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Appalachian Dam Design: Hydrualics and Hydrology

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INTRODUCTION

Clay County lacks water in many of it rural areas. The spring 2014 Civil and Environmental Engineering senior design course was approached by the county to develop the design of a reservoir that could supplement the current water supply and provide recreational tourism to the area. The goal of designing the reservoir is to develop a cost estimate and present it to Clay County in hopes that federal funding will be provided to construct the dam. A team of 17 students divided into sub-disciplines that included: Management, Geotechnical, Structural, Transportation, Construction, Water Resources, and Environmental. Students from the team designed the water resources portion of the dam. I was part of water resources design team, and for this report, I will be discussing my contributions to the project.

All designs from a hydraulic or hydrology standpoint must assure the dam for the reservoir is appropriately sized for any storm condition, and that the spillways can adequately transport water from the reservoir to the other side of the dam. For dam design, there is an iterative process that involves several spreadsheets combined with a HEC-HMS model. HEC-HMS is a program that analyzes a watershed basin and meteorological data, and calculates an output flow.

DAM COMPONENTS

Two spillways, the principal and emergency spillway, are necessary for the proper functioning and safety constraints for this dam. The purpose of the principal spillway is to keep the reservoir at a constant depth, when not under heavy storm conditions. Anything over a 100-

year storm flows into the emergency spillway, which exists so that the earthen dam is not overtopped. There are five important elevations that must be determined in the design and analysis of the dam: sediment elevation, normal pool depth, emergency spillway crest, maximum design flood level, and dam crest. The sediment elevation, assumed to be 10 feet (ft.) in elevation, accounts for 100 years of sediment deposit from runoff and streams. This volume is neglected from the reservoir's water storage in any calculations to account for a condition when the reservoir has accumulated the maximum amount of intended sediment. The next elevation is the principal spillway, which is located at normal pool depth. When the water elevation rises above normal pool depth, water flows into the principal spillway, eventually discharging downstream of the dam. Water will continue to flow through the principal spillway until the water level returns to normal pool depth. If the storm is severe, the water level will rise to the emergency spillway crest, which is located above the principal spillway. The emergency spillway is only used in events that are more severe than the 100-year storm. The maximum design flood level is a theoretical water elevation that represents the absolute highest water elevation in the reservoir attainable by rainfall on the watershed. Just above the maximum design flood level is the dam crest, or top of the dam.

DESIGN PROCEDURE

For the design approached used, the principal spillway elevation is initially assumed, determining a normal pool elevation. Then a 100-year storm is modeled in HEC-HMS, resulting in a rise in water elevation above the normal pool depth. At an elevation just above the highest water elevation for a 100-year storm is the emergency spillway crest. The last storm that is modeled is the probable maximum precipitation (PMP) storm. This is a theoretical storm that

represents the absolute maximum amount of rain that can impact the watershed given the ideal environment conditions. Kentucky requires that all dams be able to withstand the PMP storm without failing (i.e. overtopping). The PMP storm is modeled using HEC-HMS, and in a similar manner, the water elevation will rise above the emergency spillway crest. The highest elevation the water reaches during the PMP storm is the maximum design flood level. From the maximum design flood level a freeboard is added resulting in the required dam crest elevation.

DESIGN INPUT PARAMETERS

The software, HEC-HMS, used to design these elevations requires thousands of parameters, which can be divided into basin models, meteorologic models, control specifications, time-series data, and paired data. Basin inputs incorporate all watersheds, reaches, and reservoirs within the basin into an interactive network. Each piece requires inputs such as curve number, watershed area, reach length and cross-section, and reservoir initial water elevation. The basin model was divided into 7 sub-basins, 2 reaches, 3 junctions, and 1 reservoir, as displayed in Figure 1. The meteorologic model replicates a particular storm event onto the basin model. The storm event is a measurement of depth of rainfall. The meteorological models included a 4.6 inch 100-year, 6 hour storm and a 28.6 inch, 6 hour PMP storm. Control specifications determine when the simulation starts and stops. A date and time is entered for the starting and stopping point and all simulations were run over a simulated 30 day period.

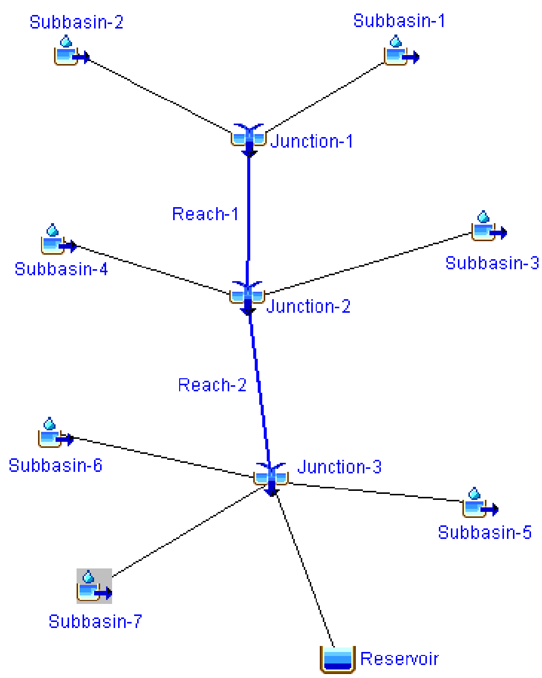

Figure 1. HEC-HMS basin model overview of the watershed.

Time-series data is a very important part of HEC-HMS. Due to the two meteorologic models being 6 hour storms, the typical 24-hour hyetograph needed to be compressed into a 6-hour hyetograph. Through spreadsheet calculations, the hyetograph was altered as needed, and then transferred into the time-series data. The meteorologic data then follows the time-series data as to the depth of rain that falls per each 15 minutes of the storm.

The basin model also requires paired data to accurately model elevation determined data such as area and discharge. Elevation-area and elevation-discharge curves were developed through spreadsheets and transferred into the paired data in HEC-HMS. Contour maps on GIS were used to determine incremental areas of the reservoir for every 20 ft. of elevation. When a simulation runs, the basin model pulls from its assigned paired data to carry out elevation

dependent calculations. HEC-HMS encompasses all aspects of the watershed, reservoir, and outflows, and produces various output analysis data for a given storm event.

The spreadsheet used to develop the elevation-discharge curve was complex in that it had dozens of variables. Water flowing from the principal spillway, emergency spillway, and overtop the dam were all accounted for in the elevation-discharge relationship. The principal spillway was specified as a 2-ft. pipe orientated vertically, to reduce the required design iterations. The circumference of the pipe acts as either a weir or an orifice, depending on the water elevation. Initially, the pipe acts as a weir, with the pipe circumference as the weir length. Eventually, the outlet will be governed by the orifice properties. This occurs when the flow through the inlet using the orifice equation is less than the flow using the weir equation. Another component that could potentially govern was the friction occurring in the pipe that transports the water from the principal spillway to the other side of the dam. An energy equation was used to determine if the friction in the pipe under the dam limited its discharge to an extent that the flow would be decreased. It was discovered that this was not the case, and that initially the weir, and then the orifice condition governed flow into the principal spillway.

In addition to the principal spillway, analysis of the reservoir was contingent upon the two broad-crested weirs: the emergency spillway and any water flowing over the dam. Two separate elevation-discharge relationships were developed for the emergency spillway and dam top, and then they were added to the principal spillway to create one final elevation-discharge relationship. The dam was designed so that the water will never exceed the dam crest. However, it was necessary to add discharge going over the dam to theoretically predict what the flow would be if the water elevation does in fact exceed the dam crest.

METHODOLOGY

The iterative process involving the elevation-discharge spreadsheet and HEC-HMS was essential to the design of the dam. The elevation-discharge spreadsheet was set up so that it referenced variables for the orifice, weir, and energy equations, allowing for a more efficient iteration process. When variables needed to be manipulated, the elevation-discharge curve automatically accounted for the changed variables and produced a new curve. The new curve was then input into HEC-HMS, the simulation was re-analyzed, and the model produced new outputs. These outputs were analyzed, adjustments were made to the elevation-discharge spreadsheet, and the process is restarted. Such iterations continue until the goal of having the highest normal pool elevation with the maximum design flood level being 200 ft. above the bottom of the dam is achieved.

Discharges leaving the principal and emergency spillways were analyzed to see if they would be destructive to the dam infrastructure and downstream channel. If flows through the spillways are too great, infrastructure could be at risk, and it could cause the entire dam to fail. For example, if the emergency spillway exceeds 20 feet per second (fps.), then it has the shear strength to rip up the concrete bed. The emergency spillway must be designed with a weir length that will decrease flow to a point that concrete bed will not fail. Also, flow through the principal spillway must also be considered. There is an elbow that transfers the discharge from the emergency to the exit pipe under the dam. When there is pressurized flow hitting an elbow, a resultant force must be accounted for. As the discharge increases, the resultant force will also increase. To prevent any destructive force from the bend, a smaller orifice was used at the entrance of the principal spillway. The flows through the spillways are a function of their individual sizes and their elevation relative to the normal pool elevation and to one another. The

known constraints of the flows allowed for fewer iterations in HEC-HMS, resulting in a design that could be supported by the dam infrastructure.

DESIGN SUMMARY

The final reservoir analysis achieves the highest normal pool level elevation possible for recreational use while maintaining the maximum design flood level not to exceed 200 ft. above the bottom of the reservoir. All of the design elevations are shown in Table 1. Referenced from the bottom of the reservoir, the normal pool elevation is at 183 ft. and the maximum design flood level is at 199.4 ft as shown in Figure 2. The volume between sediment level and normal pool depth is the useful storage, or the amount of water in storage that is available for use. The sediment volume and useful storage are shown in Table 2.

The output from the storm events simulated in HEC-HMS are shown in Table 3. The HEC-HMS simulation of the 100-year, storm resulted in a maximum water elevation of 0.9 ft. above the normal pool level. This storm is contained single handedly by the principal spillway. The emergency spillway crest is an additional 8 ft. above what the 100-year storm elevation, which accommodates appropriate design standards of practice. Having a greater distance between the emergency spillway crest and the 100-year storm elevation allows more water to be stored during the PMP storm. Therefore, less water exits the emergency spillway when storms exceed the 100-year storm. This critical deviation from typical dam designs decreased the flow into the emergency spillway during the PMP storm to a velocity less than 20 fps. Concrete energy dissipaters are located at the bottom of the emergency spillway to further decrease the velocity of the flow before it enters the stream below the dam.

The principal spillway has an inlet pipe diameter of 2 ft., which expands to the main pipe diameter, which is larger and able to decrease friction losses. After the elbow at the base of the principal spillway, the flow enters into an even larger pipe that leads under the dam. All of the pipe and weir sizes are shown in Table 4. Along this pipe are anti-seep collars which prevent subsurface flow from traveling a preferred path along the pipe and potentially causing seepage failure of the dam.

Table 1. Dam component and storm event water level elevations.

	Relative to Dam Bottom *FT*	**True Elevation** *FT*
Dam Bottom	0	920
Sediment	10	930
Principal Spillway	183	1103
Normal Pool Elevation	183	1103
100-Year, 6 Hour Pool Elevation	183.9	1103.9
Emergency Spillway	192	1112
Max Design Flood Level (6 Hour PMP Storm)	199.4	1119.4
Top of Freeboard	220	1140

Table 2. Sediment storage and useful water storage volumes.

Height Differences	**Elevation Range** *FT*	**Height** *FT*	**Volume** *FT^3*	**Volume** *ACRE-FT*
Sediment Deposition	920 - 930	10	6889957	158
Useful Storage	930 - 1103	173	958165249	21996

Table 3. Flows of principal and emergency spillways for the storm events.

Storm Event	**Max Discharge (CFS)**	
	Principal Spillway	**Emergency Spillway**
100-Year, 6 Hour	12.8	0
PMP, 6 Hour	62	1256

Table 4. Pipe and weir sizes.

Outlet Pipe Diameter	8	*FT*
Dam Drain Orifice Diameter	6	*FT*
Principal Spillway Diameter	4	*FT*
Principal Spillway Orifice Diameter	2	*FT*
Emergency Spillway Weir Length	20	*FT*

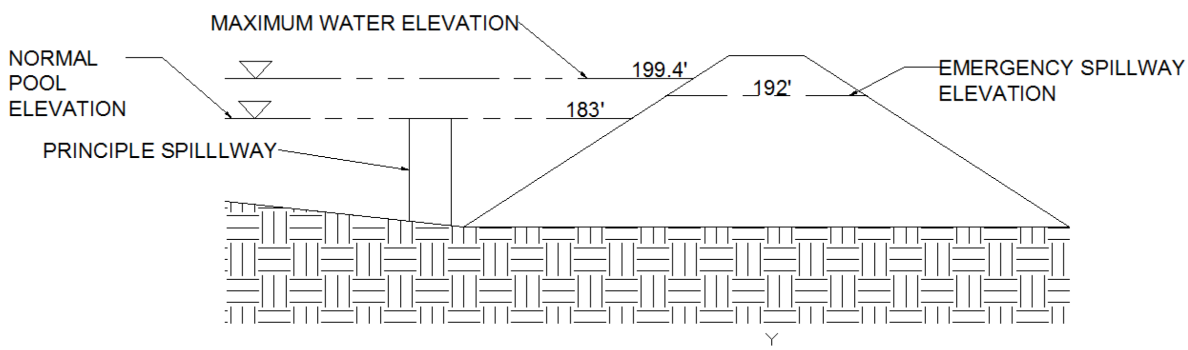

Figure 2. Elevations of the various components of the dam.