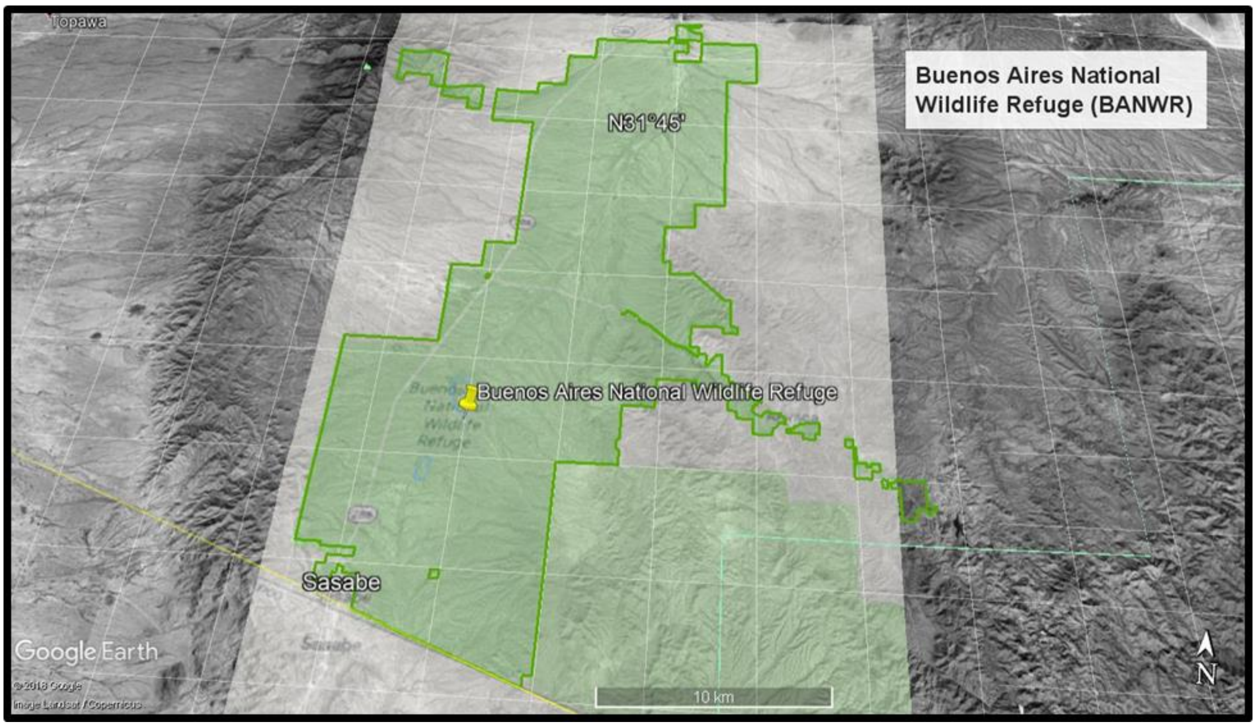

Figure 1. USFWS BANWR boundary (Google Earth Pro, 2018). [31°34'6.90" N 111°30'2.37" W]

Water control structures including earthen spreader berms, concrete drop spillways, drop-board gates, and dirt stock tanks are found to this day on the BANWR though long lacking maintenance and upkeep. When operating as intended, earthen spreader berms would backup or retain floodwaters for increased forage production, concrete drop spillways would serve to divert floodwaters further downstream, drop-board gates could manipulate the direction of floodwater flow, and dirt stock tanks would capture and store floodwater for cattle. Due to either inadequate design and/or a lack of maintenance, many of the structures have failed over the past 60-70 years resulting in increased arroyo downcutting and disconnectedness of the grasslands in the floodplain. The objective of this study was to conduct a hydrologic engineering analysis of one of the concrete drop spillways that had been flanked and structurally compromised to determine the maximum spillway capacity and the storm size that would generate large enough runoff to exceed that capacity. In other words, was the spillway capacity adequate to pass peak runoff resulting from a 25-year return period storm.

Materials and Methods

High-resolution remotely-sensed LiDAR data (flyover in 2015) (Pima Association of Governments, 2018) obtained from Pima County Flood Control District and the Pima Association of Governments was processed in ArcMap 10.5.1 (ESRI, 2017) to create a 1-m digital elevation model (DEM). This DEM was used to delineate the watershed based on the concrete spillway as the outlet or pour point. The resulting 1501ha watershed is shown in Figure 2. Note that the watershed was comprised of three sub-watersheds, A, B and C with areas of 94.2ha, 122ha and 1285ha respectively.

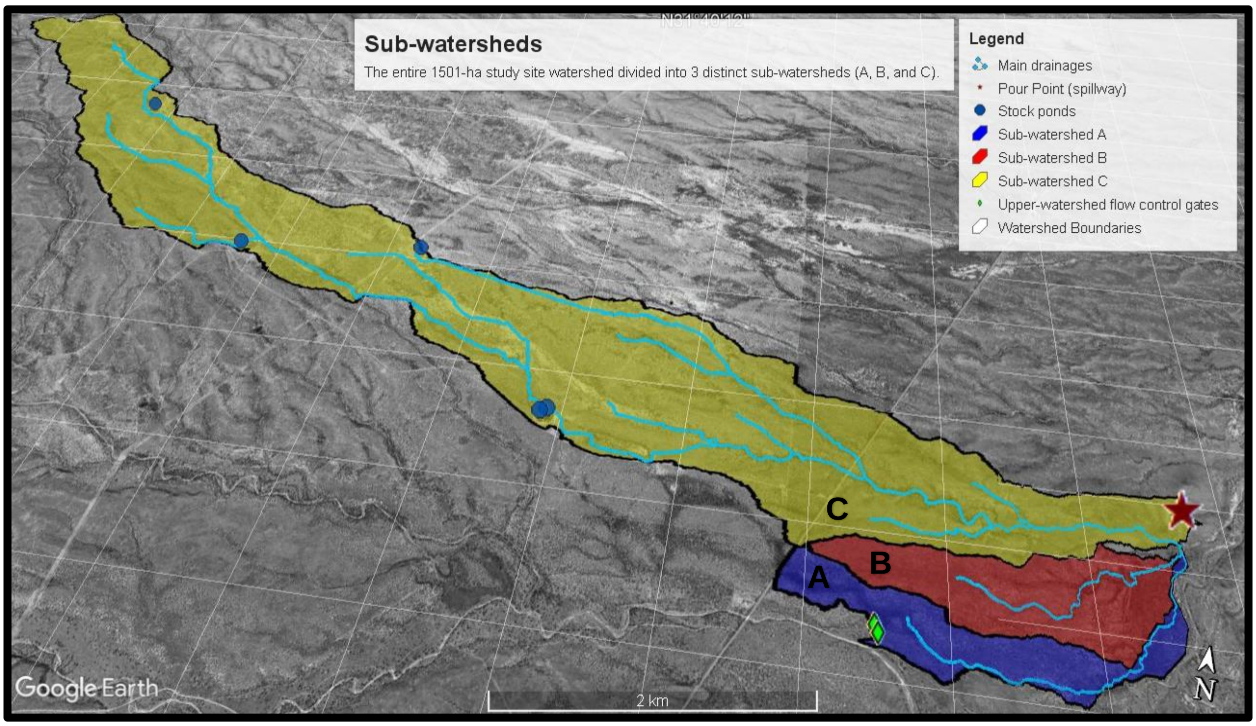

Figure 2. The entire 1501-ha study site watershed divided at each of the main drainages into three sub-watersheds for fine-tuned analysis. Sub-watershed A, 94.2 [ha]; Sub-watershed B, 122 [ha]; Sub-watershed C, 1285 [ha]. [31°36'35.05"N 111°30'36.86"W]

A series of available historic aerial imagery of the study site watershed were obtained through the U.S. Geological Survey Earth Explorer, Google Earth, and the imagery collection at Arizona State University. Ground-level photos were also taken during field visits. The imagery analyzed dates as early as December 1956 and as recent as June 2017. An additional 1936 aerial photograph from the Fairchild Aerial Survey Company and Arizona State University imagery collection was also reviewed and indicated that no spillway or soil berm was present at that time.

Figure 3 shows the lower section of the watershed near the spillway. Site visits were conducted to visually characterize the hydrological network down to the spillway and for dimensional measurements of the spillway. **Figure 4** is a ground level photo of the failed structure clearly showing the "flanking" failure around the right end of the structure.

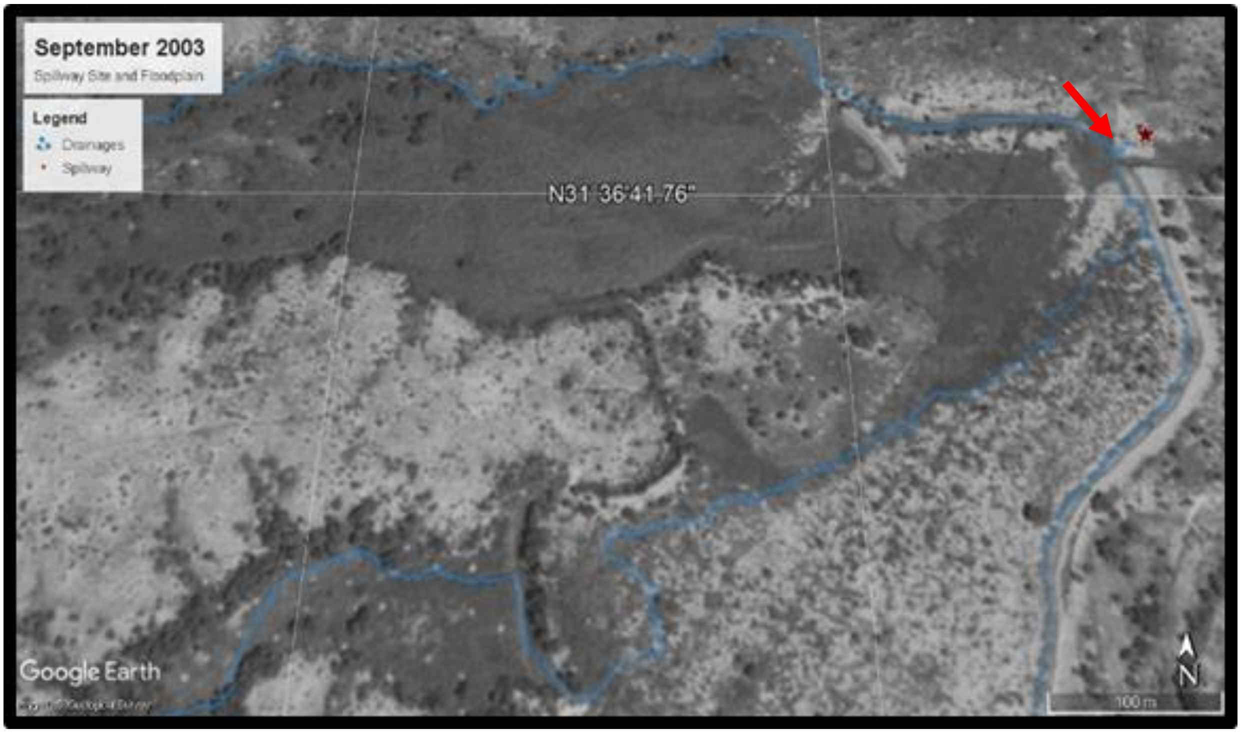

Figure 3. September 2003 image at 100-m scale from Google Earth Pro showing location of the spillway (red arrow). [31°36'43.10"N 111°29'43.30"W]

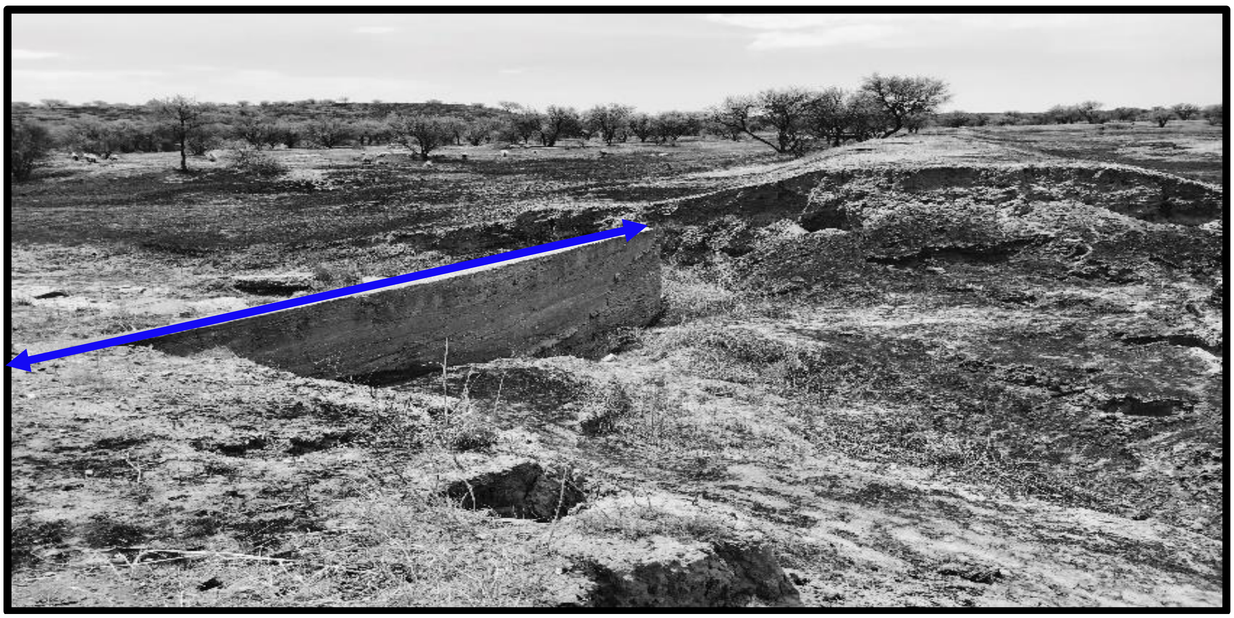

Figure 4. Ground-level image of the failed spillway looking downstream and clearly showing the massive erosion around the right end of the structure. [31°36'43.48"N 111°29'32.49"W]

Measurements of the length of the crest of the spillway (shown in blue), *L*, were taken as well as the hydraulic head, *H* (the difference in elevation between the crest of the spillway and the top of the weir notch and adjacent earthen berm), using a self-leveling level. These measurements were used in the broad-crested weir formula to determine the spillway capacity. The broad-crested weir formula is given as:

$$q = CLH^{3/2} \tag{1}$$

Where q = flow rate through the spillway [m^3s^{-1}], C = weir coefficient [1.70 in this case, L= length of the crest of the weir [m] and H = depth of flow across the weir crest or hydraulic head [m]. The field measurements gave L= 13.4m and the depth of the weir "notch" which would be the maximum head as H =0.95m. Thus, the maximum capacity of the spillway, without overtopping the structure would be q=21.1[m^3s^{-1}].

Peak runoff rates from the watershed were determined by two different methods, the Rational Method and the US-Soil Conservation Service (now NRCS) Curve Number (CN) Method. Peak runoff estimations using the Rational Method required inputs of rainfall intensity [mm/hr] and storm duration. Design storms generated using the Curve Number Method required inputs of rainfall depth [mm]. These data were acquired through the National Oceanic and Atmospheric Administration (NOAA) National Weather Service (NWS) Hydrometeorological Design Studies Center Precipitation Frequency Data Server (PFDS) (NOAA,2018).

The peak flow rate determined by the Rational Method is calculated as follows:

$$q_p = \frac{CiA}{360} \tag{2}$$

Where: q_p = peak runoff [m^3s^{-1}], C = runoff coefficient, i = rainfall intensity [mm/hr] and A = drainage area [ha].

Runoff coefficients were estimated based on soil type, basin slope, and percent area composition within the watershed. Soil types and percent area composition were determined by creating an Area of Interest (AOI) by uploading the sub-watershed shapefiles (.shp) into the USDA-NRCS Web Soil Survey (WSS) (United States Department of Agriculture-Natural Resources Conservation Service) (NRCS, USDA, 2018). A single runoff coefficient weighted according to the percentage of the watershed area comprised of each soil type was then calculated for each of the sub watersheds.

The Rational Method stipulates that rainfall intensity, i, occurs for a duration equal to the time of concentration, t_c for the watershed. The Kirpich formula (Haan, et al. 1994) was used to calculate the time of concentration for each sub watershed. The Kirpich formulas is expressed as:

$$t_c = 0.0195\mathrm{L}^{0.77}\left(\frac{\mathrm{L}}{H}\right)^{0.385} \tag{3}$$

Where: t_c = time of concentration [min], *L* = max length of flow [m], *H* = difference in elevation between the outlet of the watershed and hydraulically most remote point in watershed [m]. *L* was determined for each sub watershed using ArcMap Attribute Tables to compute flow path length, while *H* was computed based on elevation profiles between the spillway outlet and the top of Drainage A using the 'path tool' in Google Earth Pro.

After calculating the time of concentration, corresponding rainfall intensity tables were generated for each sub watershed for return periods of 10 and 25 years based on information found in the NOAA NWS Hydrometeorological Design Studies Center PFDS (NOAA, 2018). This information was then used in **equation 2** to calculate peak flow rates for each sub watershed for return periods of 10 and 25 years. Results are presented in the results and discussion section.

A second analysis was undertaken using a rainfall-runoff hydrograph model, Wildcat 5, developed by Hawkins and Barreto-Munoz (2014) and supported by the US Forest Service. This software is an add-on package for Microsoft Excel and assists watershed analysts in predicting peak flow and runoff volumes from single-event rainstorms for a variety of conditions. Model inputs include storm characteristics (depth, duration, distribution), watershed soil and cover parameters for runoff depth, runoff timing parameters to the outlet, and unit hydrograph shape and scale (Richard H. Hawkins; Armando Barreto-Munoz | United States Dept. of Agriculture | U.S. Forest Service, 2016). The software generates runoff volumes for a given watershed utilizing the SCS Curve Number method and then uses storm characteristics and scaled unit hydrographs to develop storm hydrographs.

The Curve Number Approach is the basis for the estimation of runoff volume and the generation of the hydrograph:

$$Q = \frac{(P-0.2S)^2}{P+0.8S}, \mathrm{P} > 0.2S \tag{4}$$

Where: *Q* = runoff depth/volume [mm], *P* = precipitation depth [mm] and *S* = soil water retention parameter [mm]. The soil water retention parameter is based on the Curve Number (CN) and is found using:

$$S = \frac{25400}{CN} - 254 \ (\mathrm{Q, P, S\ [mm]}) \tag{5}$$

Curve Numbers were selected by consulting Table 3.2d in Appendix 3C (Haan et al., 1994) under 'Cover Type' of 'Desert Shrub—major plants include: saltbush, greasewood, creosote bush, black brush, bursage, palo verde, mesquite, and cactus', 'Hydrologic Condition' of 'Fair—30 to 70% ground cover', and 'Hydrologic Soil Group' of Soil Type C. A single curve number, weighted according to the percentage of the watershed area comprised of each soil type, was then calculated for each of the sub watersheds in the same way as the runoff coefficients were weighted for the Rational Method.

The watershed lag time method provides another means of finding the time of concentration. Lag time is defined as the delay between when runoff from an event begins and when runoff peaks and can be considered a weighted form of time of concentration. This generally yields the relationship of lag time equaling 60% the time of concentration though this is not always the case (Simas & Hawkins, 1996; USDA NRCS, 2010).
Lag time and time of concentration were determined in Wildcat5 via:

$$t_{lag} = 0.0051 \times width^{0.594} \times slope^{-0.150} \times S_{nat}^{0.313} \tag{6}$$

Where: t_{lag} = lag time [hr], *width* = watershed area divided by watershed length [ft, m], *slope* = average land slope [decimal fraction or %], S_{nat} = soil water retention parameter [in, mm] and:

$$t_c = 0.0085 \times W^{0.5937} \times S^{-0.1505} \times S_{nat}^{0.3131} \tag{7}$$

Where: t_c = time of concentration [hr], W = watershed area divided by watershed length [ft, m], S = average land slope [ft/ft, m/m], and S_{nat} = soil water retention parameter [in, mm].

The SCS Dimensionless Curvilinear unit hydrograph was utilized in the software to generate the storm hydrographs. Storm *inputs* (duration and rainfall) were split into two categories: “flash floods” and “floods.” The “flash flood” storms were limited to durations of 2 and 6-hours while the “flood” storms had durations of 12 and 24-hours (NWS NOAA, 2019). The selected recurrence intervals for the analysis were 10, 25, 60, and 80-year storms for each category. The Wildcat5 software utilizes these inputs to generate an SCS Dimensionless Curvilinear hydrograph for each situation.

A small reservoir with an average surface area of 0.427ha existed above the spillway and to properly determine the adequacy of the spillway capacity, it is necessary to “route” the “design” storms through the reservoir. Wildcat5 has a routing routine which utilizes the average reservoir area, spillway length, weir coefficient and depth of the weir notch. **Figure 5** shows an outline of the reservoir upstream of the structure.

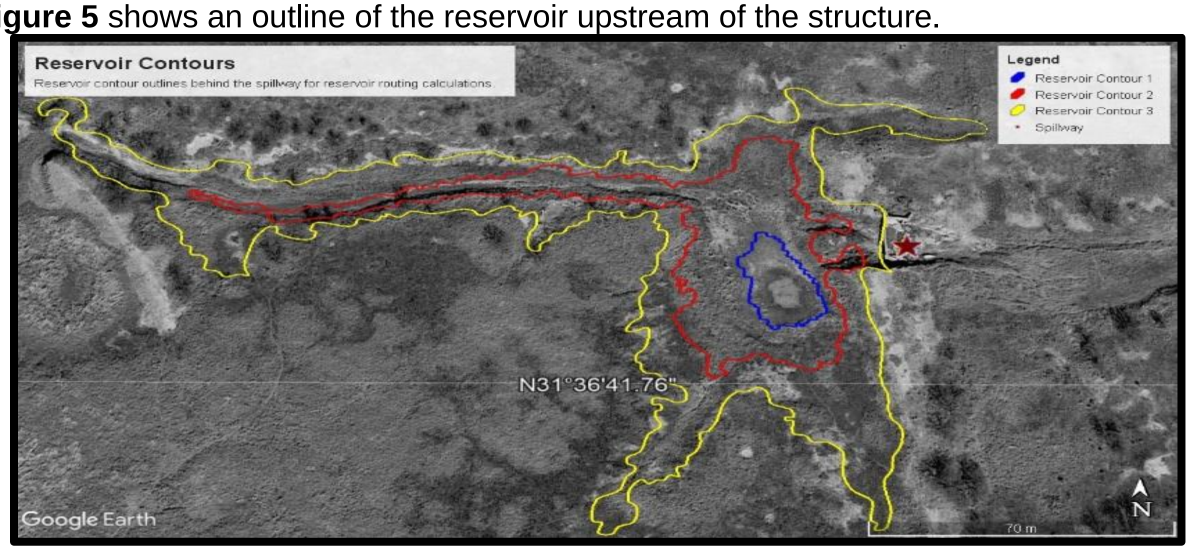

Figure 5. Reservoir contours behind spillway generated in Google Earth Pro. Average area calculated to be 0.427 [ha]. [31°36'42.71"N 111°29'36.57"W]

Results and Discussion

Results are presented in the following. Within these tables, values that fall below or within the spillway capacity limit are indicated in black standard font. Those flowrates which exceed the spillway capacity are shown in **red bolded font**.

Spillway Discharge Capacity

The spillway weir coefficient, *C*, was calculated as 1.70. The spillway length, *L*, and hydraulic head, *H*, were measured as 13.4 [m] and 0.95 [m], respectively. These variables yielded a spillway carrying capacity, *Q*, of 21.1 [m^3s^{-1}] using **equation 1**.

Peak Runoff Estimations (Rational Method)

It should be re-emphasized that the Rational Method determines only peak flowrates with no corresponding hydrograph. Therefore, the flows calculated in this manner cannot be routed through the reservoir and there would thus be no diminution of flood peak due to reservoir storage.

Sub-watershed A

The runoff coefficient was calculated to be 0.268. The final precipitation intensities [mm/hr], *i*, were utilized for the calculated time of concentration, t_c, of 65.1-min (1.09-hr) for Sub-watershed A (**Table 1**). Thus, the storm duration was 65.1 minutes. Total watershed area was 94.2 [ha]. The obtained variables were used with **equation 2** and yielded the peak runoff results shown in **Table 1**.

Table 1. Precipitation Intensity (mm/hr) and corresponding peak runoff rates (m^3s^{-1}) for Sub-watershed A | Latitude: 31.6042°, Longitude: -111.5129° | Elevation (USGS): 1063.3 m

	Storm Recurrence Interval [yr]					
	10	**25**	**50**	**60**	**80**	**100**
Intensity (mm/hr)	46.2	54.9	61.6	63.0	65.7	68.5
Peak runoff (m^3s^{-1})	3.10	3.68	4.13	4.23	4.41	4.59

Sub-watershed B

The runoff coefficient was calculated to be 0.296. The final precipitation intensities [mm/hr], *i*, were utilized for the calculated time of concentration, t_c, of 46.5-min (0.77-hr) for Sub-watershed B (**Table 2**). Thus, the storm duration was 46.5 min. Total watershed area was 122 [ha]. The obtained variables were used with **equation 2** and yielded the peak runoff results shown in **Table 2.**

Table 2. Precipitation Intensity Estimates (mm/hr) and corresponding peak runoff rates (m^3s^{-1}) for Sub-watershed B | Latitude: 31.6074°, Longitude: -111.5094° | Elevation (USGS): 1064.3 m

	Storm Recurrence Interval [yr]					
	10	**25**	**50**	**60**	**80**	**100**

Intensity (mm/hr)	61.1	72.8	81.6	83.2	86.5	89.8
Peak runoff (m^3s^{-1})	6.11	7.29	8.17	8.33	8.66	8.99

Sub-watershed C

The runoff coefficient was calculated to be 0.312. The final precipitation intensities [mm/hr], *i*, were utilized for the calculated time of concentration, t_c, of 176.9-min (2.95-hr) for Sub-watershed C (**table 3**). Thus, the storm duration was 176.9-min. Total watershed area was 1285 [ha]. The obtained variables were used with **equation 2** and yielded the peak runoff results shown in **table 3**.

Table 3. Precipitation Intensity Estimates (mm/hr) and corresponding peak runoff rates (m^3s^{-1}) for Sub-watershed C | Latitude: 31.6596°, Longitude: -111.6144° | Elevation (USGS): 1232.3 m

	Storm Recurrence Interval [yr]					
	10	**25**	**50**	**60**	**80**	**100**
Intensity (mm/hr)	20.5	24.6	27.7	28.5	30.1	31.7
Peak runoff (m^3s^{-1})	**22.8**	**27.4**	**30.9**	**31.8**	**33.6**	**35.4**

Entire 1501-ha Watershed

The runoff coefficient was calculated to be 0.309. The final precipitation intensities [mm/hr], *i*, were utilized for the calculated time of concentration, t_c, of 176.9-min (2.95-hr) for the Entire 1501-ha Watershed (**Table 4**). Thus, the storm duration was 176.9-min. Total watershed area was 1501 [ha]. The obtained variables were used with **equation 2** and yielded the peak runoff results shown in **Table 4**.

Table 4. Precipitation Intensity Estimates (mm/hr) and corresponding peak runoff rates (m^3s^{-1}) for Entire 1501-ha Watershed | Latitude: 31.6596°, Longitude: -111.6144° | Elevation (USGS): 1232.3 m

	Storm Recurrence Interval [yr]					
	10	**25**	**50**	**60**	**80**	**100**
Intensity (mm/hr)	20.5	24.6	27.7	28.5	30.1	31.7
Peak runoff (m^3s^{-1})	**26.4**	**31.6**	**35.6**	**36.7**	**38.8**	**40.8**

Design Storms via Wildcat5 (Curve Number Method)

Although we modeled the runoff and developed hydrographs for storms of 10, 25, 60 and 80 year return periods and durations of 2, 6, 12 and 24 hours for all four watersheds (A,B,C and total), it was obvious from the results of our Rational Method Analysis that these combinations would not produce runoff which exceeded spillway capacities except for the case of sub watershed C and the total watershed. Therefore, we are only presenting results of those storms on those two watersheds including routing through the reservoir.

Sub-watershed C

As noted, the total watershed area was 1285 [ha] with 80.6% Type C soils and 19.4% Type D soils. This provided a weighted CN of 82.0 (AMC II) for the watershed. Watershed width (area/length) was a calculated 980.9 [m] and the average land slope was determined to be 2.2%. These inputs yielded a calculated SIMAS lag time of 1.40 [hr]. The precipitation depth values for design storm hydrograph generation are shown in **Table 5**.

Table 5. Precipitation Depth Estimates (mm) for Sub-watershed C | Latitude: 31.6596°, Longitude: -111.6144° | Elevation (USGS): 1232.3 m

Storm Duration [hr]	Storm Recurrence Interval [yr]			
	10	25	60	80
2	59	71	81.8	85.4
6	67	81	94.2	98.6
12	77	92	107.4	112.2
24	86	102	116.4	121.2

All obtained variables noted above were used with **equation 4** and **equation 5** as the basis and yielded the peak flows listed in **Table 6**.

Table 6. Design Storm Peak Flows (m^3s^{-1}) for Sub-watershed C | Latitude: 31.6596°, Longitude: -111.6144° | Elevation (USGS): 1232.3 m

Storm Duration [hr]	Storm Recurrence Interval [yr]			
	10	25	60	80
2	**104.8**	**153.6**	**200.7**	**217.0**
6	**74.5**	**105.4**	**136.0**	**146.3**
12	**56.8**	**76.1**	**96.4**	**102.8**
24	**35.7**	**46.5**	**56.4**	**59.7**

The hydrograph for the 10-yr, 24-hr "flood" and the corresponding reservoir routing hydrograph for the same storm are shown in **Figure 6**.

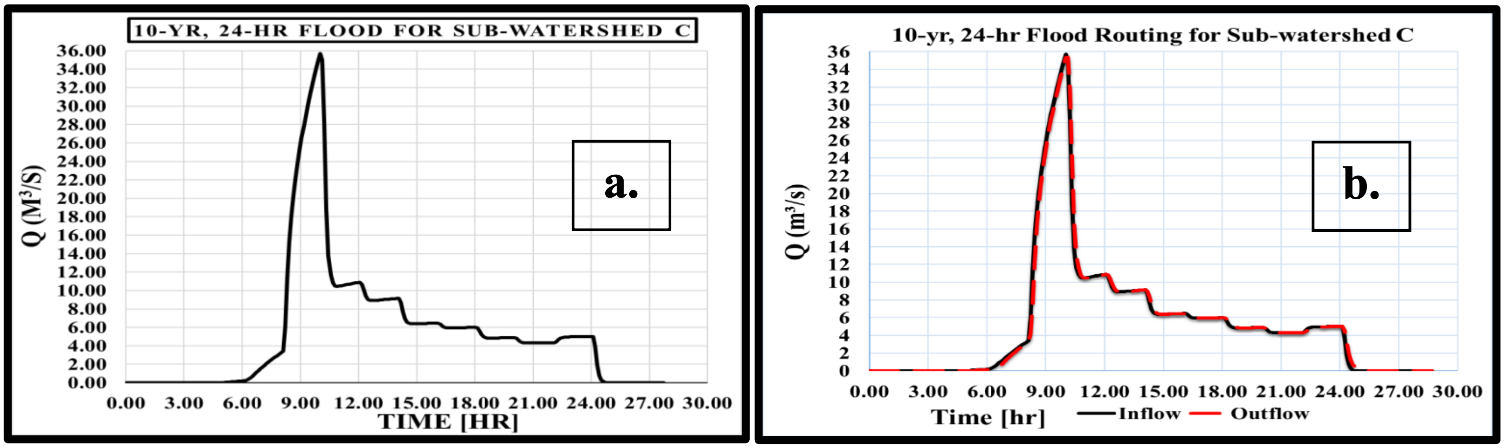

Figure 6. Design storm hydrographs producing peak "flood"-flow at spillway capacity threshold/limit a) 10-yr, 24-hr (35.7 [m^3s^{-1}]) and b) routed through reservoir (35.4 [m^3s^{-1}]) over Sub-watershed C.

Entire 1501-ha Watershed

As noted, the total watershed area was 1501 [ha] with 83.4% Type C soils and 16.6% Type D soils. This provided a weighted CN of 81.8 (AMC II) for the watershed. Watershed width (area/length) was a calculated 1145.8 [m] and the average land slope was determined to be 2.2%. These inputs yielded a calculated SIMAS lag time of 1.54 [hr]. The precipitation depth values for design storm hydrograph generation are shown in **Table 7**.

Table 7. Precipitation Depth Estimates (mm) for Entire 1501-ha Watershed | Latitude: 31.6596°, Longitude: -111.6144° | Elevation (USGS): 1232.3 m

Storm Duration [hr]	Storm Recurrence Interval [yr]			
	10	25	60	80
2	59	71	81.8	85.4
6	67	81	94.2	98.6
12	77	92	107.4	112.2
24	86	102	116.4	121.2

All obtained variables noted above were used with **equation 4** and **equation 5** as the basis and yielded the peak flows listed in **Table 8**.

Table 8. Design Storm Peak Flows (m^3s^{-1}) for Entire 1501-ha Watershed | Latitude: 31.6596°, Longitude: -111.6144° | Elevation (USGS): 1232.3 m

Storm Duration [hr]	Storm Recurrence Interval [yr]			
	10	25	60	80
2	118.1	174.8	229.7	248.6
6	83.4	120.8	158.1	170.9
12	64.5	87.2	111.3	118.9
24	42.0	54.6	66.2	70.0

The hydrograph for the 10-yr, 24-hr "flood" and the corresponding reservoir routing hydrograph for the same storm are shown in **Figure 7.**

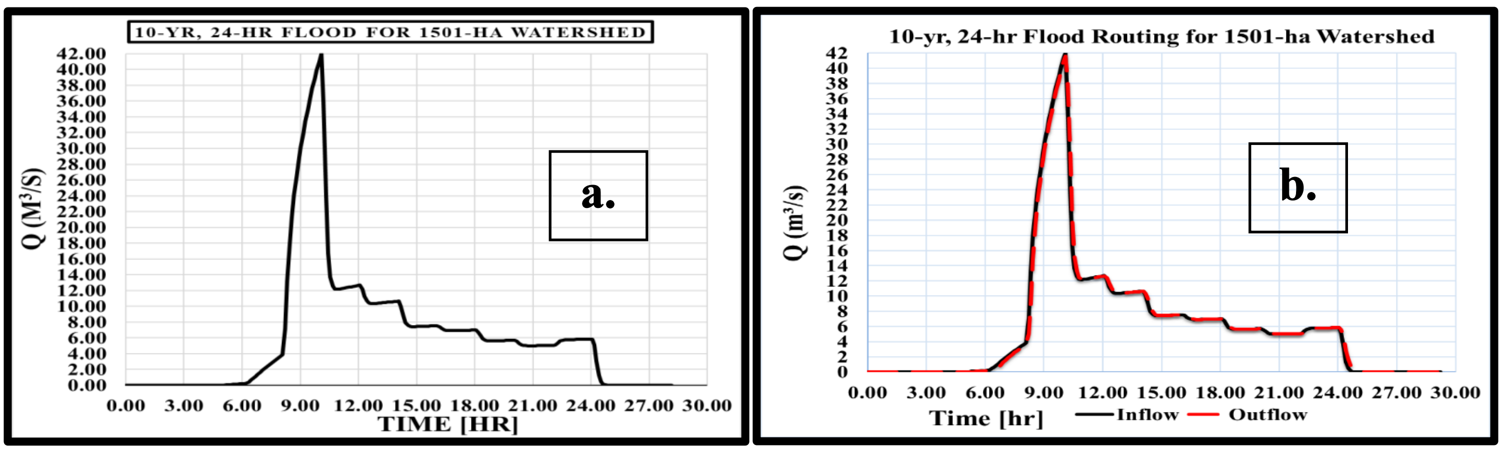

Figure 7. Design storm hydrographs producing peak "flood"-flow at spillway capacity threshold/limit a) 10-yr, 24-hr (42.0 [m^3s^{-1}]) and b) routed through reservoir (41.6 [m^3s^{-1}]) over Entire 1501-ha Watershed.

Conclusions

A standard weir formula was applied to calculate the discharge capacity of the concrete drop spillway as 21.1 [m^3s^{-1}]. Based on obtained results from the Rational Method, the spillway was of adequate capacity to accommodate peak runoff flows generated within either Sub-watershed A (94.2-ha) or Sub-watershed B (122-ha). Peak runoff rates at the confluence of these two watersheds were also below the spillway capacity limit. However, runoff from Sub-watershed C (1285-ha) associated with a 10-yr return period rainfall event with a duration of 177-min (2.95-hr) yielded a peak discharge of 22.8 [m^3s^{-1}] which exceeds the capacity of the spillway. A 25-yr return period rainfall of the same duration yielded a 27.4 [m^3s^{-1}] peak discharge. After performing the same calculations over the entire 1501-ha study site watershed, a 10-yr return period rainfall with a duration of 177-min (2.95-hr) was of adequate magnitude to generate a runoff that exceeds the capacity of the spillway at a peak discharge of 26.4 [m^3s^{-1}].

These results were complimented by calculations and design storms generated using the Wildcat5 software program and the CN Method which determined that Sub-watersheds A and B did not yield peak runoff rates exceeding the capacity of the spillway singularly

In evaluating Sub-watershed C, a 10-yr return period rainfall of 24-hr duration after reservoir routing yields a peak runoff flowrate of 35.4 [m^3s^{-1}], which falls well above the spillway capacity limit. Lastly, the entire 1501-ha Watershed was evaluated. A 10-yr return period rainfall of 24-hr duration was sufficient to generate a peak flood of 42.0 [m^3s^{-1}]. After routing this storm through the reservoir located upstream and adjacent to the spillway, the discharge through the spillway was calculated to be 41.6 [m^3s^{-1}]—still well above the spillway capacity.

In conclusion, based on the Rational Method and the CN Method, the spillway was of adequate capacity to accommodate runoff volumes generated during 10-yr to 25-yr recurrence interval rainfalls of variable durations and intensities provided the spatial extent of rainfall was limited to one of the two smaller sub-watersheds (Sub-watersheds A and B). However, if the rainfall occurred over the entire 1501-ha watershed or Sub-watershed C, then the spillway capacity was exceeded for runoff volumes generated for 10-yr or 25-yr recurrence interval rainfalls of all durations examined and evaluated.

Bibliographic References

ASABE. (2016, November 22). *ASABE Guide for Authors.* Retrieved from asabe.org: https://www.asabe.org/Portals/0/aPubs/GuideForAuthors.pdf?ver=2018-05-09-160038-000

Branson, F. A., Gifford, G. F., Renard, K. G., & Hadley, R. F. (1981). *Rangeland Hydrology.* Dubuque, Iowa: Kendall/Hunt Publishing Company.

Environmental Systems Research Institute. (2017). ArcGIS 10.5.1. Redlands, CA: esri.

Google Earth Pro. (2018, November 29). *Google Earth.* Retrieved September 7, 2017, from Google Earth: https://www.google.com/earth/

Google Maps. (2018, December 19). *Buenos Aires National Wildlife Refuge Admin Building And Visitor Contact Station*. Retrieved from Google Maps: https://www.google.com/maps/place/Buenos+Aires+National+Wildlife+Refuge+Admin+Building+And+Visitor+Contact+Station/@31.5719799,-111.5058042,15z/data=!4m5!3m4!1s0x0:0x2a372d18e0062418!8m2!3d31.5720017!4d-111.5057966

Haan, C., Barfield, B., & Hayes, J. (1994). *Design Hydrology and Sedimentology for Small Catchments.* Amsterdam: Academic Press.

Hawkins, R. H., & Barreto-Munoz, A. (2014, April). Wildcat 5 For Windows | Rainfall-Runoff Hydrograph Model. The University of Arizona | Tucson, Arizona, United States.

Leavengood, B. (2006). IN THE LAND OF GOOD WINDS: An Informal History of the Buenos Aires Ranch. *Journal of Arizona History*, 1-30.

National Oceanic and Atmospheric Administration. (2018). *NOAA's National Weather Service | Hydrometeorological Design Studies Center | Precipitation Frequency Data Server (PFDS)*. Retrieved 2017, from https://hdsc.nws.noaa.gov/hdsc/pfds/

NRCS, USDA. (2018, 8 21). *Web Soil Survey | Home Page*. Retrieved 2017, from Web Soil Survey: https://websoilsurvey.sc.egov.usda.gov/App/HomePage.htm

NWS NOAA. (2019, February). *Flood and flash flood definitions*. Retrieved from weather.gov: https://www.weather.gov/mrx/flood_and_flash

Osborn, H. B., & Simanton, J. R. (1989). Gullies and Sediment Yield. *Rangelands, 11*(2), 51-56.

Pima Association of Governments. (2018). *GIS Data and Maps*. Retrieved November 29, 2018, from https://www.pagregion.com/Programs/TechnicalServices/GISDataandMaps/tabid/84/Default.aspx#LiDAR

Rango, A., Tartowski, S. L., Laliberte, A., Wainwright, J., & Parsons, A. (2006). Islands of hydrologically enhanced biotic productivity in natural and managed arid ecosystems. *Journal of Arid Environments, 65*(2), 235-252.

Richard H. Hawkins; Armando Barreto-Munoz | United States Dept. of Agriculture | U.S. Forest Service. (2016, April). *Wildcat5 for Windows, A Rainfall-Runoff Hydrograph Model: User Manual and Documentation.* Retrieved from fs.fed.us: https://www.fs.fed.us/biology/nsaec/assets/rmrs-gtr-334-508compliant.pdf

San Diego State University (SDSU). (2019, January 29). *The Kirpich formula calculator.* Retrieved from sdsu.edu: http://onlinetc.sdsu.edu/kirpich.php

San Diego State University. (2019, February 4). *Discharge over a broad-crested weir.* Retrieved from onlinechannel14: http://onlinecalc.sdsu.edu/onlinechannel14.php

Sayre, N. F. (2002). *Ranching, Endangered Species, and Urbanization in the Southwest: Species of Capital.* Tucson: The University of Arizona Press.

Simas, M., & Hawkins, R. (1996). *Lag time characteristics for small watersheds in the U.S.* Retrieved from NRCS USDA: https://www.nrcs.usda.gov/Internet/FSE_DOCUMENTS/16/stelprdb1044199.pdf

Smart, P. (1995, August 31). *HydroCAD | Understanding Hydrology*. Retrieved from A Comparison of Hydrologic Methods: https://www.hydrocad.net/understanding.htm

Stromberg, J. C., Tiller, R., & Richter, B. (1996). Effects of Groundwater Decline on Riparian Vegetation of Semiarid Regions: The San Pedro, Arizona. *Ecological Applications, 6*(1), 113-131.

TeachuGis. (2017, September). *TeachuGis: Basics on watershed delineation (playlist, 4 videos).* Retrieved from YouTube: https://www.youtube.com/playlist?list=PLmZEgym3CibX6gp3IqJN2y2DUyiEAMHWv

Texas Department of Transportation (TxDOT). (2019, January 29). *4. Hydrology | Hydraulic Design Manual | Section 11: Time of Concentration.* Retrieved from onlinemanuals.txdot.gov: http://onlinemanuals.txdot.gov/txdotmanuals/hyd/time_of_concentration.htm

Texas Department of Transportation (TxDOT). (2019, January 25). *4. Hydrology | Hydraulic Design Manual | Section 12: Rational Method.* Retrieved from onlinemanuals.txdot.gov: http://onlinemanuals.txdot.gov/txdotmanuals/hyd/rational_method.htm

Thompson, D. B. (2006, September 20). *The Rational Method*.* Retrieved from drdbthompson.net: http://drdbthompson.net/writings/rational.pdf

United States Geological Survey. (1998, March 16). *USGS Aerial Camera Specifications.* Retrieved November 29, 2018, from https://pubs.usgs.gov/tm/11/b5/downloads/2012AerialMappingCameraSpecifications.pdf

United States Geological Survey. (2018, November 29). *Earth Explorer.* Retrieved June 13, 2017, from https://earthexplorer.usgs.gov/

USDA NRCS. (2010, May). *Part 630 Hydrology | National Engineering Handbook.* Retrieved from Chapter 15 | Time of Concentration: https://directives.sc.egov.usda.gov/OpenNonWebContent.aspx?content=27002.wba

Washington State University. (2019, February 13). *Irrigation in the Pacific Northwest | Rectangular Contracted Weir.* Retrieved from irrigation.wsu.edu: http://irrigation.wsu.edu/Content/Calculators/Water-Measurements/Rectangular-Contracted-Weir.php

Webb, R. H., & Leake, S. A. (2006). Ground-water surface-water interactions and long-term change in riverine riparian vegetation in the southwestern United States. *Journal of Hydrology, 320*(3-4), 302-323.