Boysen Reservoir Pump Storage Facility

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DISCLAIMER

This report was written to fulfill the course requirement for CE 4900, Senior Design in Water Resources, at the University of Wyoming during the spring 2016 semester. This work was created by students and should not be used by anyone for any purpose. This project is solely a student exercise.
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Overview of the Project Scope and Objectives

The purpose of this project was to fulfill the design criteria for the CE 4900 Senior Design in Water Resources class at the University of Wyoming. The assigned project was to design a renewable energy pumped-storage facility that includes a hydroelectric dam to create green electricity by emptying an upper reservoir into a lower reservoir. During the early hours of morning when the energy consumption is low, as shown in Figure 1, the upper reservoir will be pumped full from the lower reservoir, using energy that is not being used and cannot be stored efficiently. The main idea of this project was to provide people with green electricity while utilizing excess electricity that would otherwise be wasted.

For this project Team D proposed a renewable energy pumped-storage facility approximately 20 miles upstream from Thermopolis, Wyoming, utilizing Boysen Reservoir as the lower reservoir in the design. The design consist of the additional upper reservoir right next to the existing Boysen Reservoir. The new upper reservoir will be pumped from the Boysen Reservoir
during the low energy demand period, and then during peak energy usage the water will flow through the penstock in the dam, into a Francis Turbine, where the energy will be generated and collected. Once the reservoir is recessed down one quarter of its overall capacity, the facility will then begin to recharge the upper reservoir using Boysen Reservoir’s water supply. This will take place during the time of low electricity usage which is generally between the hours of 12 am and 8 am, allowing eight hours of recharge.
Project Location

The proposed location for this project is towards the lower end of Boysen Reservoir in central Wyoming. The large amount of water available within Boysen Reservoir encouraged the choice to use this location. Around Boysen Reservoir there exist several canyons and other natural bowl shapes that are ideal for holding water. Boysen Reservoir will act as the afterbay for the project, and the new reservoir indicated in pink in Figure 2 will act as the forebay. Boysen

Figure 2: Overview of the location of the proposed pump storage facility.
Reservoir holds 150,000 acre-feet of water, which is more than enough to supply the designed upper reservoir which will hold about 1,975 acre-feet of water, which was estimated from a digital elevation model (DEM) using ArcGIS. The designed reservoir covers approximately 42 acres of land, and at the deepest point will be about 249 feet deep.

The location that was chosen for the proposed upper reservoir was decided upon by the relative closeness to Boysen Reservoir. This location minimized the amount of piping to convey the water to Boysen reservoir during times of energy production and to convey water back to the upper reservoir during periods of recharge, the shorter distance results in a small friction energy loss which is important to the design and functionality of the project.
The watershed delineation for the proposed reservoir is shown in Figure 3 in green. The watershed covers approximately 191 acres, of which one hundred percent will drain into the purposed reservoir. From the watershed delineation created using ArcGIS, data was exported into MatLab in order to run a script that returned data relative to an elevation storage curve. That data was placed into Microsoft Excel where the curve exhibited in Figure 4 was constructed. From this curve the storage was estimated, compared, and confirmed with the ArcGIS drawing to insure correct data was being utilized. In order to increase the total capacity of the upper reservoir, two earth filled dams will be put in place, as shown in Figure 3, on the perimeter of the reservoir. In designing the earth fill dams a slope of 1 to 3 was utilized for the sides and a level crest of approximately 20 feet wide was speculated. To account for this design it was determined that the earth fill dams would require an estimated 24,000 cubic yards of fill for the both of them to be constructed. The volume of fill was calculated using a shrinkage factor of 20% to accommodate for high compaction to increase stability and insure safe slopes that will not be washed out from the pressure of the upper reservoir. To insure that the earth fill dams will not seep or become saturated and unstable a clay liner will be placed along the
inside slope of the dam. Riprap will also be places along the upper portion of the inside slope of the dam to prevent any erosion that may occur due to surface waves on the reservoir. Without these dams the facility would not have enough capacity to function in an efficient manner, therefore the earth fill dams were adequately and thoroughly designed to reduce any safety or structural issues in the future. The final design of the new reservoir will hold 1,975 acre feet amounting to 2,436,123 cubic meters of water.

**Dam Location and Design**

Figure 5 illustrates the layout of the facility that was designed relative to the layout of the land. The forebay will be contained by a concrete gravity dam at the southeast side of the reservoir. The dam will span the valley at a length of 876 feet. The turbines and the penstock will go through the dam and empty into Boysen Reservoir. The dam will be constructed from concrete which will be reinforced with steel at 5% of the total volume of concrete. By using a

*Figure 5: An overview of the layout of the project in relation to the contours of the land and Boysen Reservoir.*
gravity dam, the largest cost contribution will be the large amount of concrete that will be required to adequately construct the dam. A gravity dam was selected due to the large span of valley in which the dam occupies. An earthen fill dam was considered but was quickly ruled out due to the large amounts of imported soil needed to construct the dam. It was also determined that the side slopes of the valley were not rock and the valley proceeds to become wider below the dam indicating a stout and structurally sound dam was necessary resulting in the best option being a gravity dam. Based on the dam span and size, it was estimated that the dam will need 1,365,120 cubic yards of concrete. The dam will be roughly 30 feet above the water surface making the total height of the dam 250 feet. This additional height is to account for any flood conditions that may occur.

**Historical, Cultural, & Environmental Concerns**

This section examined the sustainability of the proposed project. Historical, cultural, and environmental aspects were key to the selection and design of the project. The location of the dam, off of Boysen Reservoir falls within the boundary of the Wind River Indian Reservation, so all historical, sacred, and Indian Burial sites were avoided for the design of the forebay reservoir. Figure 6 shows the location of Boysen Reservoir in the red square, and it is noticed that the location is on the edge of the Wind River Reservation.
The Wyoming cultural aspect of hunting, fishing, and camping were assessed to determine the best location for the reservoir. Boysen Reservoir holds the record for the largest walleye caught in Wyoming, so a prime consideration was to keep the environment as natural as possible.

Using the U.S. Fish and Wildlife Service, the area around Boysen Reservoir could potentially impact three endangered species. The *Spiranthes diluvialis* is a threatened flower plant in the area. To minimize the impact, these flower will be removed during the duration of the project and will be replanted after construction. The black footed ferret, and the gray wolf have been recognized as endangered species, but are in experimental populations with no critical habitat.
designated for either of the species. The greater sage grouse is also known to occupy this region. Although it is no longer listed as an endangered species, the land will be returned to natural prairie after construction. Twenty-four migratory birds are protected under the Migratory Bird Treaty Act and the Bald and Golden Eagle Protection Act. The birds are protected for either year round or for breeding seasons. To accommodate these birds, disturbed habitat will be return to its natural state after the construction of the reservoir and dam system. Nests will be located and will not be disturbed if at all possible. Impacts to wetlands will be minimized and the regulations under Section 404 of the Clean Water Act will be followed. The created reservoir will comply with wetland standards set up by the U.S. Army Corps of Engineers. Fish screens will be implemented to keep fish from being sucked into the turbines or pumps of the system. Penstocks will be buried underground with the aboveground being returned to its natural conditions.

Hydrology

Hydrology of the area is important to determining the capacity of the spillway for the dam. The dam located at the outflow of the reservoir will have excess runoff during storms. During major storms, the reservoir and dam must be able to accommodate the extra water and release it through its spillway. The maximum flood can be determined for any length of time, but to create the most reasonable number a 100 year flood over a 24 hour period was collected from the Wyoming Climate Atlas and a maximum flood of 3 inches in a 24 hour period was determined. Figure 7 shows the map used with the location of the reservoir highlighted in the red box.
The probable maximum precipitation (PMP) is the theoretical largest depth of precipitation for a given duration that is possible over a specific drainage area. The PMP for the area was determined from Figure 8 provided by the Wyoming Water Development Commission. Again the red square is the location of the proposed reservoir and dam. Using this figure a PMP of 10”-12” was determined for a 24 hour period.
The probable maximum flood was determined by the unit hydrograph described in the Design of Small Dams: Bureau of Reclamation, 1987. The unit hydrograph theory is as follows, paraphrased from Design of Small Dams.

The basic concept of the unit hydrograph theory can be understood by considering a situation in which a storm of, say, 1-hour duration produces rainfall at a constant rate, uniformly over the drainage basin above a recording stream gauging station. Assume that the rainfall rate is such that 1 inch of the total rainfall does not infiltrate into the soil, but runs off over the ground surface to tributary watercourses, eventually arriving at the stream gauging station. The runoff at the gauging station will be recorded to form a hydrograph representing the temporal distribution of discharge from 1 inch of ‘rainfall excess’ occurring in 1 hour. This hydrograph is the ‘1-hour unit hydrograph’ for the drainage basin tributary to the gauging station. The unit hydrograph in this case is said to have a ‘unit duration’ of 1 hour.

Letter Symbols for Hydrology Calculations

\[ L_g = \text{unit hydrograph lag time, in hours} \]
\[ K_n = \text{Manning’s N} \]
\[ L = \text{Length of longest watercourse} \]
\[ L_{ca} = \text{Length of longest watercourse opposite centroid of drainage basin} \]
\[ S = \text{overall slope along } L \]
\[ t = \text{time} \]

\[ L_g = 26K_n \left( \frac{L + L_{ca}}{S^{0.5}} \right)^{0.33} \quad \text{Equation 1} \]

Using a Manning’s N of 0.05, determined from a topography as a floodplain with scattered brush and heavy weeds, a lag time of .500 hours was calculated. Converting this into time using
Equation 2 from the *Design of Small Dams*, a time of 00.09 hours or 5.45 minutes was calculated. This number is rounded up to 5 minutes.

\[ t = \frac{L_a}{5.5} \]  

Equation 2

Continuing with the process outlined in *Design of Small Dams*, and using the Table 1, taken from *Design of Small Dams*, for the Rocky Mountain Region, Table 2 was created using Excel with the specific data for the Region.

*Table 1 Rocky Mountains, General Storm Dimensionless S-Graph Data.*

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<th>Discharge, Time, % of $L_u$</th>
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Table 2: Unit Hydrograph Data

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From this graph, the columns of Time, 5 of lag was plotted on the x axis while, the Unit Hydrograph column was plotted on the y axis to create the unit hydrograph as shown in Figure 9.
Penstocks

Letter Symbols for Penstock Calculations

- $Q =$ Flow Rate (cfs)
- $T =$ time facility will run (hr)
- $\Delta Storage =$ Change in storage volume (ft$^3$)
- $h_L =$ Headloss (ft)
- $V =$ Velocity (ft/s)
- $g =$ Gravity (ft/s$^2$)
- $L =$ Length (ft)
- $f =$ friction factor
- $D =$ Diameter of Penstock (ft)
- $K_e =$ Entrance Loss (ft)
- $K_v =$ Valve loss (ft)
- $h_f =$ Friction headloss (ft)
- $RE =$ Reynolds Number
- $v =$ Viscosity of water
- $K_s =$ Constant for steel pipe
- $P =$ Pressure
- $h_p =$ Headloss of pump
- $h_t =$ Headloss of turbine
- $\Upsilon =$ Unit weight of water
- $z =$ Elevation

Figure 9: Unit Hydrograph
The total flow through the penstocks for the facility is 740 cubic feet per second (cfs). This was obtained using the storage elevation curve (Figure 4) and Equation 3 below.

\[ Q = \frac{\Delta \text{Storage}}{T} \]  

Equation 3

There will be two penstocks for the whole system, one for turbine operation and the other for pump operation. The location of the penstocks was determined by the shortest route from the upper to the lower reservoir. There will be one penstock for the turbine operation, directing flow from the upper reservoir into two parallel turbines to produce hydroelectricity. This penstock will go through the middle of the dam and have a length of 250 feet. Located to the left will be one penstock for the pumping operation that will refill the upper reservoir. This penstock will be longer with a length of 600 feet, allowing it to reach out to deeper water. Figure 10 shows the penstock location.
Headloss for the system was found using Equation 4. Each penstock has a different headloss. In order to get an accurate headloss entrance and valve losses were considered.

\[
\Sigma h_L = \frac{v^2}{2g} \left( f_L \frac{Q^2}{D} + K_e + K_v \right)
\]  \hspace{1cm} \text{Equation 4}

In Table 5-3 in the *Hydraulic Engineering* book it is found Ke = 0.5 and Kv = 0.2. The penstock for the turbine operation has a headloss of 1.3 feet and for the pumping operation the headloss is 1.8 feet.

The diameter of the penstocks was found using Equation 5 obtained from *Hydraulic Engineering*.

\[
D^5 = \frac{f_L Q^2}{0.785^2 \times (2gh_f)}
\]  \hspace{1cm} \text{Equation 5}

This equation is an iterative process. First, a friction factor was assumed and the diameter is found. This allowed the calculation of the Reynolds number using Equation 6 and relative roughness using Equation 7.

\[
RE = \frac{VD}{v}
\]  \hspace{1cm} \text{Equation 6}

\[
Relative \text{ Roughness} = \frac{K_s}{D}
\]  \hspace{1cm} \text{Equation 7}
Obtaining a Reynolds Number (RE) equaling $7.72 \times 10^6$, it is known that the flow is turbulent. With the RE, and the relative roughness equaling 0.00003 it was determined to find the actual friction factor ($f$) using Figure 11.

![Moody Diagram](image)

*Figure 11: Moody Diagram.*

Obtaining an actual friction factor of 0.0095, the facility requires a pipe diameter of 10 feet for both the turbine operation and pumping operation.

The pressure in each penstock was determined using Equation 8.

$$\frac{P_1}{\gamma} + \frac{V_1^2}{2g} + Z_1 + h_p = \frac{P_2}{\gamma} + \frac{V_2^2}{2g} + Z_2 + h_t + h_L$$  \hspace{1cm} \text{Equation 8}

This equation was manipulated solving for $P_2$ for each penstock. The turbine operation penstock has a pressure of 38 psi and the pumping operation penstock has a pressure of 120 psi. With these high pressures, steel pipe is the best option, therefore, a 1.25 inch thick steel
pipe for this facility will be utilized. Table 3 shows the specifications of both the turbine and pump operation penstocks.

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<th>Penstock to Pump Operation</th>
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<td>( f ) 0.0095</td>
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<tr>
<td>( L )</td>
<td>( L ) 600 ft</td>
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<tr>
<td>( Q )</td>
<td>( Q ) 740 cfs</td>
</tr>
<tr>
<td>( V )</td>
<td>( V ) 9.42 ft/s</td>
</tr>
<tr>
<td>( D )</td>
<td>( D ) 10 ft</td>
</tr>
<tr>
<td>( RE )</td>
<td>( RE ) 7.72E+06</td>
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<td>( hL )</td>
<td>( hL ) 1.8 ft</td>
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**Intake Structure**

For the turbine operation, the intake structure will be attached to the dam as seen in Figure 12. A trash rack will be placed on the intake structure so that there is no potential for the penstock to clog. Since the upper reservoir is going to be newly constructed, no fish should be in it; therefore, fish considerations can be eliminated.
For the pumping operation, the intake structure will be a simple concrete structure as shown in Figure 13. It will include a fish screen, as there will be fish considerations pumping from Boysen to the upper reservoir. The fish screen will act as a barrier so that fish are not able to get to the upper reservoir.
Cold Weather Considerations

Winter conditions in this area should be considered in the design. It is crucial that cold weather considerations are taken into account as winter conditions typically begin in November and last through April and sometimes even into May. These conditions amount to 50% of the year; therefore, by keeping these extreme conditions in mind prevention of economic loss, safety, and possible failure of the facility during this time can occur.

The inlet and outlet of both penstocks are the areas of concern when considering cold weather. One particular concern is problems that occur due to frazil as shown in Figure 14.

![Figure 14: Accumulation of Frazil Ice](image)

Frazil is defined as ice forms that accumulate around trash racks at the inlet and outlet of penstocks. Blockages can lead to overflow and further jamming of the pipes. According to the technical report *Frazil Ice Concerns for Channels, Pump-Lines, Penstocks, Siphons, and Tunnels in Mountainous Regions*, by Robert Ettema and Gokhan Kirkil, the problems with frazil ice are caused by the changes in freezing temperature associated with the substantial pressure...
changes that water flows in pressurized conduits may undergo. The principle factor to be considered in this regard is the depression of freezing temperature as water pressures increase. Figure 15 illustrates the phase diagram of water. As the pressure increases, water’s freezing point decreases (Ettema and Kirkil, 2007).

![Phase diagram of water with respects to pressure and temperature](image)

*Figure 15: Phase diagram of water with respects to pressure and temperature (Ettema and Kirkil, 2007).*

In the technical report by Ettema and Kirkil, it states that pressure changes of the order of 0 to 2 mega pascal (MPa) magnitude are common in closed conduit flows associated with hydropower penstocks. An elevation difference of 100 meters produces a pressure difference of about 1 MPa.

Considerations for cold weather temperatures affect the turbines as well. As the water emerges from the penstock and passes through the turbine, its pressure decreases abruptly, with the consequent increase in freezing temperature, and the flow of super-cooled water out
of the turbine (Ettema and Kirkil, 2007). When that occurs, frazil concentrations may eventually accumulate back up into the turbine leading to choking of the flow through it. Figure 15 illustrates frazil formation and accumulation within a turbine and its outlet.

Figure 16: Frazil formation and accumulation within a turbine and its outlet (Ettema and Kirkil, 2007)

As frazil ice accumulates, it decreases the flow of water through the penstocks leading to the turbine, which can decrease the efficiency of the facility and may not produce enough energy to meet our goal. Figure 17 shows a map of the frost depth through the US. The location of interest has a frost depth of 30 inches, therefore, to decrease the effect of super-cooling, the pipes will be buried at 48 inches.
Cold weather also affects the pump. When frazil forms in pump lines, water pressure rises to head $H_p$, then reduces along the hydraulic grade line (HGL) to the equivalent of the flow outlet, as seen in Figure 18. The freezing temperatures drop correspondingly as the HGL rises above the pipe, causing the water potentially to be super-cool and the ice to melt (Ettema and Kirkil, 2007).
In order to mitigate the problems with Frazil ice the inlet will need to be at an elevation that won’t freeze, in both the inlet and outlet of the penstocks. For this design it can be tricky because the water elevation will always be changing in the upper reservoir. During turbine operation a quarter of the upper reservoir will be drained. This changes the water elevation by 12 feet. Figure 19 shows a cross section of the facility. As seen in Figure 19, the inlet needs to be installed at least 16 feet below the capacity water elevation in the upper reservoir. In order to have sufficient active storage, the inlet will be at half the dam height. Since Boysen Reservoir is much larger compared to the proposed upper reservoir, the water elevation will not make a significant change, therefore, the outlet into Boysen reservoir only needs to be installed 48 inches below the water surface.

*Figure 19: Cross Section showing placement of the inlet and outlet, change in water elevation, and all other components of the facility.*
Turbines

Using Figure 16 from *Hydraulic Engineering* with a head of 230 feet and an electrical output roughly 3,000 kW desired, a Francis Turbine is selected as optimal.

The selection of a Francis Turbine provides a high efficiency of over 90%. Figure 21 provides the selection of the specific speed. Following the normal practice in the mid-1960s a specific speed...
of 70 was selected. The specific speed relates to the speed of rotation of a turbine which develops 1 horsepower under 1 foot of head.

**Letter Symbols for Turbine Calculations**

- $n =$ specific speed
- $P =$ power
- $Q =$ flow
- $\gamma_w =$ density of water
- $\Delta H =$ head
- $\eta =$ efficiency
- $N =$ revolutions per minute
- $D_{\text{turbine}} =$ Diameter of the Turbine

The calculation to predict the power output of the turbine is calculated from Equation 9 as found in *Hydraulic Engineering*.

$$P = \frac{Q \gamma \Delta H \eta}{550} \quad \text{Equation 9}$$

The power determined from this equation is 17613.82 horse power (hp) which converts to 14 MW of energy generated from each turbine. The rotational speed was calculated from Equation 10 from *Hydraulic Engineering*.

$$N = \frac{7200}{n} \quad \text{Equation 10}$$
A rotational speed of 103 revolutions per minute was calculated. Next the diameter of the turbine was calculated to be 16.9 feet which was rounded up to 17 feet. Using two turbines, the diameter of each turbine will be 8.5 feet which will be rounded to 9 foot diameter to meet standard turbine sizes.

\[
D_{\text{turbine}} = \frac{60 \cdot 0.75 \sqrt{2g \cdot \Delta H}}{\pi N}
\]

Equation 11

Pumps

An important feature of this project was the incorporation of pumps that will be used to recharge the upper reservoir. In determining the pumps many consideration were taken into account, such as noise, environmental impact, and efficiency. It was concluded that there was no need for low noise pumps due to the location not being close to any relative features that would be disturbed by the noise. With the recharge time less than the run time, multiple large pumps will be needed. It was important to protect the aquatic life when considering pumps, for

![Figure 22: MWI Corporation radial and mixed flow speculation graph based on total head and flow output.](image)
that reason it was decided upon to use four 60 inch pumps which would require 80 inch intakes. Based on the volume needed to recharge the reservoir from \( \frac{3}{4} \) capacity to full the design team selected pumps that will be manufactured by MWI Corporation. Using Figure 22, which was provided by MWI, the calculated head of about 40 feet was used, which is from the intake in Boysen Reservoir up to the outlet through the dam into the upper reservoir. From Figure 22, it was decided upon to use a mixed flow pump.

With a mixed flow pump chosen the team then calculated the required volume of recharge that would be needed and divided it out amongst the eight hours of recharge and concluded four 60 inch diameter pumps would be needed to recharge the reservoir. A total of 335,000 gallons per

![MIXED FLOW 60" PUMPS](image)

*Figure 23: MWI 60 inch pump performance curve.*
minute was required in order to sufficiently recharge the reservoir. The 60 Inch pumps were chosen because they are specked to provide a flow of 110,000 gallons per minute, and with four pumps running the recharge will be able to be adequately met in the eight hours of time set aside for recharging the upper reservoir. By installing four 60 inch pumps in parallel, not all four pumps have to be running at once during recharge. This allows any of the pumps to be taken out of service for maintenance or any other reason and still allow the facility to be recharged on time during each cycle. By using the 60 inch pumps the facility would be able to use them at a determined horsepower that was the most efficient for the head that was being overcome. From Figure 23, it was concluded that the pumps will be ran between 1100 hp and 1800 hp to achieve an efficiency of 85%.

Valves

Each penstock has a 10 foot diameter butterfly valve, a butterfly valve, as shown in Figure 24, is a disk that sits in the middle of a pipe and swivels sideways, to admit fluid, or upright, to block the flow completely. It is located downstream below the surge tanks.
Howell Bunger valves break up the water into a large, hollow, expanding spray and can be used in submerged applications, as seen in Figure 25. In the project, the 18 inch Howell Bunger valves are located in the bottom of the penstock.
Surge Tanks

Surge tanks are used to absorb the sudden rises of pressure such as water hammer occurring when the valves close or the pressure from the turbine over speed. Surge tank are important to keep the penstock and facility safe against pressure rising. Surge tanks are located at the downstream end and placed above each penstock and connected into them. Each surge tank is required to hold 600 cubic feet of water and withstand pressures up to 400 pounds per square inch (psi).

Letter Symbols for Turbine Calculations

To find the size of the surge tanks simple hydraulics calculations had to be conducted. First the required pressure that each tank would have to hold needed to be found. By finding the initial pressure \( P_1 \) on the system and multiplying the water head \( h \) with the specific weight \( \gamma \) of water following Equation 12.

\[
P_1 = \gamma \ast h \quad \text{Equation 12}
\]

During a water hammer event \( P_2 \), the water hammer equation was used is the change in head height \( \Delta H \) and the change of velocity \( \Delta V \) during the event due to the closing of the valves times the speed of sound of water \( c \) over the force of gravity \( g \). \( \Delta H \) was found to be 195 feet following Equation 13.

\[
\Delta H = \Delta V \ast \left(\frac{c}{g}\right) \quad \text{Equation 13}
\]
Finding $P_2$ is the determining factor for the surge tank, by multiply $\Delta H$ with the specific weight ($\gamma$) of water and then $P_1$ was added. Using Equation 14 $P_2$ was found to be 450 psi.

$$P_2 = (\Delta H \times \gamma) + P_1 \quad \text{Equation 14}$$

The initial volume ($V_1$) before valves closes is measured, and ($V_2$) during a water hammer event is known. The difference between $V_1$ and $V_2$ is the total water the surge tank will be taking during the water hammer event. The difference in volumes was calculated next by taking the flow rate ($Q$) times the total time for an event ($t$). Variable $t$ was found to be 8 seconds using the Equation 15.

$$t = \frac{(2 \times L)}{c} \quad \text{Equation 15}$$

$$V_1 - V_2 = t \times Q \quad \text{Equation 16}$$

A second equation, Equation 17, was needed to solve the previous equation, Equation 16. It was assumed relating to compression of the air in the surge tanks using Equation 17.

$$P_1 \times V_1 = P_2 \times V_2 \quad \text{Equation 17}$$

By solving those two equations, a volume of 705 cubic feet of water was determined.

To find the thickness of the surge tanks wall Equation 18 was used and the thickness of the wall was determined to be 1.25 inches.

$$\text{Thickness} = \frac{P_2 + D}{1.66 + \sigma} \quad \text{Equation 18}$$
The gravity dam design allows for the spillway to flow over the dam itself, rather than a side drainage system. The spillway will be designed for a 200-year flood. The total area of the watershed was found to be 191.4 acres. Using the unit hydrograph from the watershed the maximum flow was determined. Using this flow rate, the Ogee Crest head pressure is determined with the following Equation 19.
Letter Symbols for Spillway Calculations

Q = Flow rate (cfs)  
C = Discharge Coefficient  
L = Length of spillway crest (ft)  
H = Head pressure at Spillway crest  
n = Constant from Figure NUMBER  
H = Total head

X = Horizontal coordinate on ogee curve  
Y = Vertical coordinate on ogee curve  
K = Constant from Figure NUMBER  
S = slope of spillway  
AR = Hydraulic Radius  
n = roughness coefficient

\[ Q = C \cdot L \cdot H^{\frac{3}{2}} \]  

Equation 19

Figure 27: Discharge coefficients for vertical-faced ogee crest, Design of Small Dams
An assumption is made that the velocity of the water in the reservoir is zero. This means that the velocity that is approaching the spillway will also be zero. Using Figure 27 from Design of Small Dams the discharge coefficient can be determined. This is a matter of interpolation. To reduce the amount of concrete that is needed for the crest length of depth a ratio of .6 is used for the P/H value on the chart. As seen in Figure 27 the value of .6 indicates the discharge coefficient value to be 3.75-3.85 range.

Solving Equation 19 with Q=19,000 cfs, C = 3.8, and L =100 ft., the head pressure is then calculated to be 4.64 feet.

The Ogee Curve is determined using Equation 20.

\[
\frac{y}{H} = -K \star \left(\frac{x}{H}\right)^n
\]  

Equation 20
Solving this equation for x and y gives points along the ogee curve. These points then
determine points on the radius of the circles that make up the ogee curve. The radius is then
determined using Figure 28 from Design of Small Dams.

With these equations $R_1$ is determined to be 1.93 and $R_2$ is 4.36 feet. Due to the topography of
the dam location the spillway had to be arranged in a certain manner. Since the spillway
overflow is overtopping the dam some form of flow mitigation must be used. This is achieved
by having a chute for a spillway. This allows for concrete corrosion prevention as well as a
lower velocity entering the stilling basin.
The flow through the spillway is generated using the Mannings equation, Equation 21.

\[
Q = 1.49 \times \left( \frac{A R^{3/2}}{S^{1/2}} \right) \frac{s^1}{n} \quad \text{Equation 21}
\]

This is then used to solve for the normal flow depth which is 1.4 feet. The critical depth is also calculated for the maximum PMF using equation 21.

![Diagram showing flow depth and velocity](image)

\[
Dc = 3\sqrt{q^2/g}
\]

Equation 19

The critical depth for this spillway is found to be 10.98 feet. From this a wall height is determined to be a minimum of 11 feet. Being conservative of potential larger flows a side wall
height of 13 feet was chosen. This will be added to the overall height of the dam to ensure that any overflow is channeled into the spillway.

*Figure 93: Schematic of Spillway Channel*

**Stilling Basin**

There will be a stilling basin located at the end of the spillway next to the edge of Boysen Reservoir. This will also be designed for a 200-year flood. Using previously calculated numbers of the height of water above crest, $H$, and normal height of flow rate through the spillway the exit velocity can be determined. Using the following formula:

$$q = V_2 \cdot y_2$$

$V_2 =$ Velocity at end of spillway
\( y_2 = \text{Height of flow at end of spillway} \)

The velocity of the water leaving the spillway ends up at 131.543 feet per second.

\[
Fr = \frac{V_2}{\sqrt{g \cdot d_1}}
\]

\( Fr = \text{Froude Number} \)

\( d_1 = y_2 = \text{Normal flow height} \)

Using Equation 23 a Froude number of 19.826 is calculated.

Since the Froude Number is high a Type II stilling basing may be used.

*Figure 2410: Type II Stilling Basin Dimensions*
The depth of the water after the hydraulic jump is now calculated using the following equation:

\[ d_2 = d_1 \times \frac{1}{2} \left( \sqrt{1 + 8 \times Fr^2} - 1 \right) \]

where:
- \( d_2 \) = depth of water after jump
- \( d_1 \) = depth of water before jump
- \( Fr \) = Froude number

The depth after the hydraulic jump comes out to be 41 feet. Since the hydraulic jump has to dissipate before it reaches the end of the stilling basin, a length of 180 feet is used.

**Economic analysis**

Green energy does not come at the cheapest price. The initial cost of this project came out to be around $165 million. Table 3 summarizes the initial cost of the materials that are needed to construct the dam along with the annual estimated maintenance cost and the anticipated monthly electrical output generated. All of the initial costs include the labor that is needed for installation. Because the dam was designed as a concrete structure, the concrete is the largest expense. An economic analysis was performed in order to generate a benefit cost ratio. This consisted of taking all of the cost and benefits and converting them to a future worth at a life of 100 years using Equation 23. An interest rate of 1% was used for the calculation to
account for inflation. That value was then brought back to a present value using Equation 24
and compared to the initial cost of the project. This analysis generated a benefit cost ratio of
1.23. With a B/C over a value of 1.0 this project is justifiable and will generate a profit in the
end.

\[ FV = P \times \left( \frac{(1 + r)^n - 1}{r} \right) \]  

Equation 23

\[ PV = \frac{FV}{(1 + r)^n} \]  

Equation 24
This project is expected to span two years for start to end. The Gantt chart for the project can be seen in appendix A-A. The project design initiation started in February of 2016, where Team D designed and calculated the aspects of the project. This design time lasted through the month of April 2016. The project is scheduled to begin construction in June of 2016 and last approximately two years.

<table>
<thead>
<tr>
<th>Item</th>
<th>Cost</th>
</tr>
</thead>
<tbody>
<tr>
<td>9 Foot Francis Turbines * 2</td>
<td>$455,000/turbine*2=$910,000</td>
</tr>
<tr>
<td>720 Feet of 10 Diameter Steel Pipe</td>
<td>$2000*ft. pipe= $1.44 million</td>
</tr>
<tr>
<td>Pumps</td>
<td>$40,000*4=$160,000</td>
</tr>
<tr>
<td>Earthwork</td>
<td>$90/CY*24,000CY= $2.16 millions</td>
</tr>
<tr>
<td>Concrete (1,365,122 CY)</td>
<td>$80/CY=$109,209,760</td>
</tr>
<tr>
<td>Reinforcing Steel (7,000 CY)</td>
<td>$6,930/CY=$48,510,000</td>
</tr>
<tr>
<td>7 Howell Bunger Valves</td>
<td>$1.2 million</td>
</tr>
<tr>
<td>7 Butterfly Valves</td>
<td>$1.7 million</td>
</tr>
<tr>
<td><strong>Total</strong></td>
<td><strong>$165,289,760</strong></td>
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<tr>
<td>Annual Maintenance Cost</td>
<td>$15,000</td>
</tr>
<tr>
<td>Electric Output</td>
<td>$54,600/month</td>
</tr>
</tbody>
</table>
Summary

The purpose of this project was to design key components for a pumped storage facility. This facility consists of a dam, penstocks, forebay and afterbay, pumps, turbines, spillway, and stilling basin. During hours of low energy demand water will be pumped to refill our storage reservoir. When energy is in high demand, water will flow into our hydroelectric operation to produce energy to meet the demand. This system is extremely green and efficient.

Our project location is on the Northwest side of Boysen Reservoir in central Wyoming. We chose this location due to the high storage capacity of Boysen Reservoir and the fact that it will easily be able to refill our proposed storage reservoir. The topography of the land allowed us to design a reservoir that will hold 1975 acre-ft of water. Comparing this to Boysen’s 150,000 acre-ft of water, there should be no problem refilling the upper reservoir.

A concrete gravity dam was chosen due to the large span of the valley the dam will occupy. It will be 876 ft long and have a total height of 250 ft. A spillway that was designed for a 200-year flood will be constructed over the dam and flow into a stilling basing to allow the velocity of the water to slow before entering into Boysen Reservoir.

The penstocks were designed based on the flow rate and length. There will be two penstocks, one for the pumping operation and the other for the turbine operation. Both penstocks will have a diameter of 10 ft, and will be equipped with a surge tank and butterfly valves. The surge is a safety accessory on the penstock that absorbs sudden pressure rises due to valves closing too fast, or other pressures from the turbine. The penstocks will be buried underground going through the dam, and the power house will be located on our dam.
The design and construction of this green pumped-storage facility is not cheap. After an extensive economic analysis was done on this design, a benefit cost ratio of 1.23 was calculated. With this number we can conclude that this project is justifiable and will pay for itself in the end. Also, this project is very feasible and would be a good project in the real world.
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