Investigation Report – McGregor Dam

Williams County, North Dakota

SWC Project #528
July 2018

North Dakota
State Water Commission

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McGregor, Williams County, North Dakota

SWC Project #528
North Dakota State Water Commission
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Prepared for:
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1. Introduction

The condition and hydraulic capacity of the principal and emergency spillway systems are particular concerns for McGregor Dam according to previous investigation and inspection reports (Casteel, 2009; ND Office of State Engineer, 2015). As a result, the North Dakota State Water Commission (SWC) and the North Dakota Game and Fish Department (G&F) entered into an investigation agreement (Appendix A) to define the problems and investigate alternatives to resolve them and meet safety standards.

2. Background

2.1 Location and Basin Description

McGregor Dam is located in Section 22, Township 159 North, Range 95 West in northeastern Williams County (Figure 1). It is 0.6 mile upstream of the town of McGregor, North Dakota on a tributary of White Earth Creek. The watershed area is 7.96 square miles and is mainly crop land (Figure 2). The major soil type is Williams–Bowbells Loam, SCS soil group B with a medium-high infiltration rate (NRCS, 2016). Annual precipitation is 15.2 inches (NDAWN, 2016). McGregor Dam is 45 feet in height (Table 1) and is classified as a high hazard, Class V dam because the town of McGregor is located just downstream (ND State Engineer, 1985).

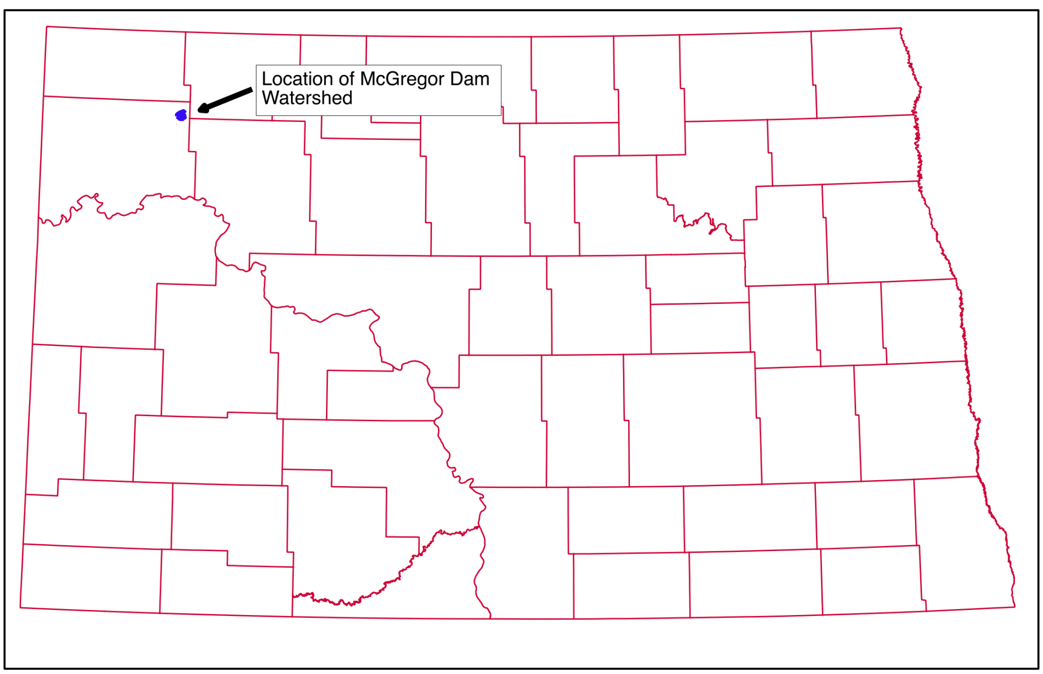


Figure 1. Location of McGregor Dam watershed.

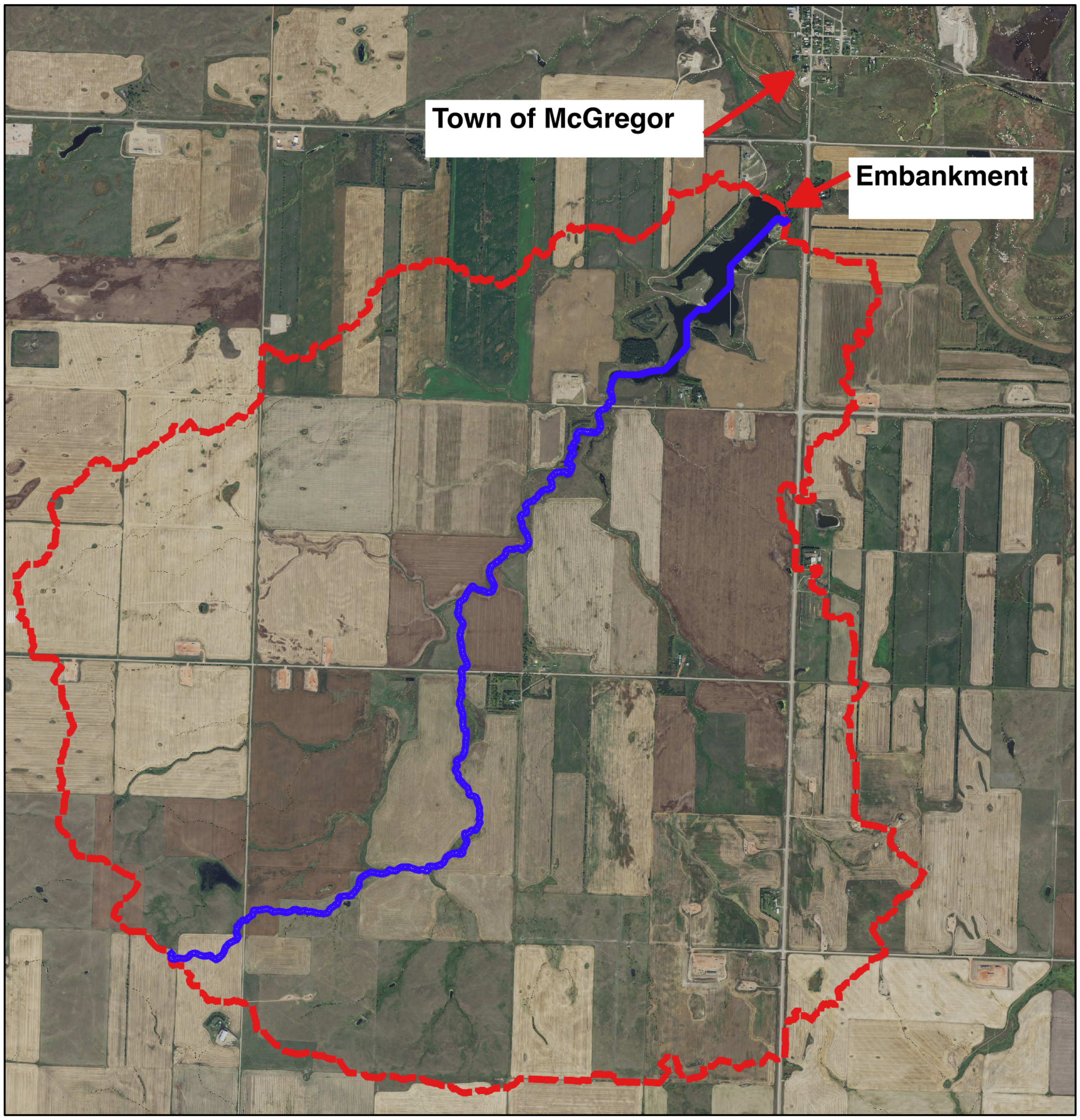


Figure 2. Aerial photo and boundary of McGregor Dam Watershed.

Table 1. Description of McGregor Dam (NDSWC, 1978)

	Height (ft)	Elevation (ft), NAVD 88	Area (acre)	Volume (acre-ft)
Max Pool	45	2269.33 (Top of embankment)	102.5	1,575
Emergency Pool	38.17	2262.50 (Bottom of Emergency Spillway)	69.5	990
Normal Pool	34.28	2258.61 (Inlet of principal spillway)	55.5	740

2.2 Structural, maintenance, and discharge capacity issues

The SWC inspected the dam in October, 2014 and May, 2015 (ND State Engineer, 2015) and found the following safety issues with the principal spillway:

1) The concrete was stripped away, leaving the aggregate exposed in the 30-inch diameter reinforced concrete outlet conduit.
2) Some cracks were found between joints.
3) The inlet screen and low-level valve were rusty.
4) The conduit outlet was submerged by high tailwater, which may reduce discharge capacity of principal spillway.

Safety issues with the emergency spillway included the following:

1) There was no means to divert water away from the downstream embankment toe. This geometry could concentrate water along the embankment toe causing erosion.
2) Trees were present in the downstream portion of the emergency spillway, which would impede discharge and result in erosion damage caused by turbulence and localized flow.

In addition, the estimated total discharge capacity of the dam did not meet dam safety standards according to a hydrologic analysis in 2009 (Casteel, 2009). The dam could not pass 40% of the PMP (Probable Maximum Precipitation). The dam safety standard for a high hazard, Class V dam requires it to pass 100% of the PMP without overtopping.

3. Methods

3.1 Hydrologic and Hydraulic model

In order to evaluate the response of McGregor Dam to rainfall events, it was necessary to construct numerical models of the system. HEC-HMS Version 4.1 software was selected for modeling basin runoff and reservoir elevation change. HMS is a hydrologic computer model developed by the U.S. Army Corps of Engineers' Hydrologic Engineering Center (USACE-HEC). It allows the use of synthetic or user-specified unit hydrographs, various infiltration methods, and user-defined or synthetic meteorological inputs to calculate the basin's response. It also allows for storage routing through user-defined reservoirs. Required data include meteorological data, reservoir/dam properties, linkages among subbasins, streams, and reservoirs, and parameters of loss, transformation, and routing of precipitation. This type of hydrologic model is typically assembled with field data and then tested with data from recorded events to verify their reliability before applying synthetic (ex. frequency-based) events.

HEC-RAS Version 5.0 software was selected for the 1-dimensional and 2-dimensional analysis of discharges through the dam's emergency spillway and principal spillway. HEC-RAS 5.0 is a hydraulic computer model developed by USACE-HEC that allows for more detailed routing of discharge. It also allows the use of 2-dimensional computational cells with user-identified roughness coefficients to show spatial distributions of velocities and water surfaces within the computational boundary. This type of hydraulic model is typically assembled with inflows from hydrologic observations or simulations.

3.2 Geometrical data

LiDAR (light detection and ranging) data, a remote sensing method (NOAA, 2016), were obtained from Williams County to identify elevations of the dam and surrounding topography (Williams County, 2016). Pixel size of the LiDAR raster grid was 3ft by 3ft. Survey data collected in June, 2016 were used to verify the quality of the LiDAR data. While the previous survey data were recorded in the NGVD 29 (National Geodetic vertical datum of 1929) datum (Casteel, 2009), these survey data were recorded in the NAVD 88 (North American Vertical Datum of 1988) datum. The 2016 survey included 2 benchmark points, 6 principal spillway points, 7 embankment points, 5 emergency spillway ground points and 1 water surface point. The LiDAR data were close in elevation to the survey data at the benchmarks (Appendix B). All elevation data shown in this report are referenced to NAVD 88.

3.3 Data collection of reservoir stage

A HOBO® water level logger (U20L) (ONSET, 2018) was installed in the reservoir to collect water surface levels every 15 minutes (Figure 4). The equation to convert pressure to water surface elevation was calibrated and validated by comparison with survey data and physical stage data. The water surface elevation data were used for calibration of loss and transformation parameters in the HEC-HMS model.

In the HEC-HMS model, rating curves of elevation versus capacity and capacity versus discharge for the reservoir were used. The elevation versus capacity curve for McGregor Dam was obtained from the Phase I Inspection Report (NDSWC, 1978).

3.4 Precipitation data

Daily precipitation data from two ARB (Atmospheric Resource Board of SWC) ground stations were collected for the simulation period from 7 am CDT, May 25 to 3 pm CDT, June 7, 2016 (NDSWC, 2016). One station is 7.4 miles southwest of the dam (ID: 4337) and the other is 1.9 miles northwest of the dam (ID: 972). Hourly radar precipitation data from the Earth Observing Laboratory (EOL) were collected to estimate hourly precipitation distribution of the daily data during the same simulation period (EOL, 2016). In HEC-HMS, the same rainfall depth weight (0.5) was assigned to both ARB ground stations because both stations were close to the watershed (Table 2). The time weight of EOL radar was 1.0 because that data set was used to decide hourly distribution (Table 2). Hourly precipitation generated by HEC-HMS has the distribution pattern from radar data but the magnitude was adjusted to match total precipitation from ground stations (Figure 3). The generated precipitation (red line in Figure 3) was used for calibration of hydrologic parameters in HEC-HMS (Refer to next section).

The 1%-chance exceedance precipitation (P_{100}) was obtained from Atlas 14 (NOAA, 2013). The 72-hour PMP with 15 minute-increments was obtained from Hydrometeorological Report No 52 (USACE, 1987) and was executed by MMC automated PMP generator (USACE, 2017). These precipitation events are displayed in Table 3 and were used to simulate synthetic events.

Table 2. Combining precipitation data from ground station and radar.

Gage Name	Total precipitation (inch)	Depth Weight	Time Weight
ID 972	3.34	0.5	0
ID 4337	3.23	0.5	0
EOL Radar	4.5	0	1.0

Table 3. Hourly cumulative precipitation of PMP, 0.4 PMP, and P_{100}.

Time (hr)	PMP (inch)	0.4 PMP (inch)	P_{100} (inch)
6	0.2	0.1	3.62
12	0.4	0.2	3.92
24	1.2	0.5	4.29
36	5.4	2.2	n/a
42	24.4	9.8	n/a
48	25.6	10.2	n/a
60	26.5	10.6	n/a
72	27.0	10.8	n/a

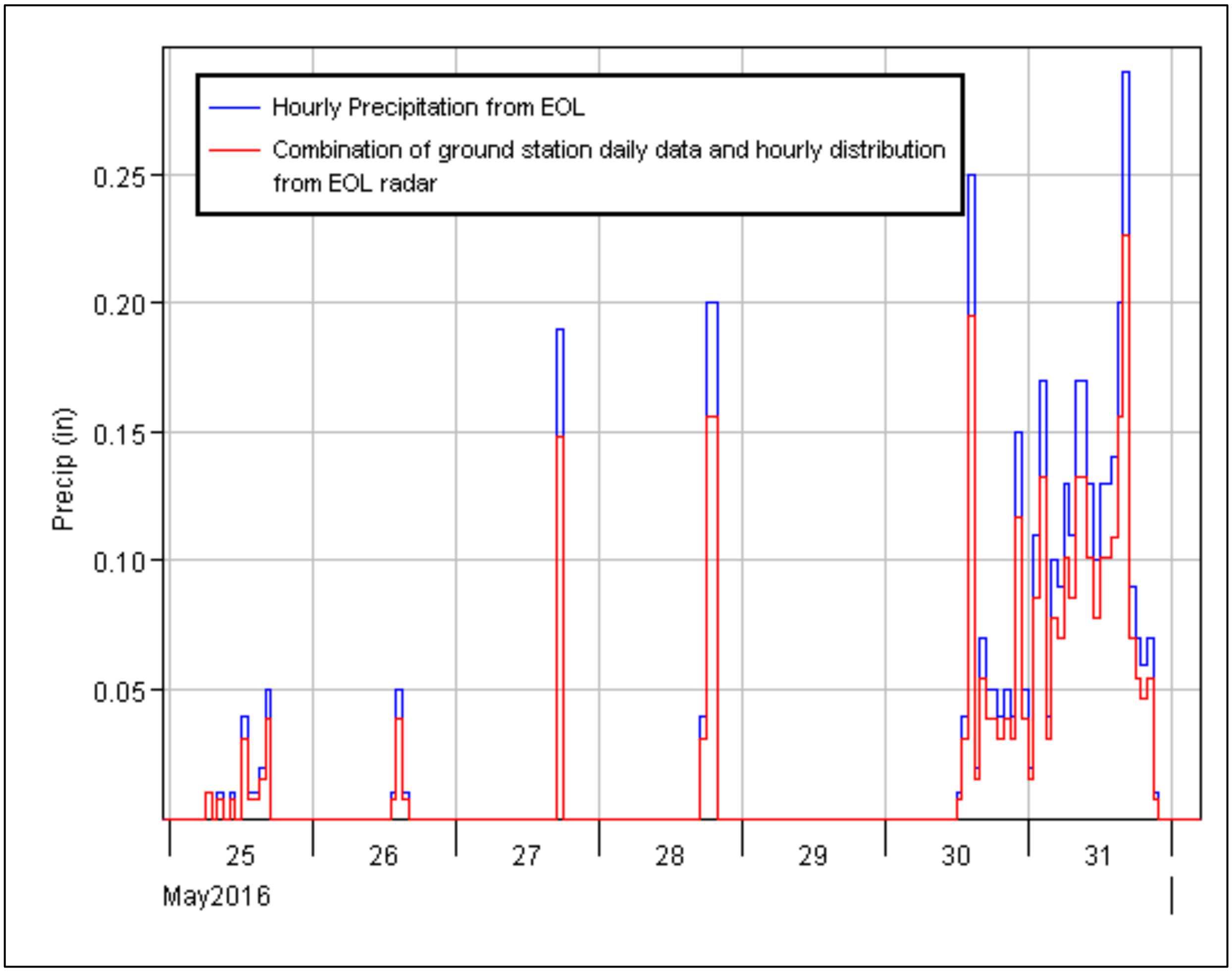


Figure 3. Hourly precipitation generated from the combination of ground station daily data and hourly distribution from radar (red line), and hourly precipitation from EOL (blue line).

3.5 Modeling pool elevation, discharge, and velocity

3.5.1 Calibration/Validation of HEC-HMS model

HEC-HMS version 4.1 was used to estimate reservoir water surface elevations under extreme precipitation cases. The model consisted of one watershed and one reservoir. The run period was 7 am CDT, May 25 to 3 pm CDT, June 7, 2016 for calibration and simulation. The Initial/Constant loss method was selected for computation of excess rainfall rate (inch/hr) and the Clark unit hydrograph method was selected for simulating the distribution of runoff at the inlet of the reservoir (Eq-2, 3, and 4). No baseflow was assumed in this project. The Initial/Constant loss method is described in Eq-1, which determines runoff rate (USACE, 2000).

$$pe_t = \begin{Bmatrix} 0 \;\; if \;\; \sum P_i < \; I_a \\ P_t - \; f_c \;\; if \; \sum P_i > \; I_a \; and \; P_t > \; f_c \\ 0 \;\; if \; \sum P_i > \; I_a \; and \; P_t < \; f_c \end{Bmatrix} \qquad (Eq - 1)$$

where P_i= cumulative precipitation (inch), I_a= deficit (initial loss), P_t= precipitation rate (inch/hr), f_c= constant loss rate (inch/hr), and pe_t= excess rainfall rate (inch/hr).

The Clark unit hydrograph transforms runoff into streamflow at the basin outlet by treating the temporary storage potential of the land surface as a linear reservoir. These flows then are aggregated time step by time step as contributing watershed area increases (Eq-2, 3, and 4) (USACE, 2000).

$$UH_t = \left(\frac{\Delta t}{R + 0.5\Delta t}\right) I_t + \left\{1 - \left(\frac{\Delta t}{R + 0.5\Delta t}\right)\right\} UH_{t-1} \quad (Eq - 2)$$

where UH_t = Unit Hydrograph (runoff rate per inch of excess rainfall per square mile of contributing watershed) at time t, R= a constant linear reservoir parameter, I_t = average inflow to storage at time t, and Δt = computational time step. The average Unit Hydrograph is calculated as follows:

$$\overline{UH_t} = \frac{UH_{t-1} + UH_t}{2} \qquad (Eq - 3)$$

The cumulative contributing watershed area is calculated by the following equation:

$$A_t = \begin{Bmatrix} 1.414A\left(\frac{t}{t_C}\right)^{1.5} \quad for \; t \; \leq \frac{t_C}{2} \\ A - 1.414A\left(\frac{t}{t_C}\right)^{1.5} \quad for \; t \; \geq \frac{t_C}{2} \end{Bmatrix} \qquad (Eq - 4)$$

where A_t = cumulative contributing watershed area at time t, A = total watershed area, and t_C = time of concentration. Finally, flow rate is calculated by multiplying average UH by A_t.

The calibration was conducted by comparing simulated to observed pool elevations. There was no consideration of discharge through spillways because the stage had not reached the elevation of the principal spillway inlet during the data collection period. The criterion for initial loss was the amount of precipitation corresponding to the first noticeable increase of the reservoir water surface level. Then the constant loss rate was adjusted targeting the maximum water surface elevation of the reservoir. Time of concentration (t_c) and storage coefficient (R) were adjusted targeting the time of peak stage (Figure 4).

The USACE has a recommended range of calibration parameter constraints (USACE, 2000). All of the calibrated parameters fell within this recommended range (Table 4). The pool elevation began to increase at 3 pm, May 30, at which time the cumulative precipitation (0.67 inch) was determined to be the initial loss.

Table 4. Calibrated parameter values compared to the USACE's recommended calibration range.

Model Method	Parameter	Calibrated	Minimum	Maximum
Initial and Constant rate loss	Initial loss, inch	0.67	0	20
	Constant loss rate, inch/hr	0.175	0	12
Clark's Unit Hydrograph	Time of concentration, hr	3	0.1	500
	Storage Coefficient, hr	10	0	150

The simulated pool elevation was in good agreement with observations from the stage gage. Since no evaporation or seepage was assumed in HEC-HMS, the declining pool was not indicated by the simulation (Figure 4). The calibrated parameters were used for the simulation of the P_{100}, 0.4 PMP, and PMP scenarios.

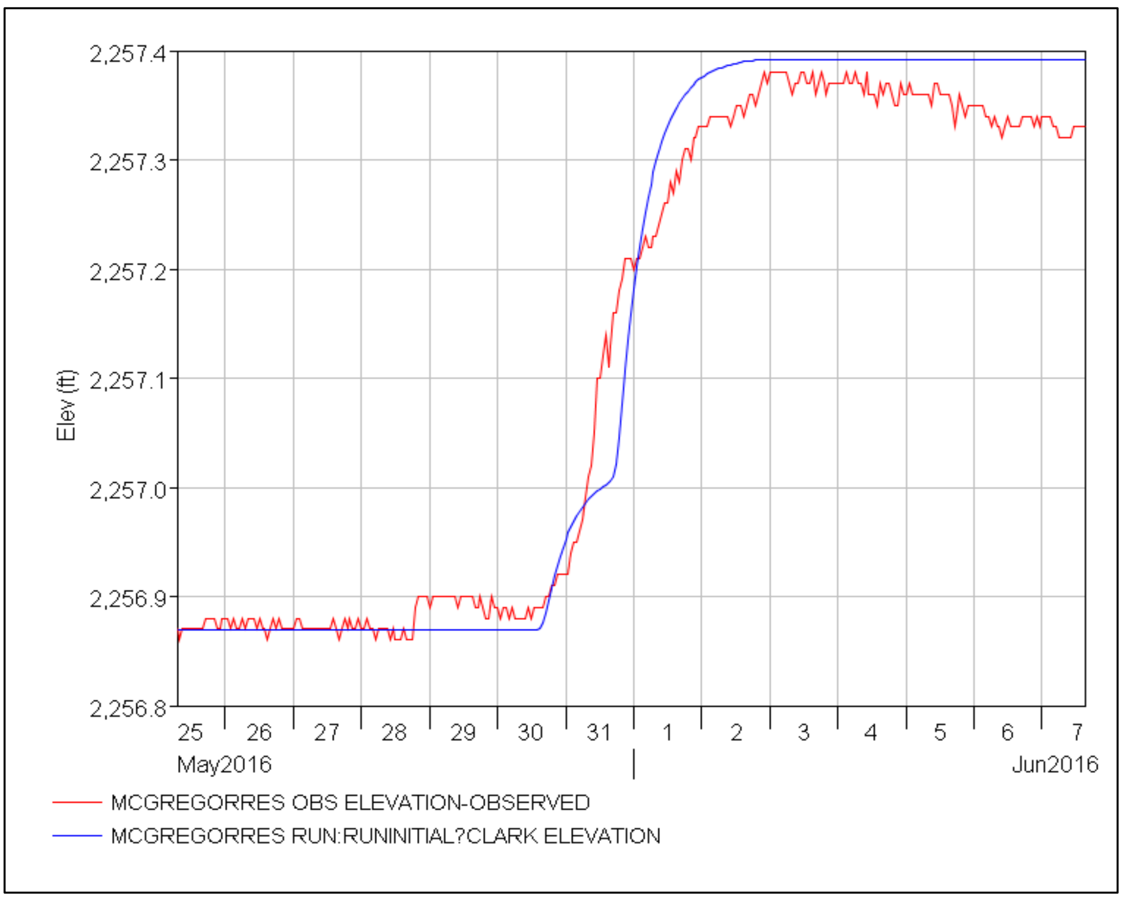


Figure 4. Agreement between observed and simulated pool elevation time series for calibration.

3.5.2 Developing discharge rating curves

Rating curves were developed for the principal spillway, emergency spillway, and embankment overtopping. Discharges through the principal spillway and embankment overtopping were obtained from the weir equation. Parameters of the weir equation were obtained from a previous report (Casteel, 2009). Discharges through the emergency spillway were developed from a HEC-RAS simulation (Appendix E). Discharges for given elevations were compiled and entered into HEC-HMS (Appendix D).

3.5.2.1 Principal spillway

Three types of flow occur in a principal spillway: weir flow, pipe flow, and orifice flow. Each flow type was calculated using the methodology described below and combined into a final rating curve as shown in Figure 5. Weir flow occurs as water enters the inlet box and is calculated using the following trapezoidal weir flow equation:

$$Q = CLH^{3/2} \qquad (Eq-5)$$

where Q = flow (cfs), C = weir coefficient, L = length of weir (ft), and H = head (ft). Length of weir (L) is calculated as follows:

$$L = B_W + 2\sqrt{(El_1 - El_2)^2 + (z(El_1 - El_2))^2} \qquad (Eq-6)$$

where B_W = bottom width (ft), El_1 = top of sloped section elevation (ft), El_2 = bottom of sloped section elevation, and z = slope.

Pipe flow is governed by friction within the conduit and is calculated using Bernoulli's Equation.

$$Z_1 - Z_2 = \left(\frac{fL}{D} + K_e\right)\frac{V^2}{2g} \qquad (Eq-7)$$

where Z_1 = beginning elevation (ft), Z_2 = ending elevation (ft), f = friction factor, L = length of pipe (ft), D = diameter of pipe (ft), K_e = minor loss coefficient, V = velocity (ft/s), and g = acceleration due to gravity (32.2 ft/s^2). The friction factor (f) is estimated through iteration by using the Swamee and Jain's equation.

$$f = \frac{.25}{\left[log_{10}\left(\frac{k_s}{3.7D} + \frac{5.74}{Re^{0.9}}\right)\right]^2} \qquad (Eq-8)$$

where k_s = roughness height (ft), D = pipe diameter (ft), and Re = Reynolds Number.

Orifice flow occurs at the entrance of the conduit, inside of the inlet box. Ordinarily, it is precluded by a smooth transition from weir control to full pipe flow. It needs to be considered in this case because the steep slope of the conduit may allow orifice conditions to create a zone of unstable flow conditions in the rating curve. Orifice flow was calculated using the following equation:

$$Q = CA\sqrt{2gh} \qquad (Eq-9)$$

where Q = discharge (ft/s), C = orifice coefficient, A = area (ft^2), g = acceleration due to gravity (32.2 ft/s^2), and h = height (ft).

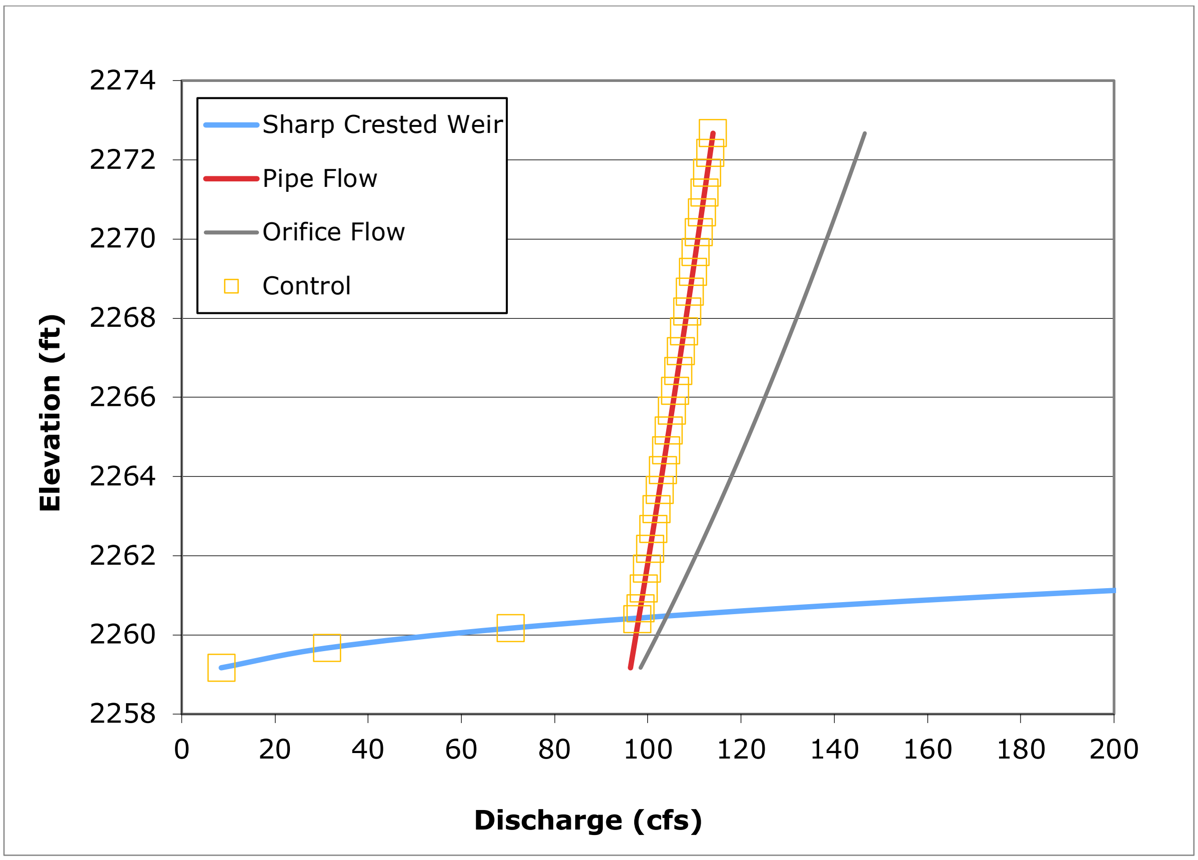


Figure 5. Principal Spillway Rating Curve

3.5.2.2 Emergency spillway

Three cross sections were used to estimate the emergency spillway rating curve in HEC-RAS 5.0 (Figure 6). Ground survey and LiDAR revealed the emergency spillway was not a simple trapezoidal shape (Figure 7). Rather an extra flow area existed to the southeast of the nominal spillway. This area would convey water if the reservoir reached the overflow elevation of this area (Figure 7). HEC-RAS modeling was used to simulate water surface levels for 21 different discharges from 123 to 50,000 cfs through the emergency spillway. The most upstream cross section shown in Figure 6 represents the crest of the emergency spillway as illustrated in Figure 7. The simulation results from this cross section were used to develop the rating curve for the emergency spillway.

The Manning's roughness coefficient values used in the HEC-RAS simulation were 0.035 for a channel with some weeds but without rifts and pools and 0.035 for flood plains with high grass (USACE, 2016). Manning's roughness was not adjusted to account for the trees in the downstream portion of the spillway because they would most likely not affect the velocity at the crest of the emergency spillway. Discharge through the emergency spillway becomes supercritical due to the increase in slope of the downstream face of the embankment. At the toe of the embankment and further downstream where the trees are located, the slope decreases causing subcritical flow. In order for the trees to affect the rating curve at the crest of the spillway, their effect would have to propagate upstream through the supercritical flow, which is highly unlikely. The resulting rating curve for the emergency spillway is shown in Figure 8.

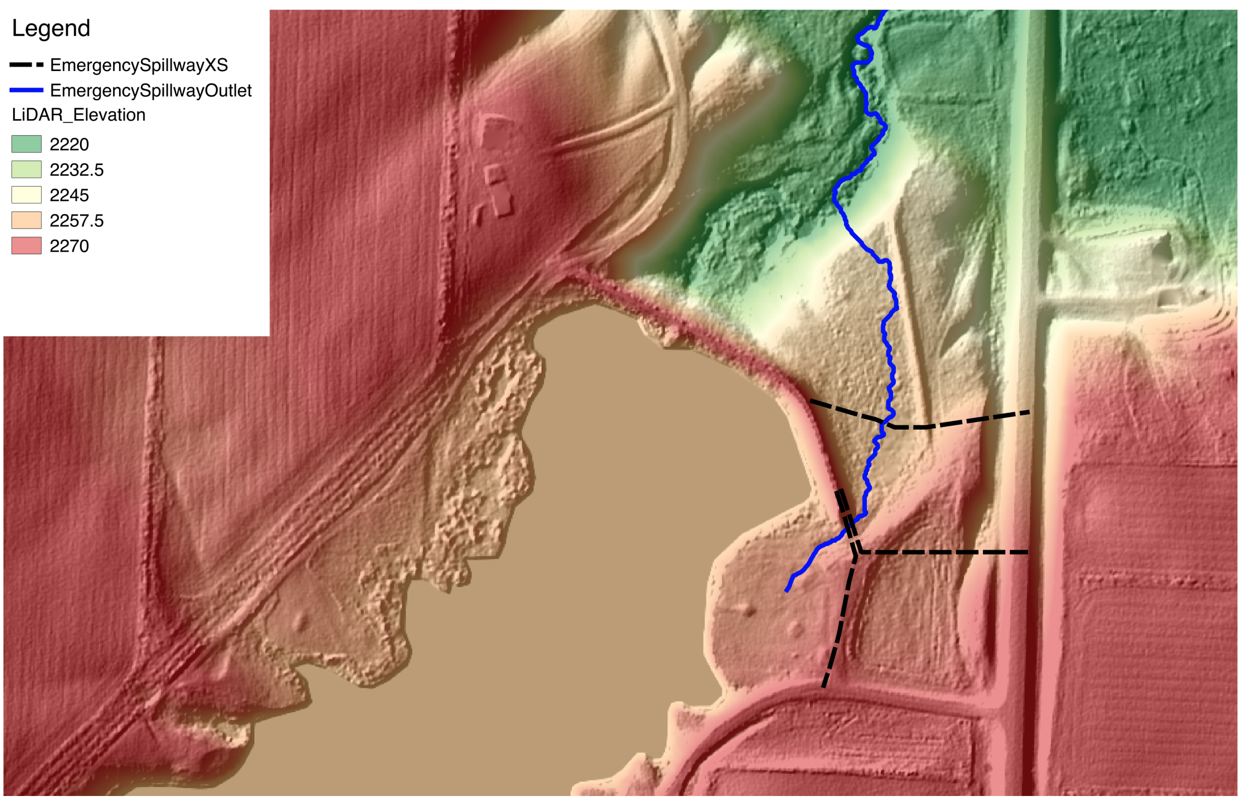


Figure 6. Cross sections for HEC-RAS simulation (black dashed lines represent cross sections and the blue line represents the flow path along the emergency spillway).

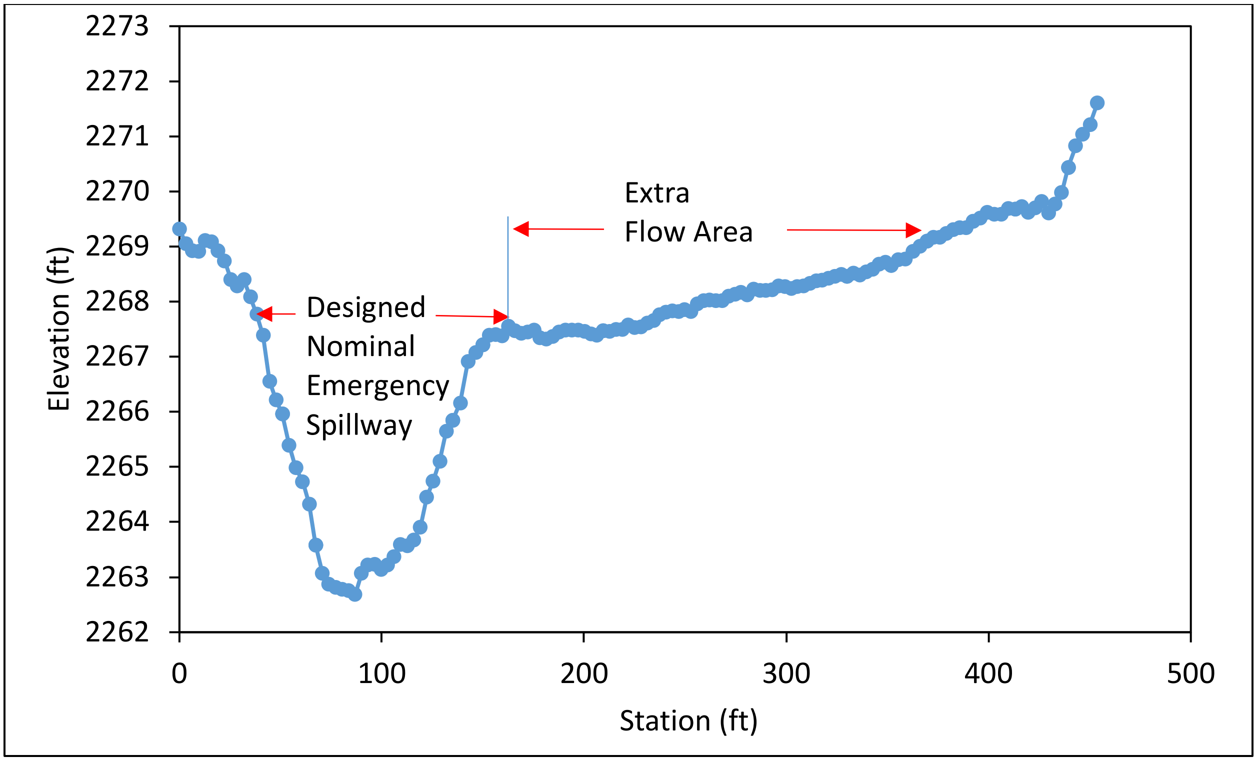


Figure 7. Profile of emergency spillway crest.

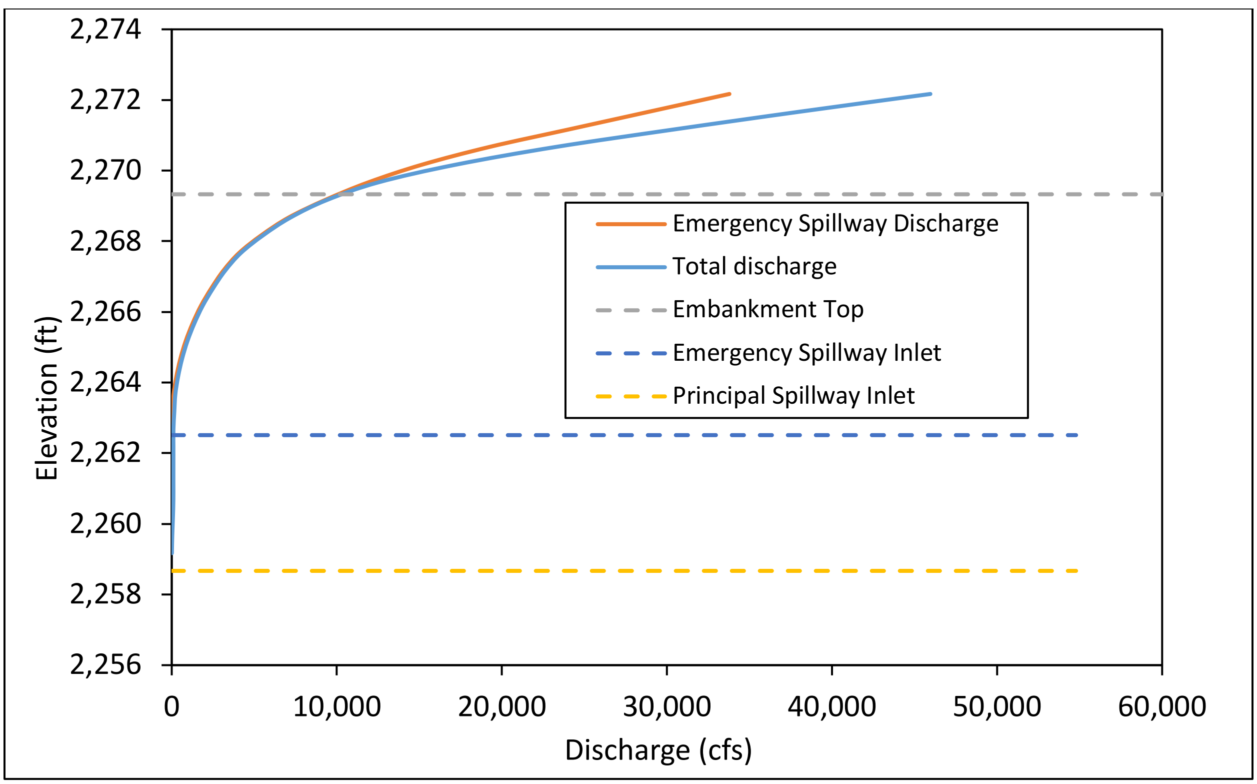


Figure 8. Total discharge (combination of principal spillway, emergency spillway, and overtopping) and emergency spillway rating curves for given pool elevations.

3.5.2.3 Embankment overtopping

Embankment overtopping was assumed as broad-crested weir flow with a weir length of 900 ft. Emergency spillway discharge was 97% and 74% of total discharge at top of embankment and at elevation 2272.17 ft (2.8 ft higher than top of embankment), respectively (Figure 8). The emergency spillway is the dominant discharge route with and without over-topping once the water surface reaches the crest of the emergency spillway. The total discharge rating curve for the dam is displayed in Figure 8.

3.6 Simulation of extreme cases

According to the ND Dam Design Handbook (ND State Engineer, 1985), McGregor Dam's principal spillway must pass the runoff from a P_{100} event and its emergency spillway must convey runoff from a PMP event without overtopping. After calibration of the parameters, pool elevations were simulated using HEC-HMS for the P_{100}, 0.4 PMP, and PMP scenarios. The starting reservoir elevation for each scenario was the principal spillway inlet elevation.

The velocity through the emergency spillway for the 0.4 PMP event was estimated using a 2D HEC-RAS model to evaluate erosion. According to the ND Dam Design Handbook, the maximum permissible velocity during a 0.4 PMP event for a Class V dam is 5 ft/s for the combination of 'group 1' and 'easily eroded soils' (ND State Engineer, 1985). The 2D model consisted of an upstream storage area representing the reservoir and a 2D area representing the spillway and downstream face of the dam. Two 1D/2D connections were installed between the storage area and the 2D area. One represented the embankment with discharge rating curve of the principal spillway (Figure 5). The other represented the inlet of the emergency spillway with its discharge rating curve (Figure 8). The 0.4 PMP event from the HEC-HMS simulation was used as the inflow boundary condition. The discharges into the 2D area through the principal spillway and the emergency spillway were estimated by the reservoir elevation and the two rating curves. The roughness coefficient (Manning's n) was 0.035 within the 2D area. The default computational cell size was 50 ft by 50 ft; however, smaller cells (5 ft to 30 ft long) were used for certain parts of the embankment, emergency spillway, and downstream areas to track details of flow directions and velocities (Figure 10). The smaller cells were also beneficial to avoid dry cell issues within shallow flow areas (Goodell, 2015). The maximum estimated velocity of this scenario was used to evaluate the dam safety standard.

4. Results

4.1 P_{100} event

During the inflow of the P_{100} event, the pool crested at approximately elevation 2264.7 ft, 2.2 ft above the elevation of the emergency spillway crest (Figure 9). Therefore, the capacity of the principal spillway does not satisfy the dam safety standard.

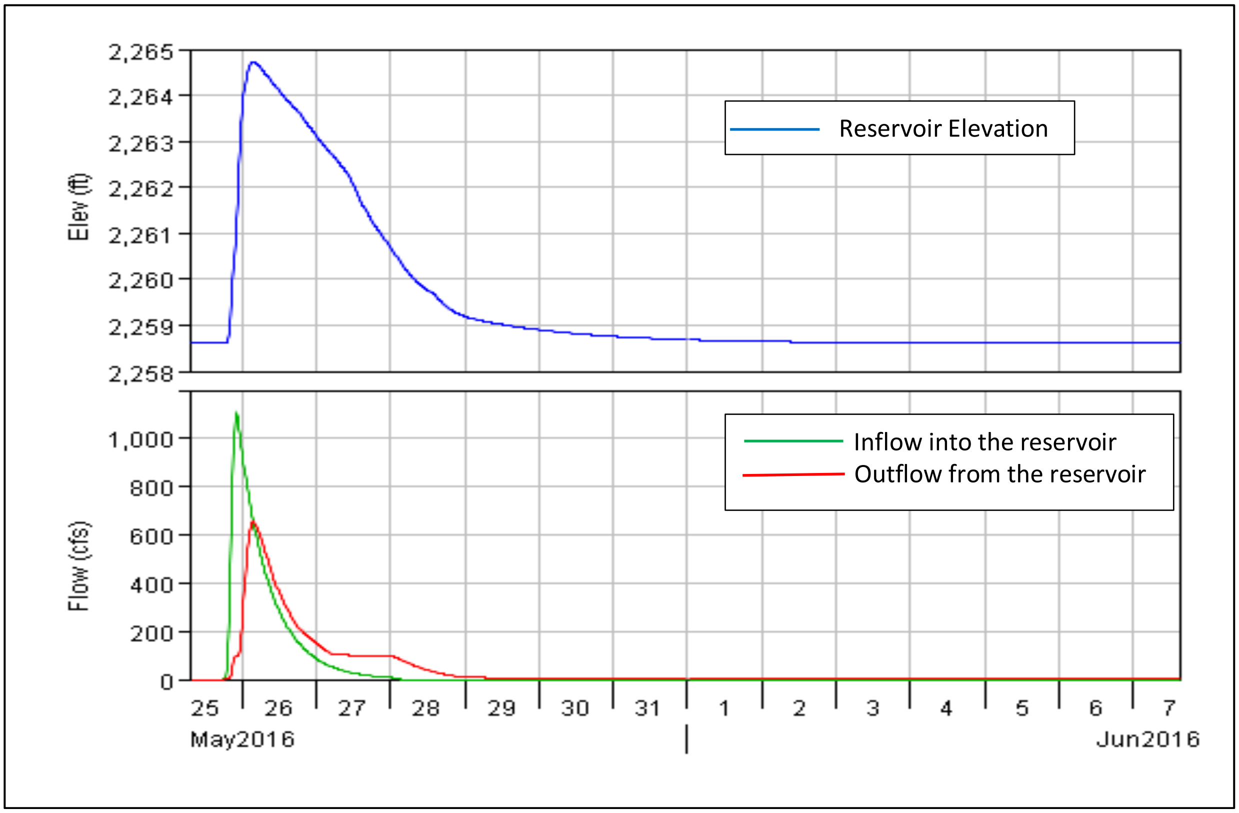


Figure 9. Pool elevation, inflow, and outflow for the P_{100} case.

4.2 0.4 PMP event

In the case of the 0.4 PMP event, the maximum pool elevation was between the emergency spillway crest and top of embankment. At the maximum pool elevation of 2266.6 ft, discharge was 2,450 cfs (Figure 11) and velocity was up to 12 ft/s at the nominal emergency spillway and immediate downstream face of the embankment (Zone 1 in Figure 10). The simulated velocity surpasses the allowable permissible velocity; and therefore, does not meet the dam safety standard.

The velocity analysis revealed additional erosion issues. In Zone 2 (Figure 10), the flow velocity increases up to 5.6 ft/s as water flows along the Highway 50 ditch. This situation increases the risk of erosion damage to the highway. In Zone 3 (Figure 10), eastward flow accelerated up to 5 ft/s through the down slope (Figure 12(a)). Water flowing through this zone would most likely cause erosion and increase the risk of higher magnitude flows being diverted along Highway 50. In Zones 4 (Figure 12(b)) and 5, the northwestward flow accelerated through down slopes up to 12 ft/s and 24 ft/s, respectively. Since Zone 4 is adjacent to the embankment toe, with Zone 5 in close proximity, the high velocity flow in these zones would most likely increase the risk of erosion and collapse of the middle of the embankment.

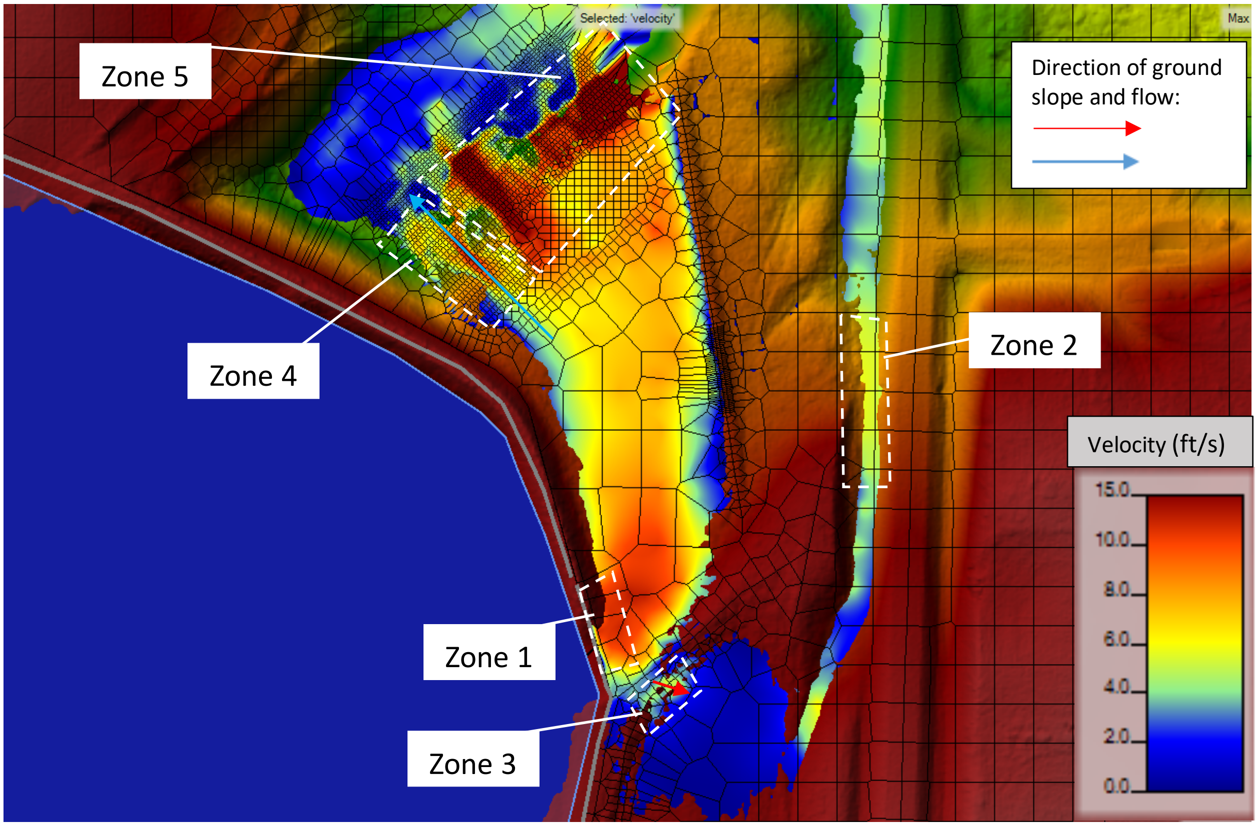


Figure 10. Discharge velocities downstream of the emergency spillway.

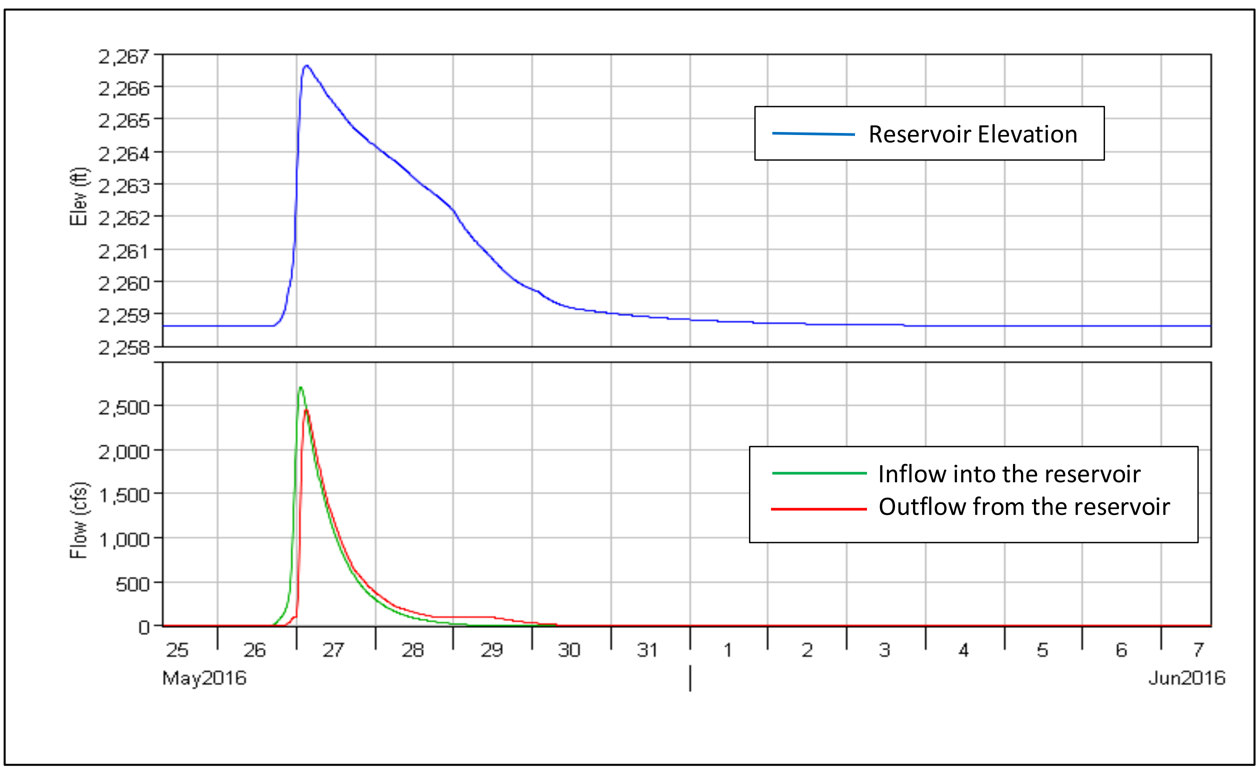
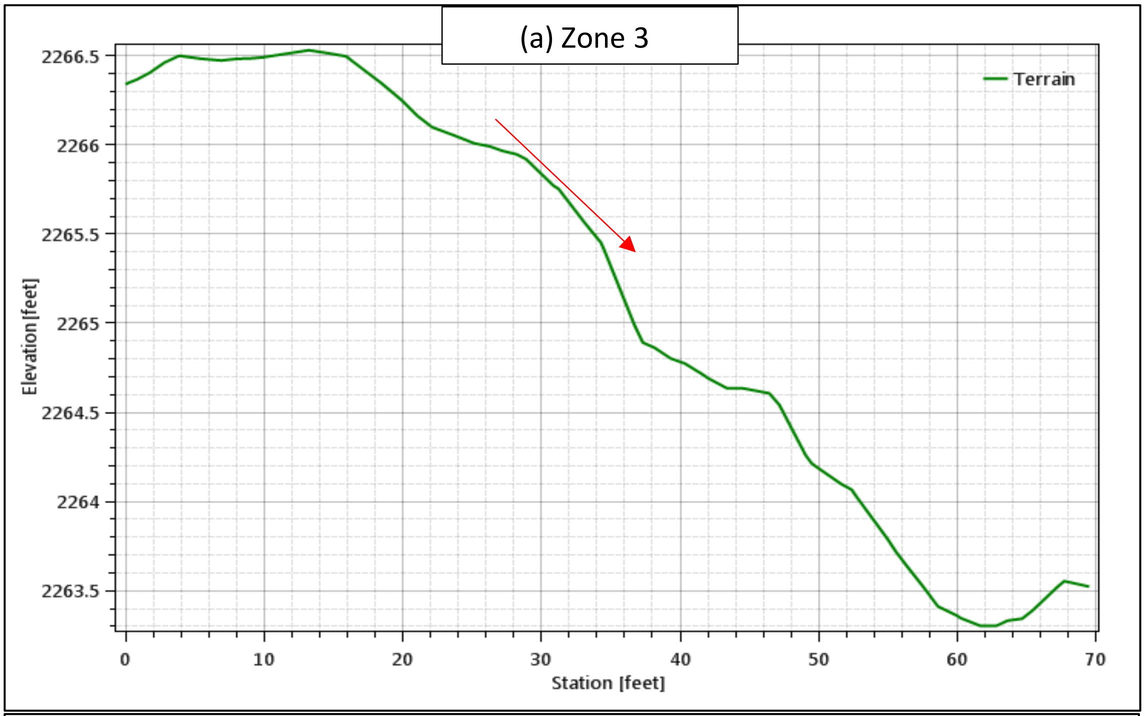


Figure 11. Pool elevation, inflow, and outflow for the 0.4 PMP case.



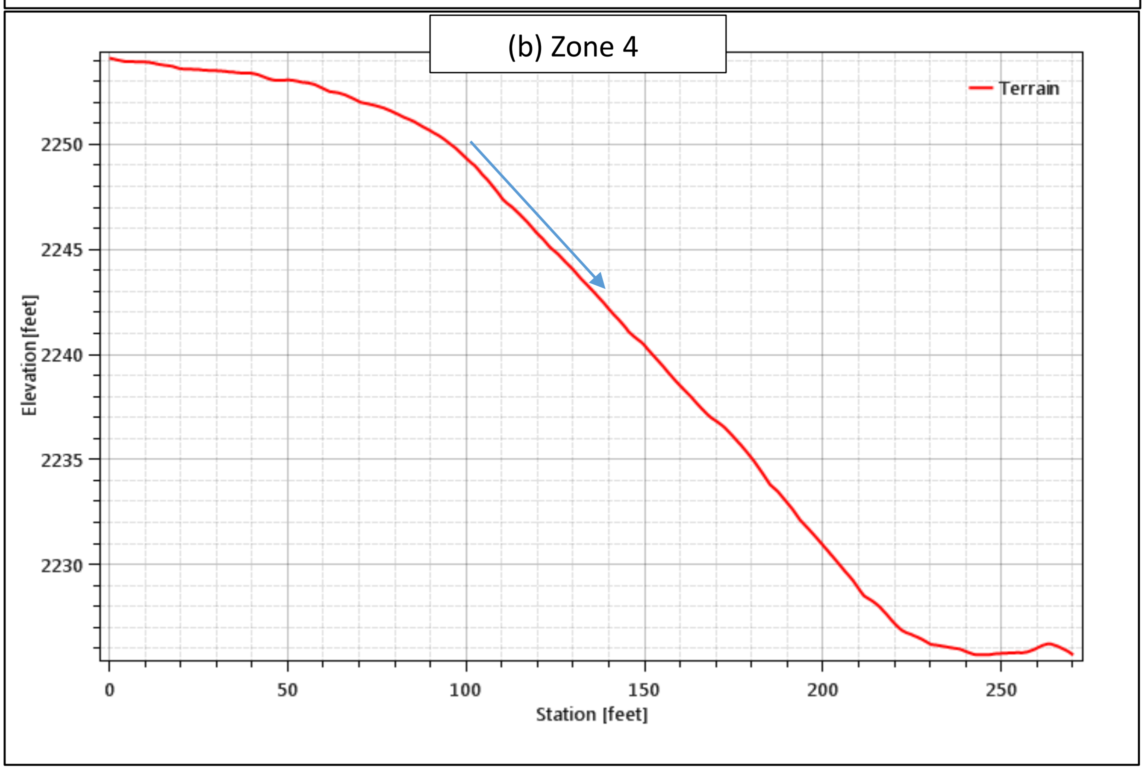


Figure 12. Ground elevation profile for (a) Zone 3 and (b) Zone 4.

4.3 PMP event

The maximum pool elevation during the PMP event was 2268.8 ft, 0.5 ft below the top of embankment. Since the emergency spillway, augmented by the extra flow area, has sufficient capacity to pass the full PMF (Probable Maximum Flood) event, the embankment is not overtopped. However, the maximum discharge was over 7,600 cfs, and inflow and outflow hydrographs are identical. The dam has no flood control function for a PMP event (Figure 13).

It should be noted that the maximum reservoir elevation is very close to the top of the embankment. If a PMP event occurred with wetter antecedent conditions or a higher starting reservoir elevation than modeled, the dam would most likely be overtopped. Considering that undetected errors may exist in data collection/processing and calibrations of models, it is strongly suggested that the dam be considered as overtopped.

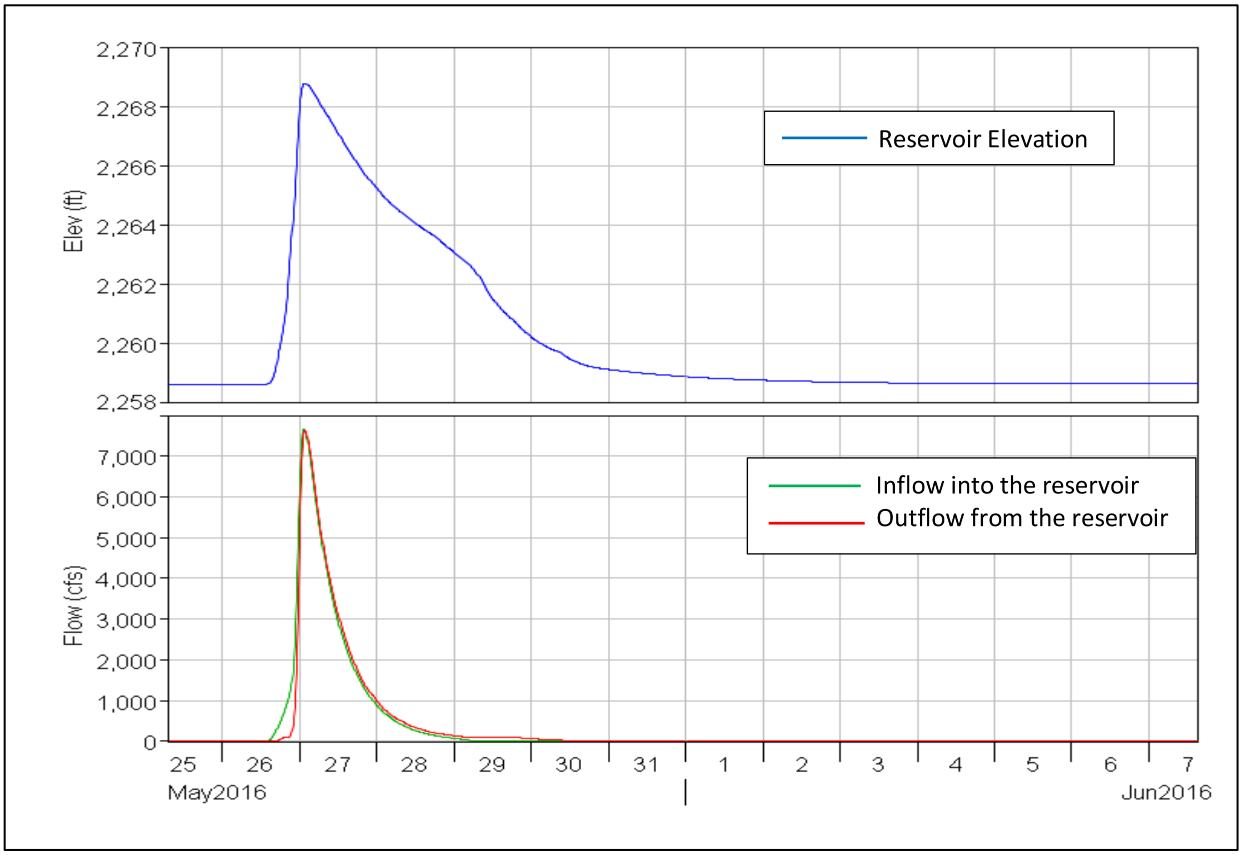


Figure 13. Pool elevation, inflow, and outflow for the PMP case.

5. Discussion

5.1 Comparison of previous and current investigations

The results in this investigation differ considerably from those in the 2009 hydrology report for the Emergency Action Plan (Casteel, 2009). In this investigation, precipitation data and GIS processing tools were available which were not available in 2009. These improvements enhanced reliability in the numerical model to match observed events. In late spring of 2016, the reservoir was instrumented with a recording stage gage, which was referenced to survey monuments near the structure and to the recently available LiDAR topographic data. This allowed continuous observation of reservoir stages during rain events. Several subsequent rainfalls occurred, and when the data was retrieved it not only showed the total volume of inflow, but also the timing. The LiDAR enabled a more reliable estimate of the basin's time response, which was verified with the stage record.

Likely the change having the largest effect was the use of a Clark unit hydrograph. Clark includes a parameter to address temporary basin storage in addition to a Time of Concentration parameter. The previously used SCS unit hydrograph uses only the timing factor. Clark allows more flexibility in matching a recorded hydrograph, but without a recorded hydrograph this advantage would have limited value.

Ground observations and radar data allowed good resolution of the spatial and temporal distribution of the rainfall events. These factors combined give good confidence in the result. As a final check, the volume of the PMF event modeled in this work and the volume reported in the 2009 report were compared. Even though the PMF peak flow in the 2009 report (14,000 cfs) is about double the peak of this report (7,640 cfs), inflow volume from the 2009 report (335 million cubic feet) is within 88% of the inflow volume in this report (380 million cubic feet). This points to the unit hydrograph as the main cause for the discrepancy and the one used in this work has a sounder basis.

5.2 Recommendations to improve dam safety

As described above, McGregor Dam has structural and discharge capacity issues. Four potential alternatives to resolve these issues are discussed below.

5.2.1 Explanation of Alternatives

5.2.1.1 Alternative A: Do nothing

This alternative has the lowest implementation cost. However, the problems mentioned above would not be addressed, and in the long term, maintenance, repair, and replacement costs would be substantial. This would result from:

1) Discharge capacity deficiency of the principal spillway
2) Corrosion inside the drain box and conduit of the principal spillway
3) Seepage along the conduit of the principal spillway
4) Erosion of the dam embankment caused by discharge through the emergency spillway

Details regarding the damage caused by a dam breach are discussed below.

5.2.1.2 Alternative B: Repair principal spillway and replace emergency spillway

This alternative includes plastic cured-in-place pipe (CIPP) liners inside the conduit of the principal spillway and installing a structural emergency spillway.

According to the ND Dam Design Handbook, the flow from a 1%-chance exceedance precipitation event must be passed without use of the non-structural emergency spillway. So, if the current emergency spillway is replaced with a structural emergency spillway, the discharge capacity of the principal spillway does not need to be evaluated, but the cracks and corrosion must be repaired.

A) Repair of the principal spillway conduit with CIPP

CIPP is typically used for an inaccessible conduit. CIPP liners can be inserted into the existing conduit by either the inversion method or the pulled-in-place method (FEMA, 2005).

In the inversion method, hydrostatic head (air or water) is utilized to push the CIPP liner through the conduit. The liner is then inflated and sealed to the original conduit by the use of hydrostatic head or air pressure. The diameter range is from 4 to 108 inches (FEMA, 2005).

In the pulled-in-place method, a winch is attached to a cable, which is attached to the CIPP liner and used to pull the liner into position. This method is usually only used where sufficient water pressure is not available or where a particular lining is required. The diameter range of this method is from 4 to 96 inches (FEMA, 2005).

CIPP is usually thermally cured by the circulation of heated water (up to 180°F) after the CIPP liner is in place.

B) Replacing emergency spillway with RCC

Roller Compacted Concrete (RCC) is defined as “concrete compacted by roller compaction that, in its unhardened state, will support a roller while being compacted” (ACI, 2013). RCC is a common method to protect the downstream slope of a dam embankment from failure due to overtopping. The height of the RCC protected dams have ranged from 15 to 110 ft (Bass et.al., 2012). The major considerations of the RCC spillway design include spillway location, hydraulics of stepped spillways, spillway channel, and width of overtopping protection. The process of RCC spillway construction consists of the following 6 steps (PCA, 2002).

- Dewatering and foundation preparation

When the structure foundation consists of soil (especially sand or silt), lowering the groundwater table is required to obtain a firm subgrade. On many RCC projects, sumps and ditches are used to control the ground water table. Prior to placement of RCC, soft and weathered materials are removed or a concrete mud slab is placed on the freshly excavated surface material to prevent further subgrade deterioration during construction.

- RCC mixture production.

There are two types of concrete mixing plants. In continuous mixing plants, RCC mix components are proportioned continuously by calibrated belts and screws based on the rate of production, with mixing in a pug mill. In a batch plant, RCC components are mixed by a drum or compulsory mixer.

- RCC delivery

The purpose of this step is to provide a quality RCC mixture free of segregation or contamination in a timely manner (PCA, 2002). Usually this step is carried out using one of three ways: 1) hauling equipment, 2) belt conveyor, or 3) a combination of (1) and (2). Hauling equipment is typically used for a low volume of mixture while a belt conveyor is typically used for a high-volume of mixture. (NRCS, 2011).

- Spreading of RCC

When dumping, placing, and spreading RCC on the fill surface, the RCC should be dumped on uncompacted rather than compacted RCC to reduce the segregation. By dumping and replacing RCC on uncompacted RCC, the spreading equipment is able to provide some additional mixing of RCC.

- Compaction of RCC

As the density increases, the strength and durability of RCC improves. But excessive rolling can actually decrease density of some mixtures and induce surface cracking (NRCS, 2011). Large diameter single and double drum vibratory compactors are ideal for production compaction of RCC because a large roller can attain the specified density in as few passes as possible (PCA, 2002).

- Curing of RCC

RCC must be cured for development of durability and strength. Generally, RCC surfaces should be kept continually moist for 14 days at or above 40°F (NRCS, 2011).

5.2.1.3 Alternative C: Remove principal spillway and replace emergency spillway

This alternative includes excavating some embankment, taking out the principal spillway's box and conduit ,and installing a structural emergency spillway. One structural spillway that can pass the range of discharges up to the PMF without overtopping would fill the role of both the principal and emergency spillways.

Generally, removal of an existing conduit through an embankment dam consists of excavating the dam down to the existing conduit, stockpiling the material, removing the existing conduit, and replacing the embankment material. A cofferdam may also be required if the reservoir cannot be drained during construction. The emergency spillway could be reconstructed by the same process as described in Alternative B.

5.2.1.4 Alternative D: Partial removal of dam embankment to drain reservoir (Dam Decommissioning)

Some old dams have safety issues or the reservoirs are full of sediment. The original purposes of the dams are no longer needed or significant environmental benefits may be achieved by removing the dam. In these cases, dam removal may be an alternative (USSD, 2015).

The decision to remove a dam should be based on a wide range evaluation of alternatives to solve a specific problem at an existing facility, including dam safety concerns, high repair costs, high operation and maintenance costs, or significant impacts on aquatic resources and water quality. In some cases, the problem can be solved by partial removal of the dam rather than by full removal of the dam (USSD, 2015). This alternative includes installing a drain channel, excavating a portion of the embankment, and draining the reservoir under controlled conditions.

5.2.2 Evaluation of alternatives

Costs for alternatives were estimated from a combination of literature (Table 5). The dimension of the embankment in a previous report was used to estimate excavation volume. The bottom width of the embankment was 235 ft based on the dimension, which includes an upstream slope of 1V:3H and a downstream slope of 1V:2H, a top width of 10 feet, and a height of 45 feet (NDSWC, 1978).

For replacement of the current emergency spillway with RCC, the dimensions of the RCC spillway were estimated using the broad-crested weir equation with a weir coefficient (C) of 2.8 (Bass et.al., 2012). The elevation of the spillway bottom was the same as the current principal spillway. The height of the spillway was 8.1 feet (difference between the bottom and top of the current emergency spillway). The width was determined to be 118 feet in order to pass the PMF peak (7,640 cfs). The unit cost for RCC placement was determined from RSMeans Heavy Construction Cost Data (RSMeans, 2013) and a 3% inflation was applied between 2014 and 2017. After estimating the cost of RCC placement, the cost distribution percentages from the Sweetbriar Dam Spillway Alternatives (BW/BEC Joint Venture, 2006) were used to estimate the costs of construction items except Dewatering, Clearing and Grubbing, and Reclamation of Disturbed Areas, whose costs are recommended by the SWC Construction Section.

Costs of CIPP placement were estimated from the Engineer's Statement of Cost for Swan Buffalo Detention Dam Repairs (Maple River Water Resource District, 2016). The material cost for the reference ($450/feet for 36"conduit) was converted to the cost for the conduit of McGregor dam ($375/feet for 30"conduit) based on circumference of coating surface. No excavation was assumed. An inflation rate of 3% was used to adjust the CIPP cost and other lump sum costs for 2017.

The amount of soil excavation for the principal spillway (concrete box and 30" conduit) removal was calculated assuming excavation lengths of 5 ft and 120 ft for the bottom and top, respectively. The cost of soil excavation was estimated from Sweetbriar Dam Spillway Alternatives (BW/BEC Joint Venture, 2006). An inflation rate of 3% was used to convert costs in 2006 to costs in 2017. The cost of compact embankment fill was estimated by the same manner as excavation.

The cost estimation of dam decommissioning was a challenge, because cost estimation data on earth embankment removal was limited. The cost was estimated by referring to work done on Briggsville Dam in Massachusetts (MA Department of Fish and Game, 2015). The amount of soil excavation was calculated assuming excavation lengths of 10 ft and 240 ft for the bottom and top, respectively.

Table 5. Preliminary Probable Construction Cost Summary (Details are in Appendix C)

Alternative	Cost
A – Do nothing	$ 0
B – Repair principal spillway and replace emergency spillway	$ 866,000
C – Remove principal spillway and replace emergency spillway	$ 1,102,000
D – Dam Decommissioning	$ 569,000

5.2.2.1 Alternative A: Do nothing

In the McGregor Dam Emergency Action Plan (Houston Engineering, 2010), two different dam breach analyses were used to delineate the area that could be inundated in the event of a dam failure. For the sunny day breach simulation, the dam was breached assuming failure due to piping. At the beginning of the simulation, the reservoir elevation was 1.5 feet above the zero-flow level (elevation 2260 ft, NAVD88). The flow entering the reservoir was 50 cfs, and the discharge from the reservoir was routed to White Earth Creek, where the flow was 400 cfs. The breach bottom width was 75 feet with a side slope of 0.9. The failure time was 0.5 hour.

For the PMF breach, the dam was breached by an overtopping event. At the beginning of the simulation, the PMF into the reservoir was 14,000 cfs, and the reservoir elevation was 1.5 feet above the zero-flow level. The flow in White Earth Creek was 1,000 cfs. The breach bottom width was 150 feet, and the side slope was 1.4. The failure time was 0.5 hour.

For the sunny day breach case, two commercial and 17 residential buildings were expected to be inundated. For the PMF breach case, 12 commercial and 48 residential buildings were expected to be inundated. In the case of a dam failure with 7,000 cfs, which is the PMF in this report, it is clear that many buildings are expected to be inundated. No action may threaten life and property of the people in the town of McGregor.

5.2.2.2 Alternative B: Repair principal spillway and replace emergency spillway

The cost of this alternative is 23% lower than Alternative C, but 43% higher than Alternative D (Table 5). Seepage was observed at the joints inside of the principal spillway conduit during 2006's safety inspections. This is a concern due to possible piping along the outside of the conduit. Even though this alternative is feasible economically to meet our dam safety standard, the possible piping along the outside of the conduit will be a continuing hazard. In addition, when a CIPP lining eliminates seepage into the conduit, the flow patterns within the surrounding embankment are changed and other undesirable seepage paths may develop.

5.2.2.3 Alternative C: Remove principal spillway and replace emergency spillway

This is the most expensive, but safest alternative (Table 5). Removal of a deteriorating conduit is time consuming and expensive compared to other renovation methods. Excavations must be wide enough at the bottom to ensure adequate working room for removal of the existing conduit and compaction of earthfill materials. However, seepage along the conduit will not occur after removal and only one spillway will pass all flows. In addition, possible erosion downstream of the embankment can be avoided by passing water through a constructed spillway and spillway channel. It is also beneficial for maintenance and safety.

5.2.2.4 Alternative D: Dam decommissioning

The risk of a dam breach would be avoided with this alternative saving dam maintenance costs (MA Department of Fish and Game, 2015), but the community would lose the recreation area (Shuman, 2002). In addition, sediment loading is often a critical issue related to dam decommissioning. The degree of sediment impact can vary depending on the local conditions and the removal methods and rates (Greimann, 2004). Those sediment issues associated with dam removal include water quality impacts, increased flooding potential, impacts on existing infrastructure, cultural resource impacts, ecological impacts on fish and wildlife, recreation impacts, reservoir restoration requirements, and potential mitigation costs (USSD, 2015). So, sediment management is occasionally required to mitigate the impacts.

6. Summary and Conclusion

1. McGregor Dam has structural issues including damage to the 30” concrete conduit of the principal spillway and a rusty inlet screen on the low-level valve.

2. The emergency spillway has safety issues including no means to divert water away from the downstream embankment toe (causing an embankment erosion issue). Also, the downstream portion of the emergency spillway is covered with trees impeding the discharge flow and causing erosion damage by turbulence and localizing flow.

3. The capacity of the current principal spillway is insufficient to pass the runoff from a 1% chance exceedance precipitation event according to the HEC-HMS simulation. The principal spillway does not meet dam safety standards.

4. The capacity of the current emergency spillway barely passes the PMF according to the HEC-HMS simulation and caution should be used in asserting that the dam complies with the overtopping dam safety standard. Erosion of the downstream embankment and embankment toe are also serious issues. The simulated velocity surpasses the permissible velocity; and therefore, does not meet the dam safety standard. In addition, erosion from high velocity flow is expected along Highway 50.

5. Four alternatives were considered including: No action; Repair principal spillway and replace emergency spillway; Remove principal spillway and replace emergency spillway; and Dam decommissioning.

6. The “No action” alternative has no construction cost but the risk to life and property remains.

7. The “Remove principal spillway and replace emergency spillway” alternative is the most expensive, but alleviates the problems, while still maintaining the recreational value of the reservoir.

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APPENDIX A. Agreement

SWC Project No. 528
Project Manager: Jlee
April 2016

Memorandum of Understanding

1. **PARTIES.** This Memorandum of Understanding is between the North Dakota State Water Commission (Commission) and the North Dakota Game and Fish Department (Department).

2. **PROJECT DESCRIPTION.** McGregor Dam, located in Williams County, is currently in a condition that may endanger downstream areas. Commission will assess the current status of the dam regarding safety and recommended remedy measures (Project).

3. **COMMISSION'S OBLIGATIONS.** Commission will:

   a. Examine hydrology of the McGregor Dam drainage basin.

   b. Evaluate options to repair McGregor Dam and bring it up to current safety standards.

   c. Complete a written report with findings, including cost estimates.

4. **DEPARTMENT'S OBLIGATIONS.** Department grants permission for access and modification to property related to the Project.

NORTH DAKOTA STATE WATER COMMISSION

Todd Sando

TODD SANDO, P.E.
Chief Engineer and Secretary

Date: 4/19/16

NORTH DAKOTA DEPARTMENT OF GAME AND FISH

Terry Steinwand

TERRY STEINWAND
Director

Date: 5/4/16

APPENDIX B. Survey Data

Point ID	Elevation	Feature Code	Time	Duration	Method	Code		LiDAR
1	2267.26	ENG BM NO. 204 I	5/2/16 12:25	00:03.0	Topo	N-204 RESET 1974		
2	2422.22	WRITING ROCK RE	5/3/16 10:42	00:30.0	Observed Control			
1000	2258.61	3FT MARK GAGE	5/2/16 13:18	00:01.0	Topo			
1001	2267.16	ENG BM 204 RESE	5/2/16 15:18	00:31.0	Observed Co	N-204 RESET 1974		2267.22
1001	2267.16	ENG BM 204 RESE	5/2/16 13:18	00:10.0	Topo	N-204 RESET 1974		2267.22
1002	2257.02	WS	5/2/16 13:25	00:02.0	Topo			
1003	2264.65	BM CHISLED SQUA	5/2/16 13:29	00:04.0	Topo			2264.65
1004	2267.55	CL SPILLWAY GRN	5/2/16 14:14	00:02.0	Topo	GROUND		2267.32
1005	2264.83	CL SPILLWAY GRN	5/2/16 14:15	00:02.0	Topo	GROUND		2265.49
1006	2262.78	CL SPILLWAY GRN	5/2/16 14:15	00:03.0	Topo	GROUND		2263.55
1007	2262.72	CL SPILLWAY GRN	5/2/16 14:15	00:02.0	Topo	GROUND		2262.77
1008	2268.38	CL SPILLWAY GRN	5/2/16 14:16	00:02.0	Topo	GROUND		2267.77
1009	2269.26	CL DAM GRND	5/2/16 14:17	00:02.0	Topo	CL EMBANKMENT		2269.4
1010	2269.39	CL DAM GRND	5/2/16 14:18	00:02.0	Topo	CL EMBANKMENT		2269.21
1011	2269.63	CL DAM GRND	5/2/16 14:18	00:02.0	Topo	CL EMBANKMENT		2269.68
1012	2268.69	CL DAM GRND	5/2/16 14:20	00:02.0	Topo	CL EMBANKMENT		2269.87
1013	2262.48	BM CHISLED SQUA	5/2/16 14:21	00:03.0	Topo	TOP CL 12IN WALL 1		
1014	2260.5	TOP WALL	5/2/16 14:23	00:03.0	Topo	TOP CL 12IN WALL 2		
1015	2258.61	TOP WALL	5/2/16 14:38	00:02.0	Topo	TOP CL 12IN WALL 3		
1016	2253.05	INV LOW LEVEL	5/2/16 14:44	00:02.0	Topo	INV LOWLEVEL		
1017	2249.9	TOP OUTLET PIPE	5/2/16 14:46	00:03.0	Topo	TOP 36IN RCP		
1018	2246.11	INV OUTLET PIPE	5/2/16 14:48	00:02.0	Topo	INV 36IN RCP		
1019	2269.41	CL DAM GRND	5/2/16 14:54	00:01.0	Topo	CL EMBANKMENT		
1020	2269.96	CL DAM GRND	5/2/16 14:55	00:01.0	Topo	CL EMBANKMENT		
1021	2271.37	BM CHISLED SQUA	5/2/16 15:20	00:02.0	Topo			
1022	2299.43	16310135DCB 2IN	5/3/16 10:05	00:02.0	Topo			
1023	2299.6	16310135DCB 4IN	5/3/16 10:06	00:02.0	Topo			
1024	2297.52	16310135DCB COI	5/3/16 10:06	00:02.0	Topo			

APPENDIX C. Cost of Alternatives.

ALTERNATIVE B - Repair principal spillway and replace emergency spillway					
Item No.	**Item Description**	**Quantity**	**Unit**	**Unit Price**	**Total Cost**
CIPP					
1	Mobilization/Demobilization @ 10%	1	Lump Sum	$ 5,632	$ 5,632
2	Clearing and Grubbing	1	Lump Sum	$ 5,150	$ 5,150
3	30" CIPP	100	Feet	$ 386	$ 38,600
4	Dewatering	1	Lump Sum	$ 8,755	$ 8,755
5	Storm Water Management	1	Lump Sum	$ 1,236	$ 1,236
6	Material Testing	1	Lump Sum	$2,575	$ 2,575
7	Unlisted Items @ 5%				$ 3,098
	Subtotal Construction Costs				**$ 65,000**
RCC					
1	Mobilization/Demobilization @ 10%	1	Lump Sum	$ 49,000	$ 52,000
2	Dewatering	1	Lump Sum	$ 10,000	$ 10,000
3	Sediment and Erosion Control	1	Lump Sum	$ 2,662	$ 2,662
4	Traffic Control	1	Lump Sum	$1,330	$ 1,330
5	Clearing and Grubbing	6	Acres	$5,500	$ 33,000
6	Reclamation of Disturbed Areas	3	Acres	$ 8,000	$ 24,000
7	Slope Protection of Median	416	Square Yard	$ 10	$ 4,160
8	Soil Excavation	3,453	Cubic Yard	$ 5	$ 17,000
9	RCC Placement	2,691	Cubic Yards	$ 142	$ 382,000
10	Bedding Material	635	Cubic Yard	$ 71	$ 45,000

11	Unlisted Items @ 5%				$ 29,000
	Subtotal Construction Costs				**$ 601,000**
	Contingency @ 30%				$ 200,000
Total Estimated Construction Cost					**$ 866,000**

ALTERNATIVE C - Remove principal spillway and replace emergency spillway					
Item No.	**Item Description**	**Quantity**	**Unit**	**Unit Price**	**Total Cost**
RCC					
1	Mobilization/Demobilization @ 10%	1	Lump Sum	$ 49,000	$ 52,000
2	Dewatering	1	Lump Sum	$ 10,000	$ 10,000
3	Sediment and Erosion Control	1	Lump Sum	$ 2,662	$ 2,662
4	Traffic Control	1	Lump Sum	$1,330	$ 1,330
5	Clearing and Grubbing	6	Acres	$5,500	$ 33,000
6	Reclamation of Disturbed Areas	3	Acres	$ 8,000	$ 24,000
7	Slope Protection of Median	416	Square Yard	$ 10	$ 4,160
8	Soil Excavation	3,453	Cubic Yard	$ 5	$ 17,000
9	RCC Placement	2,691	Cubic Yards	$ 142	$ 382,000
10	Bedding Material	635	Cubic Yard	$ 71	$ 45,000
11	Unlisted Items @ 5%				$ 29,000
	Subtotal Construction Costs				**$ 601,000**

Removing Conduit					
12	Mobilization/Demobilization @ 10%	1	Lump Sum	$ 6,600	$ 21,000
13	Conduit Soil Excavation	12,760	Cubic Yard	$ 8	$ 102,000
14	Compacted Embankment Fill	12,760	Cubic Yard	$ 7	$ 89,000
15	Demolition of conduit	100	Feet	$ 39	$ 4,000
16	Clearing and Grubbing	2	Acres	$5,500	$ 11,000
17	Reclamation of Disturbed Areas	1	Acres	$ 8,000	$ 8,000
18	Unlisted Items @ 5%				$ 12,000
	Subtotal Construction Costs				**$ 247,000**
	Contingency @ 30%				$ 254,000
Total Estimated Construction Cost					**$ 1,102,000**

ALTERNATIVE D - Dam Decommissioning					
Item No.	**Item Description**	**Quantity**	**Unit**	**Unit Price**	**Total Cost**
1	Mobilization/Demobilization @10%	1	Lump Sum	$ 38,000	$ 38,000
2	Sediment disposal	1	Lump Sum	$ 138,000	$ 138,000
3	Site Restoration	1	Lump Sum	$ 41,000	$ 41,000
4	Soil Excavation	25,000	Cubic Yard	$ 8	$ 200,000
5	Unlisted Items @ 5%				$ 21,000
	Subtotal Construction Costs				**$ 438,000**
	Contingency @ 30%				$ 131,000
Total Estimated Construction Cost					**$ 569,000**