QUIDI VIDI CONSULTING

Group 1 : Rattling Brook Hydroelectric Development Spillway Design Final Report

David Ball
Michael Cahill
Megan Kavanagh
Sara Vaughan

quidividiconsulting@gmail.com
(709) 699-5127

April 2010

Rattling Brook Hydroelectric Development Spillway Design Final Report

Prepared by:

Quidi Vidi Consulting
Memorial University of Newfoundland

Prepared for:

Gary Humby & Trina White
Newfoundland Power

April 5, 2010

Quidi Vidi Consulting
Memorial University of Newfoundland
P.O. Box 4200
St. John's, NL
A1C 5S7

April 5, 2010

G. Humby, P.Eng & T. White, P.Eng
Newfoundland Power
PO Box 8910
St. John's, NL
A1B 3P6

Subject: Rattling Brook Hydroelectric Development Final Report

Dear Mr. Humby & Ms. White,

Quidi Vidi Consulting would like to submit the attached final report for the engineering design of the new overflow spillway at the Rattling Brook Hydroelectric Development. The report is a requirement of ENGI 8700 and has been prepared to complete the requirements of the course.

Several concepts were reviewed to determine the optimal design to meet the requirements of the project. Each concept is discussed in the report, as well as estimated costs for each option. Based on cost benefit comparisons, the construction of a labyrinth spillway is the preferred option. The detailed design of the labyrinth spillway as well as the cost estimate for the construction and the demolition of the existing structure are included in the final report.

If there are any questions concerning this report, we would be pleased to discuss them with you.

Yours Sincerely,

Megan Kavanagh
Project Manager
Quidi Vidi Consulting

Attachment: Rattling Brook Hydroelectric Development Final Report

CC: Dr. S. Bruneau

Table of Contents

Table of Figures

1.0 Acknowledgements

QVC would like to acknowledge and extend gratitude to the following persons who have made the completion of this report possible:

- Ms. Trina White and Mr. Gary Humby for their support in providing information and guidance in all aspects of the project.
- Dr. Amgad Hussien for his assistance in designing the structural elements of the spillway.
- Dr. Steve Bruneau for his guidance in preparing the reports required for the completion of the course.
- Dr. Claude Daley for his assistance in determining ice loads for the spillway.
- Mr. Shah Alam and Ms. Vanessa Pynn for their assistance in designing the foundation for the spillway.
- Dr. Leonard Lye for his assistance in determining earthquake loads for the structure.

2.0 Summary

As a requirement for the completion of the civil engineering program at Memorial University, students are required to complete a design project assigned by a company in the engineering industry.

Quidi Vidi Consulting (QVC), comprised of students Megan Kavanagh, Michael Cahill, Sara Vaughan and David Ball was paired with Newfoundland Power for the design of a replacement spillway for the Rattling Brook Hydroelectric Development.

QVC began by evaluating various options for a spillway at the Rattling Brook location. The concepts reviewed include a rubber dam spillway, a stoplog spillway, a steel gated spillway, an ogee shaped concrete overflow spillway and a labyrinth spillway. From a cost benefit analysis of the options and from operational preferences of Newfoundland Power, it was determined that a labyrinth shaped spillway would be the preferred design for the location.

Loading cases were determined for hydrostatic, hydrodynamic, ice and earthquake loading. Ice loading was found to be the governing condition, with a pressure of 0.6 MPa over an assumed ice thickness of 0.6m. If this project is to go ahead, QVC recommends reviewing the ice and earthquake loading for the area more thoroughly to ensure the principles applied during this design stage are accurate.

Based on the ice and hydrostatic loading conditions, the spillway dimensions were determined. The spillway wall will have a thickness of 1m, height of 2m and a length of 102m in addition to end wall abutments.

The foundation for the spillway will be 8m wide and 102m long, with a thickness of 0.5m. It will be placed on a leveling slab and anchored into the bedrock to resist sliding and overturning of the structure. Rock anchor design calculations were based on the assumption that the bedrock has similar properties to granite. This assumption should be confirmed through geotechnical investigations before design is finalized.

A cost estimate was completed for the construction of the new spillway and the demolition of the existing spillway. The estimated cost for the replacement spillway is $2.6M.

QVC was able to complete the project by the prescribed course deadlines. Despite setbacks involving the design phase of the project, the final report and presentation were delivered on schedule.

3.0 Project Description

The Rattling Brook hydroelectric development began producing electricity in 1958. It is located in the community of Norris Arm South and is the largest hydroelectric plant currently operated by Newfoundland Power. The plant consists of two 7.5 MW generators fed by one steel penstock. Figure 1 shows a map of the hydroelectric development.

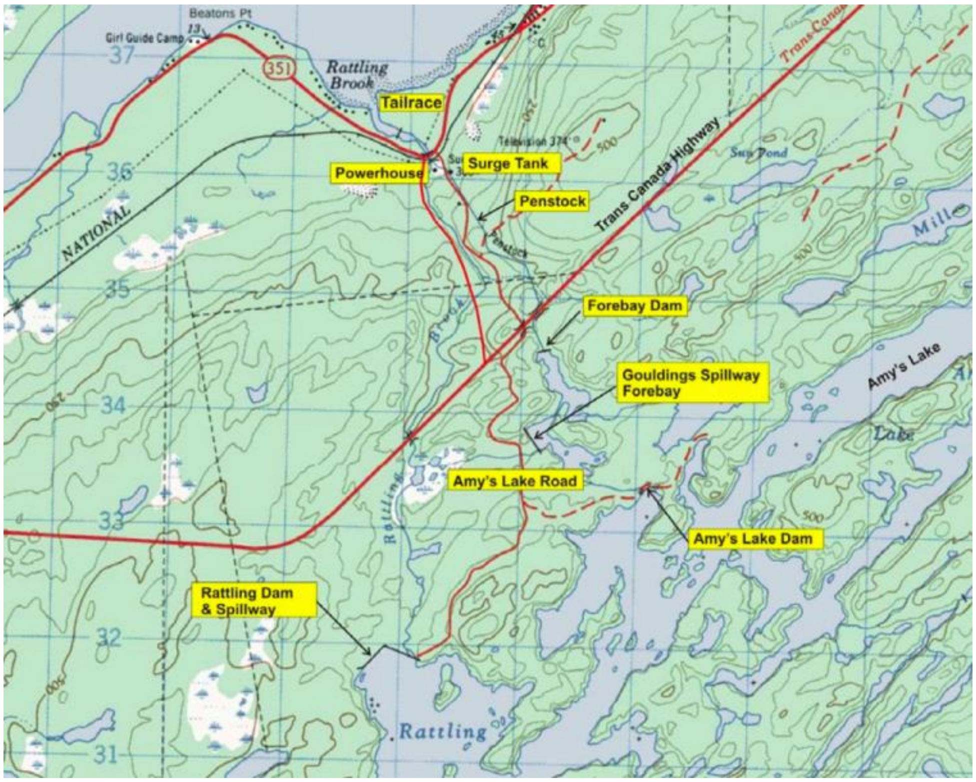


Figure 1 - Rattling Brook Hydroelectric Development

The primary storage reservoir is created by the combination of the former Rattling and Amy's Lakes. The primary overflow spillway is located adjacent to the Rattling Lake Dam and consists of a concrete base with 42 stoplog bays varying in size. Figure 2 shows the current stoplog spillway arrangement.

Figure 2 - Rattling Lake Overflow Spillway

The operation of the spillway in its present configuration leads to reduced hydro plant output as a result of inefficient operation. Because of the freeboard requirements of other dams in the system, the removal of stoplogs is required to safely pass flood flows. Removing stoplogs is very labour intensive and potentially hazardous to employees as manual lift hooks are used to raise excess stoplogs to a platform over the spillway. Flow predictions are required prior to a flood event to ensure a sufficient number of stoplogs are removed to accommodate the additional flood flow. Possible dam safety issues are created if an insufficient number of logs are removed. Further dam safety issues result if it becomes impossible to remove lower stoplogs under spill conditions. Water wastage can occur if too many logs are removed and replacing stoplogs under spill conditions is difficult, further increasing the amount of unnecessary spill.

During winter months, the reservoir elevation is lowered to prevent ice loads on the deteriorated structure. By limiting storage capacity of the reservoir to an elevation below full supply level (FSL), some inflow may not be captured or the water may not be used in the most efficient way.

Structural deterioration has been noted in the concrete base as well as the support struts. Due to the operational challenges and structural deficiencies of the structure, Newfoundland Power has decided to replace the overflow spillway. It is presently budgeted for the 2011 construction season. Figure 3 shows the existing layout of the site.

Figure 3 - Existing Site Layout

4.0 Project Requirements

Quidi Vidi Consulting (QVC) is responsible for the detailed design of a replacement overflow spillway for the Rattling Brook Hydroelectric Development.

4.1 Project Deliverables

Project deliverables include detailed design calculations, a cost estimate for construction of the new spillway and demolition of the existing spillway and detailed drawings.

4.2 Review of Spillway Options

A report was prepared for Newfoundland Power by Hatch in 2007 and options presented in that report were reviewed to ensure the optimal design was selected for construction. QVC has reviewed many of these options and a detailed discussion of each is presented in Section 5.0. The labyrinth spillway was the option chosen for detailed design.

4.3 Flood Passing Requirement

From previous studies conducted by Acres International Limited in 1999, the Inflow Design Flood (IDF) was determined as the event with the 1/10,000 Annual Exceedance Probability (AEP). This corresponds to a design flood with a peak flow of 416 m^3/s. These values were confirmed by Hatch in 2007 and this study incorporated up-to-date information from hydrometric stations in the area. The IDF of 416 m^3/s was used in the detailed design of the spillway.

4.4 Construction Requirements

QVC is not required to evaluate the optimal construction season or a schedule for construction. These items will be determined by Newfoundland Power after the design has been approved.

5.0 Spillway Option Review

5.1 Concrete Overflow

A concrete overflow was one concept considered for the Rattling Lake Spillway. The ogee (S-shaped) crest was selected because of its high discharge efficiency due to its nappe-shaped profile. This overflow would pass the design flood with no requirements for personnel to be on site and requires no power source for operation. This option was not selected primarily on the basis of cost compared with similar options like the labyrinth.

The concept design for this type was completed using the methods outlined in Chapter 17 of the *Hydraulic Design Handbook*. The parameters required for the discharge equation, including the discharge coefficient was determined from the curves presented in the text. From the discharge equation it was determined that a head of 1.75m is required to pass the design flood of 416 m^3/s (Mays, 1999).

Using the equations and curves in the text, the crest profile was plotted to in order to estimate the construction quantities. A nominal bedrock elevation of 111.86 m was chosen to be the base for the complete width of spillway with some additional concrete allotted for the varying elevations and abutments. Figure 4 is a plot of the concept crest profile.

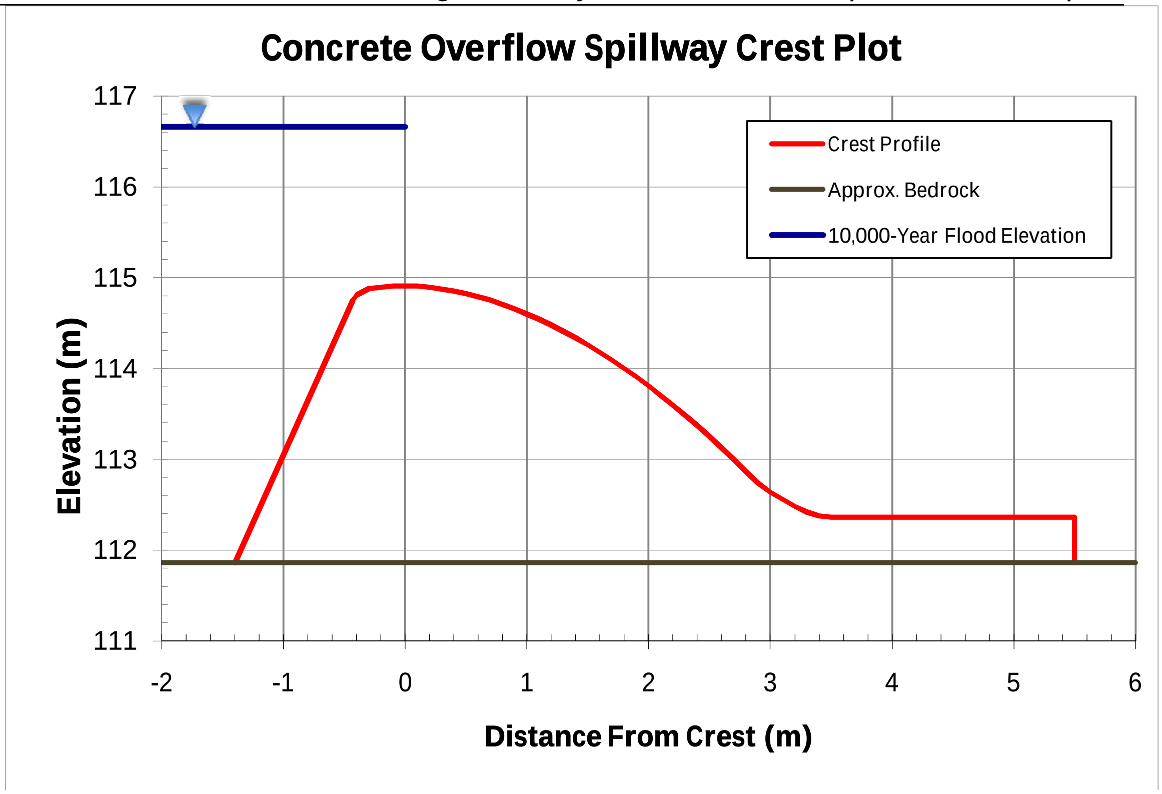


Figure 4 –Concrete Overflow Spillway Crest Plot

The flow characteristics of the weir, in particular the design head, had very large implications on the cost of this concept. A design head of approximately 1.75 m was required to pass the 10,000 year flood. Combining this design head with the allowance for wave overtopping of 1.6m gives a required dam height during normal operation conditions of 116.51m, and during 10,000 year flood of 118.26m.

To achieve this minimum elevation, the current dam crest elevation of 116.13m would have to be increased to avoid overtopping. Raising the dam crest 1.75m gives a dam crest elevation of 117.88m. This will be sufficient elevation to provide protection during normal operating and spill conditions. An additional 0.5m high riprap wall on the leading edge will provide a maximum height of 118.38m, ensuring protection from waves during extreme flood conditions.

An increase in dam crest elevation of 1.75m will require the dam footprint to be increased. For the purpose of quantity estimation a slope of 1.75:1 was chosen as the new downstream slope. This is a compromise between the original upper slope of 1.5:1 and the lower slope of 2:1. A typical section has been prepared showing the existing dam as well as the required improvements. See Figure 5 for a close up section of the improvements to the dam, and Appendix B for a section of the entire dam including additional material required to increase the footprint.

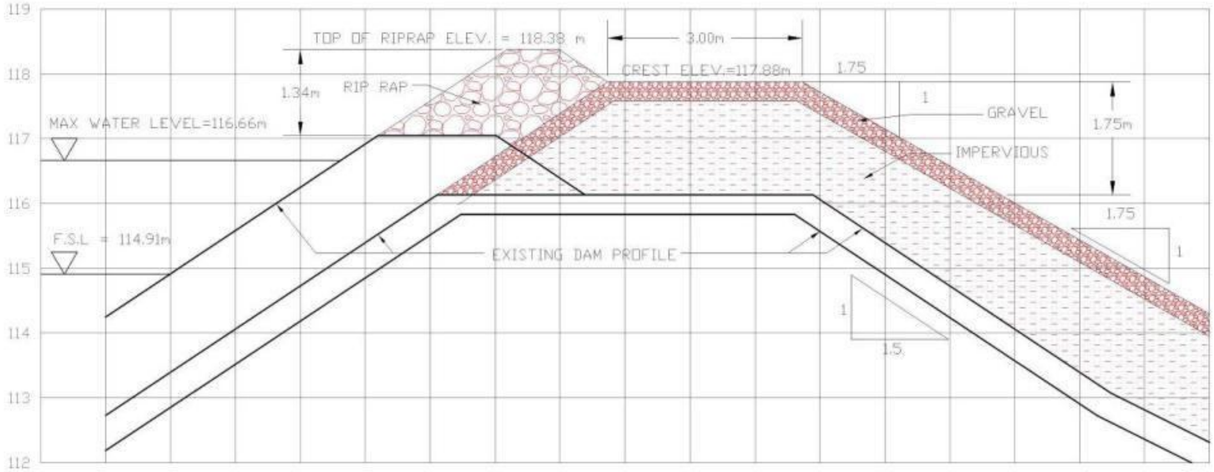


Figure 5 - Typical Dam Profile for 1.75m Increase

5.2 Labyrinth

The labyrinth spillway is used to increase the spillway crest length without increasing the width of the channel. This is achieved through thin walls that typically appear in plan as triangles or trapezoids. The increased spillway crest length allows more water to pass at a specific head then a straight spillway (Falvey, 2003). Figure 6 is a typical labyrinth spillway similar to the concept for Rattling Lake.

Figure 6 - Typical Labyrinth Spillway (Lake Salinda Dam Improvments, n.d.)

For Rattling Lake, the labyrinth spillway is the recommended option. It is the most cost effective alternative and requires no personnel or mechanical equipment for operation. It can pass the required flood flow with a much smaller head then a conventional spillway. Although forming will be more difficult than a conventional spillway, this cost is offset by the reduction in head required as well as reduced quantities of concrete.

A preliminary design was completed using the methods outlined in the *Hydraulic Design of Labyrinth Weirs* by Henry T Falvey (2003). Concrete quantities for this concept were estimated using a nominal bedrock elevation of 111.86 m, and assuming wall thickness of 0.5m and base dimensions of 8m wide by 0.6m deep with an allowance for varying bedrock elevations and abutments. The dimensions, including doubling of the thickness required, have since been updated to reflect the actual design details.

A dam crest elevation increase of 1m is required to ensure no overtopping during a 10,000 year flood event. From the original drawings it appears that this increase can be accommodated without increasing the footprint of the dam. A typical dam profile for the 1m increase can be seen in Figure 7.

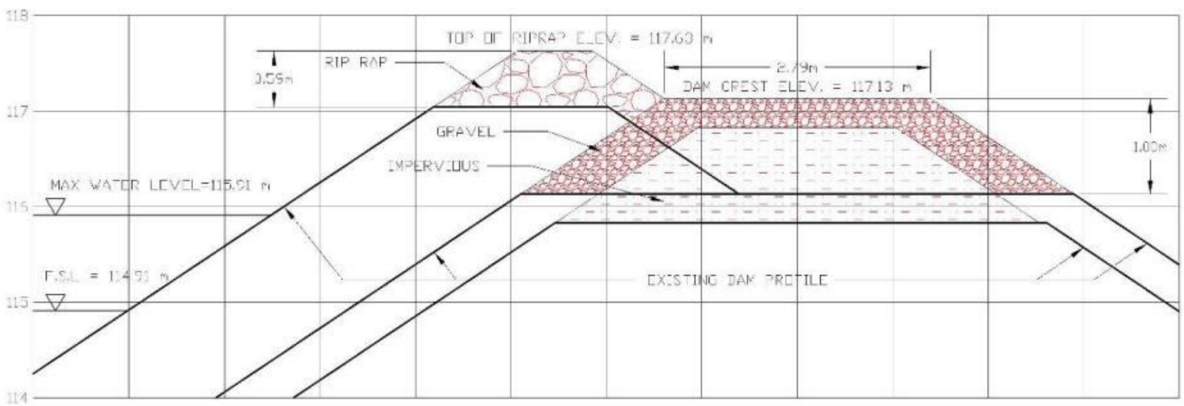


Figure 7 – Typical Dam Profile for 1m Increase

5.3 Rubber Dam

Rubber dams are used throughout the world as an alternative to concrete or steel gated spillways. Dams are installed onto concrete weirs, and when inflated with air or water, are used to prevent water flow, as shown in Figure 8. When deflated flood waters and sediment can pass safely through the channel (*Pioneering Rubber Gate Technology,* n.d.).

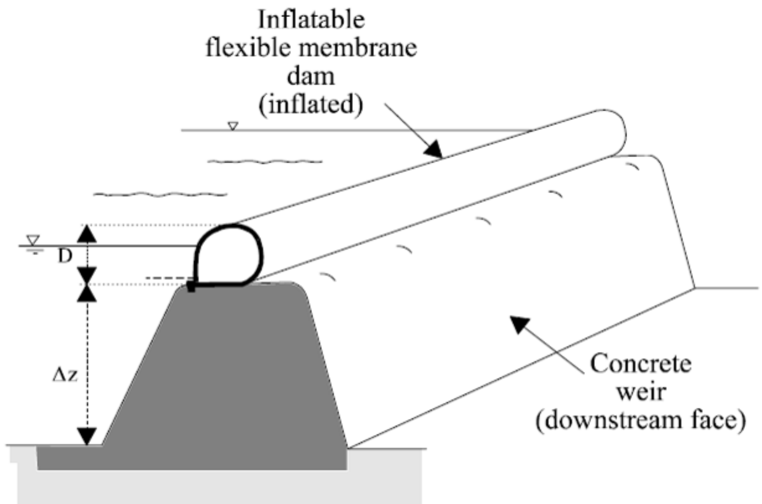


Figure 8 - Rubber Dam Layout (Chanson, 1998)

Rubber dams are typically manufactured from multiple layers of rubber and nylon with wire mesh, ceramic chips or other materials incorporated to prevent damage from vandalism (*Pioneering Rubber Gate Technology,* n.d.). In a climate such as Newfoundland`s, air would be the preferred inflation fluid, especially during winter months, to prevent damage due to freezing temperatures.

There are many advantages to using a rubber dam spillway. The material is easily and quickly installed after the concrete base is in place. Sand and silt can be flushed through the spillway after the rubber dam has been deflated, and cushioning is available in the dam to prevent damage from moving rocks when this occurs. The inflation and deflation of the dam can be handled manually, automatically or remotely, removing the requirement for personnel onsite. The rubber dam is earthquake resistant and can be installed on soft or sensitive grounds because of its light weight. The rubber dam is also corrosion resistant and can be repaired quickly (*Pioneering Rubber Gate Technology,* n.d.).

Disadvantages of choosing a rubber dam must also be considered. Rubber dams have a significantly shorter lifespan than other options. Overflows of greater than 20% can cause damage to the spillway, and must be mitigated by additional design work (Chanson, 1998). The possibility of vandalism must also be considered when installing a rubber dam in a remote location.

Using a dam height of 2m, with a concrete base of 1m (based on existing bedrock and sill elevations), the length of rubber dam required to pass the 10,000 year flood with no increase in dam height was determined to be 50m. This would allow for 57m of concrete spillway to be used as a less expensive alternative to choosing a dam entirely from a rubber spillway. Calculations for the width of rubber dam can be seen in Appendix B.

Rubber dams are not recommended for the Rattling Brook Spillway because of the increased cost associated with installing a rubber dam as opposed to a concrete structure. The rubber material is significantly more expensive than concrete, and would require replacement more often than a concrete structure. A rubber dam would also require power to be brought in to the dam site to operate compressors and control equipment, which further increases the costs.

5.4 Gates

Gates are used in spillways to regulate the rate of flow through the structure. This design allows water storage year round and flood water can be released quickly by opening the gates. This method does not require personnel to be located on site, but constant power is necessary to operate the site remotely.

The gates are required to be dimensionally large in order to pass debris during extreme floods. The gates at Rattling Lake will also have to resist ice loading and vibrations due to partial opening, resulting in an increased weight of steel. The cost of the spillway increases significantly as the size and weight of the gate increases. The instrumentation and mechanical devices required for the gate have to be protected against environmental impacts, such as corrosion, which also has an impact on the cost (Cassidy, n.d.). We have determined that gates are not a viable option for the Rattling Brook Spillway on the basis of the high costs of supplying and installing the gates and associated equipment.

5.5 Stoplogs

A stoplog dam consists of one or more slots where timber logs can be inserted or removed to change the elevation of the reservoir or increase flow to pass a large flood through the abutments (*Lake Outlet Dams*, 1999). A typical stoplog spillway is shown in Figure 9. During flood conditions, the logs are removed manually using lifting hooks and pins in each log. Lifting is done from a walkway above the spillway and logs are stored on a platform attached to the walkway. Once the flood has passed, the stoplogs are replaced individually in the same manner. The existing spillway is of manual stop log construction. As much as 50 continuous hours of manual labour may be required to pull all of the logs from the current Rattling Spillway (Smith, 2007).

A modified version of the manual stoplog spillway is the quick release stoplog spillway. Timber stoplogs supported by abutments or fixed supports with a central vertical column that can be released by removing a pin or another fixed mechanism at the top. Once the pin is pulled, the stoplogs are released downstream. The logs and vertical column then have to be collected after the flood and replaced individually (Smith, 2007).

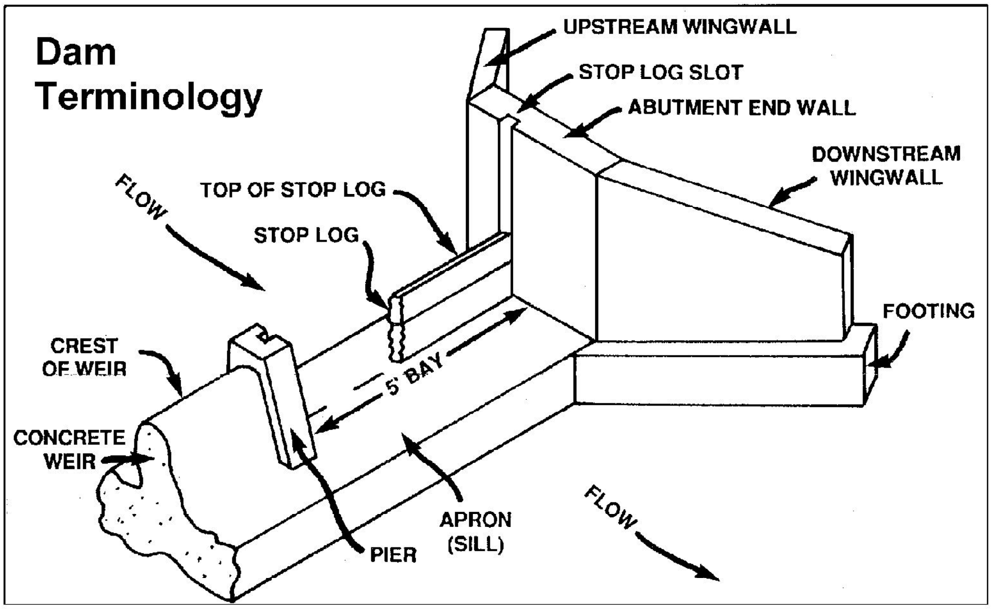


Figure 9 - Stoplog Arrangement (*Lake Outlet Dams,* 1999).

5.5.1 Operational Difficulties - Manual Release Stoplogs

There are a number of operational difficulties associated with this type of spillway. Removing stoplogs during flood conditions poses a hazard to Newfoundland Power employees, as they have to work over the flood water for extended period of time. This is compounded by the increased weight of stop log anticipated as a result of design for ice loading.

The stoplogs may also become jammed in their slots and may not move during a flood, reducing the flow over the spillway (Cassidy, n.d.) and increasing repair costs post flood.

This type of spillway requires more flood monitoring and watershed management then a static spillway. Predictions are required to coordinate the response of opening and closing the stoplog spillway as it is very slow process.

For these operational reasons as well as the increased cost over other alternatives, the manual release spillway is not an acceptable alternative. A cost estimate for this option is included in Appendix B.

5.5.2 Operational Difficulties - Quick Release Stoplogs

A quick release stoplog spillway requires personnel to be on site to remove the pin at the time of flooding or equipment that can control it remotely, taking significantly less time to implement a flood response then the manual stoplog spillway.

Retrieving the stoplogs once they have traveled downstream is very labour intensive. If stoplogs are not located after the flood, they must be replaced, increasing the operating costs of the spillway. All water from the flood will be wasted as opposed to being stored behind the dam because the central column and timber logs cannot be released until the reservoir elevation is below the sill elevation (Smith, 2007). This unnecessary spill will have an impact on power generation.

This is not an acceptable option based on the increased construction costs, operational issues and generation losses that are not present in the preferred alternative. A cost estimate of this option is also presented in Appendix B.

5.6 Comparison of Costs and Benefits

QVC carried out a comparison of costs to determine the preferred spillway design. Costs were determined for a Rubber Dam/Concrete Overflow Combination, Concrete Overflow and Labyrinth Overflow spillways. The lump sum and unit price costs originated from tender prices received from Newfoundland Power. Material quantities were estimated using quantity takeoffs from preliminary designs and used to determine the total cost of each option. Appendix B presents the costs per meter length and fixed costs for each type of spillway. Total costs were determined from variable, fixed and indirect costs as outlined in Appendix B. Indirect costs were estimated as 30% of the construction cost and include items such as engineering and detailed design. Costs contained in the Hatch report (2007) which were used to evaluate other options that are presented in Appendix B.

The labyrinth overflow is the recommended replacement spillway for construction. It meets all requirements of the project and has the lowest construction cost.

6.0 Detailed Design

6.1 Freeboard Calculations

Wave runup is the vertical distance waves will advance up the dam as they approach the structure, as illustrated in Figure 10 by the symbol R. Runup must be considered in the design process, as overtopping of the dam can cause significant damage.

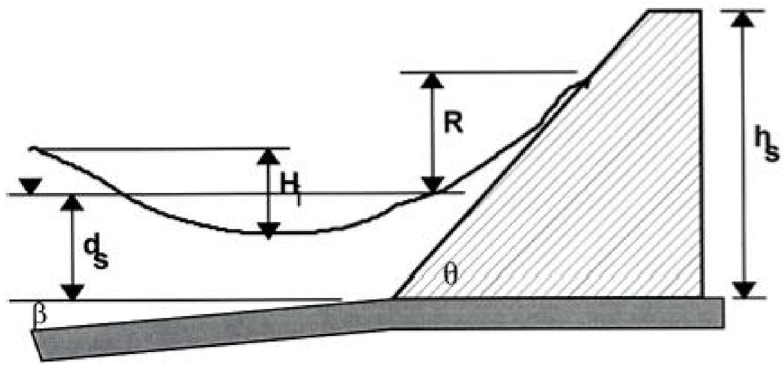


Figure 10 - Schematic of Wave Runup (Bruneau, 2009)

According to the Canadian Dam Association (2007) sufficient dam freeboard (the distance between the FSL and dam crest) should be in place to prevent wave overtopping for 95% of waves formed for each wind case studied. Two cases should be applied during the design, the first being the maximum wind conditions when the reservoir is at FSL. The second case occurs when flood conditions and normal wind forces occur simultaneously. The basis for these cases takes into account the rarity of an area receiving maximum wind and flood conditions at once (p 64-65).

The wave height was generated using a nomogram from the USCAE Shore Protection Manual (1984). The nomogram takes into account the wind-stress force and the fetch length over which the wave can develop. Wave runup was determined to be 1.6m for the site. Calculations for the runup can be seen in Appendix A.

Wind pressure data from the Gander International Airport was used along with the National Building Code of Canada to determine the 1/1000 year one hour wind pressure, speed and force. The airport is located approximately 50km from Norris Arm, and both sites are at a similar elevation. Error could be introduced depending on the exposure conditions present at each site.

6.2 Hydraulic Design

The hydraulic design for the weir was completed in accordance with the analysis outlined in the text *Hydraulic Design of Labyrinth Weirs* produced by the ASCE. Flow over the labyrinth is primarily a function of the head, weir height, width of weir, developed labyrinth width and shape. We selected a weir that is triangular in plan with a quarter round crest profile on the upstream side that will improve discharge efficiency. The method by Tullis et al. (1995) was selected as our primary analysis method as it was the most recent and was applicable to our design case.

Because flow is a function of so many variables, the designer has significant flexibility when choosing dimensions. QVC tried to minimize the footprint of the spillway while also minimizing the head requirements. After analyzing several configurations, it was determined that a head of 1m produced reasonable spillway dimensions. Decreasing the head below 1m will require significant increases in developed labyrinth length as well as wall strength, increasing the concrete required.

Combining the design head of 1m with the allowance for wave overtopping of 1.6m results in a required dam height at FSL of 116.51m. During a 10,000 year flood, to achieve this minimum elevation, the current dam crest elevation of 116.13m would have to be increased by 1m to avoid overtopping. A crest elevation of 117.13m will be sufficient to provide protection during normal operating and spill conditions. An additional 0.5m high riprap wall on the leading edge will provide a maximum height of 117.63m, ensuring protection from waves during extreme flood conditions. See Appendix D for more information on the hydraulic design.

6.3 Loading

Detailed information on loading can be found in Appendix C.

6.3.1 Hydrostatic Loading

Hydrostatic loading was determined considering the reservoir at FSL. For this case, hydrostatic forces are the only forces present on the spillway. Hydrostatic pressure is determined through the following expression:

$$P = \rho g h$$

Where: ρ = density of water
g = acceleration due to gravity
h = height of water

Applying this expression at various points along the depth of the structure yields a triangular load distribution.

Hydrostatic forces are applied perpendicular to the structure. Total hydrostatic forces were determined by first calculating a force per meter length of the structure and then applying it over the effective length of the dam. These calculations yielded a maximum hydrostatic force of 19.62 kN/m. Moments at the base of the labyrinth were determined in the same manner, with a lever arm of 0.667m, which is one third of the total labyrinth wall height. The maximum per meter moment was found to be 13.08 kNm.

6.3.2 Dynamic Loading

Hydrodynamic loading was determined for the spill condition during a 1/10,000 year flood event. This case includes static forces along the entire length of the spillway and dynamic forces from flood water spilling over the structure. Static forces are determined in the same manner outlined in Section 6.3.1, while dynamic forces are calculated though the drag equation:

$$F_d = 1/2\rho V^2 C_d$$

Where: ρ=density of water
V=velocity of water over spillway
C_d = drag coefficient

Conservatively, the drag area was taken to be the side surface area of the entire dam. The drag force is extremely susceptible to the velocity of the flow, and so the flow speed was taken at 3 m/s, which was the maximum velocity found. The drag force at the top of the spillway was determined to 27 kN/m. The total static load along the spillway ranged from 4.91 kN/m at the top to 23.35 kN/m at the bottom. The moment for this case was found to be 44.04 kNm per meter spillway length.

6.3.3 Ice Loading

Ice pressures can produce a significant load against the face of a dam. These pressures are caused by the thermal expansion of the ice, which depends on the temperature, thickness, coefficient of thermal expansion, the elastic modulus and the strength of the ice. In addition, wind drag on the ice causes significant pressure to be exerted on the dam. The wind drag depends on the size and shape of the exposed area, the roughness of the ice surface and the direction of the velocity of the wind (Design Criteria for Concrete Arch and Gravity Dams, 1977). In Newfoundland this is generally the governing factor of any loading on a dam or spillway and therefore must be designed to resist this pressure. Based on information in *Hydraulic Design of Labyrinth Weirs* (2003) and a conversation with Dr. Claude Daley, it was suggested to use a range of ice loads of 0.5 MPa to 2 MPa with an ice thickness of 609.6 mm (or 2 ft). QVC chose an ice loading of 0.6 MPa, giving a total distributed ice pressure of 153 kPa for the entire structure. Ice loading was the governing load condition and these loads were used to determine the design of the joint connection between the dam and foundation as discussed in Section 6.4.2.4. This loading was also used to determine the applied sliding and overturning forces on the concrete foundation, as discussed in Sections 6.4.2.2 and 6.4.2.3.

6.3.4 Earthquake Loading

According to the Dam Safety Guidelines (2007) seismic loadings on a dam structure must take into account the local and regional geotechnical and tectonic information and a statistical analysis of historical earthquakes in the region. The National Building Code cannot be used to determine the seismic loads on a dam structure (Dam Safety Guidelines, 2007, p 66-67).

Loading due to earthquakes must be considered to occur at the same time as ice loading, due to the probability of both events happening at the same time.

Using a simplified approach outlined in Hydraulic Structures (Smith, 1995), the earthquake load can be calculated by the following equation:

$$P_e = Ma$$

Where: M= mass of the structure
a= earthquake acceleration

Earthquake intensity is defined in terms of a factor α, which is the ratio of earthquake acceleration (based on seismic area) to acceleration due to gravity. The above equation can then be simplified to:

$$P_e = \alpha W$$

Where: α= ratio of earthquake acceleration to acceleration due to gravity
W= weight of the structure

For Atlantic Canada, α=0.05, giving an earthquake loading of 10.5 kN/m. On the critical section of the spillway, the labyrinth wall, the earthquake loading is 5.9 kN/m. These loads are not significant compared to the ice loading determined, and will be accounted for in the safety factors used in the ice loading calculations.

6.4 Structural Design

6.4.1 Labyrinth Wall

The labyrinth walls were designed as plates of a concrete tank. The methods outlined in the text *Rectangular Concrete Tanks,* produced by the Portland Cement Association was used in this design. Moment and shear coefficients for various plate configurations are presented in tabular format in Appendix E.

The maximum per meter shear loading on the wall was calculated to be 401.5kN. In order to resist this applied force with no reinforcement, the labyrinth wall must be 1000mm thick.

To resist the moments applied to the wall, only minimum horizontal and vertical reinforcement is required. Using 20M bars, the spacing necessary for horizontal bars is 140mm and for vertical bars is 200mm. See Drawing 2010_007 in Appendix G for details of rebar spacing.

Detailed calculations for the labyrinth wall structural design can be found in Appendix E.

6.4.2 Foundation

Foundation design was completed using methods outlined in *Foundations on Rock* by D.C. Wyllie.

6.4.2.1 Dimensions

The foundation height will be 0.5m, with a length of 102m and a depth of 8m. The orientation of the labyrinth is such that the centerlines of the walls coincide with the centerline of the foundation. The compressive strength of concrete for foundation was selected to be 25 MPa. A free body diagram of a typical section is illustrated in the Figure 11.

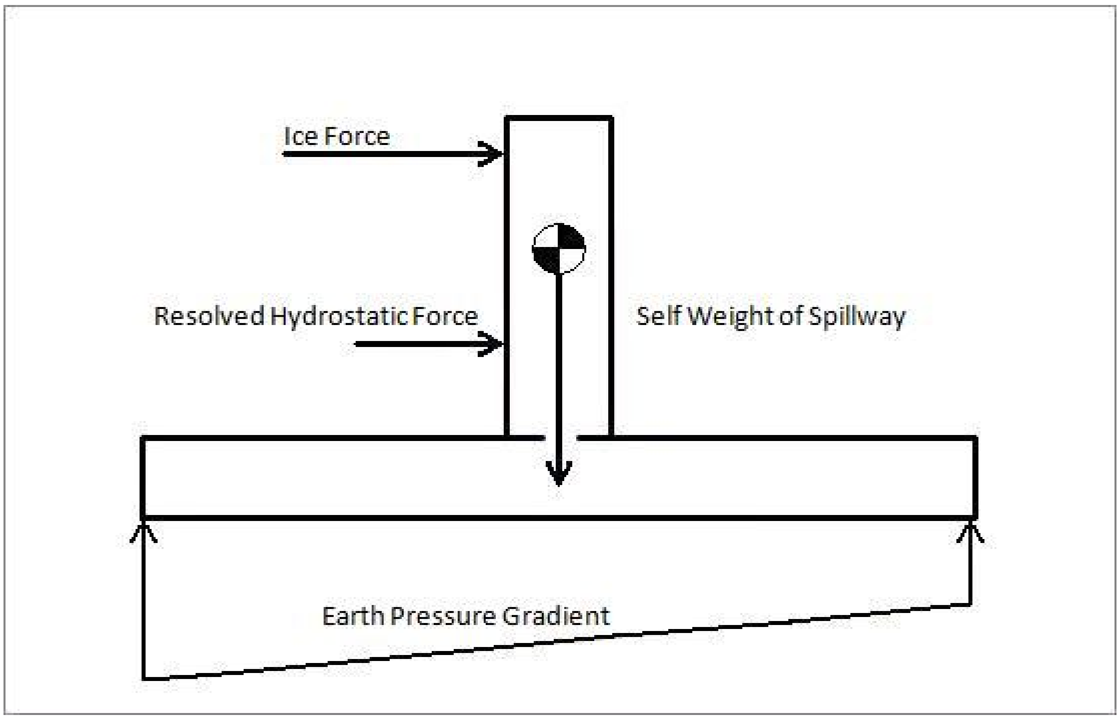


Figure 11 - Free Body Diagram of Spillway

6.4.2.2 Sliding

In order to design the foundation for sliding considerations, the total lateral force on the spillway had to be determined. The governing load case is when the dam is subject to full ice, earthquake and the associated hydrostatic loading with the reservoir at FSL. As discussed in Section 6.3.4, earthquake loading will be considered as part of the ice loading and associated safety factors. Using the per meter ice and water forces, the total force on each panel was determined to be approximately 200kN. This force was resolved into its component forces onto the labyrinth and then applied to each bay in the spillway. As seen in Figure 12, forces parallel to the foundation length will cancel for each bay while the forces perpendicular to the foundation length are additive.

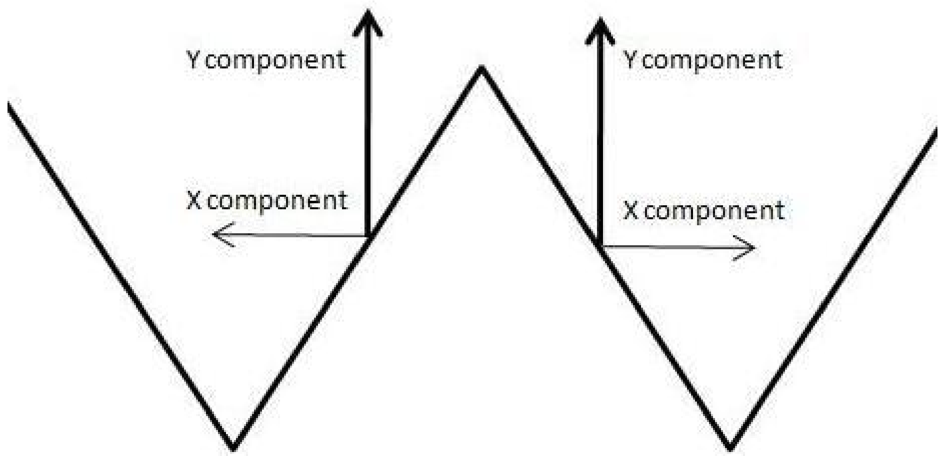


Figure 12 - Ice and Hydrostatic Loading Free Body Diagram

The resolved bay forces are then summed along the length of the spillway. The total factored component force applied on the spillway was determined to be approximately 66,200 kN, including a factor of safety of 1.5.

The sliding resistance of the spillway is a function of the self weight of the entire structure and the frictional interaction between the foundation and the subsurface materials. The coefficient of static friction, μ_s, for concrete-bedrock interaction was taken as 0.3. Assuming normal density concrete, the weight of the structure was determined to be 19,212 kN. From these values, the total spillway resistance was determined to be 5,764kN. Additional lateral support will be provided by anchoring the structure into the bedrock. Using CSA S16-01 Cl 13.12.1.1 (Design of Steel Structures), it was determined that 300 No 35M bars are required to resist the excess lateral force. Calculations are further outlined in Appendix E.

6.4.2.3 Overturning

Applied overturning moments are calculated using the lateral forces applied to the structure and their associated lever arms. A factor of safety of two was used for overturning. The total applied factored moment was determined to be approximately 181,500 kNm.

The overturning resistance of the spillway is a function of the spillway weight and the lever arms associated with the wall and foundation. The total resisting moment was determined to be 86,454 kNm. Using 300 rock anchors, as required for sliding, the length of rock anchors required is determined in Section 6.4.2.6.

6.4.2.4 Joint

The total factored applied shear force was determined to be 611 kN per meter of spillway length, which includes a factor of safety of 1.5. The shear resistance of the concrete at the labyrinth-foundation join was determined to be 342 kN/m, indicating that shear reinforcement is required. To resist the excess shear force, 3 No 25M bars are required per meter of spillway length (Concrete Design Handbook, Cl 11.3.4). The development length required to resist the force was found to be 1200 based Clause 12.2.3 of the Concrete Design Handbook. Due to the relatively shallow foundation insufficient development length is available and hooked bars are required (l_d=426mm). Calculations are further outlined in Appendix E.

6.4.2.5 Toe and Heel

The toe (downstream side) and heel (upstream side) of the foundation are exposed to primarily vertical forces from the bedrock and water pressure. Although the horizontal forces applied to the structure do not have a major impact on the design, they do have an effect on the location of the center of force and the reaction forces from the bedrock under the foundation.

The heel was assumed to behave as a cantilever beam with the water forces acting downwards. Conservatively, bedrock reactions are ignored in the calculations. Using Clause 11 of the Concrete Design Handbook, the factored shear force was determined to be 184 kN/m. The concrete shear resistance for the same section was determined to be 202 kN/m, therefore shear reinforcement is not required in the foundation. Assuming a cantilever support at the heel, the factored moment was determined to be 459 kNm per meter length. Using the design aid Table 2.1 in the Concrete Design Handbook it was determined that 730 No 25M bars spaced at 140mm in the top of the structure are required to resist the applied moment. It was determined that the bars had sufficient length over which to develop, allowing straight bars to be used. No 15M bars spaced at 500mm are required for temperature and shrinkage reinforcement. See Drawing 2010_007 in Appendix G for details.

Calculations undertaken for the toe of the spillway show that the steel characteristics specified for the heel are valid for the toe, but the steel is required in the bottom of the section as opposed to the top, as the bedrock forces are not neglected in this case. See Drawing 2010_007 in Appendix G for details.

For ease of construction, the heel and toe will have the same reinforcement quantities.

6.4.2.6 Rock Anchor Detail

Grouted rock anchors will be used to supply additional resistance to sliding and overturning, as outlined in Sections 6.4.2.2 and 6.4.2.3. Sliding was found to be the governing case for rock anchor requirements, with 300 rock anchors necessary.

The excess moment to be resisted by the anchors was determined to be 94,994 kNm. The rock anchors are laid out in three lines of 100 anchors along the length of the structure, at 2, 4 and 6 meters from the overturning point at the toe of the foundation. It was assumed that each line of anchors would resist one third of the excess moment (31,664 kNm) which results in rock anchor pullout forces ranging from 158 kN (closest to the pivot point) to 52 kN (furthest away from the pivot point). The bedrock was assumed to be weak granite, giving conservative estimates of the rock-grout development length, the rock compressive strength, density and apex angle (Wyllie, 1992). The lengths of the rock anchors were determined to be 4m, 3.11m, and 2.6m, from closest to the pivot point to the furthest away, respectively. See Drawing 2010_005 in Appendix G for further details.

7.0 Cost Estimation

Most costs associated with the construction and materials for this project were determined using recent tender prices obtained by Newfoundland Power. The main components of the cost estimate included: mobilization and demobilization of equipment and personnel, site preparation and excavation, and supply and placement of concrete and dam material. Information on the material and installation costs for the rock anchor placement were determined from RS Means (2008) and were then escalated by 10% to account for inflation. The cost for all concrete works associated with the project was set at $\$1100/m^3$ due to the remoteness of the area and the complicated formwork. 30% was added to the estimate to account for indirect costs such as engineering, site supervision, travel expenses and other related expenses. The final cost for the project is estimated at $2.6 million.

This cost is approximately $0.6M higher than the original cost benefit estimate but is still lower than the other reviewed options. This increase is a result of updated unit costs, increased concrete quantities and the requirement for rock anchors. Detailed design of other options will likely result in increased costs in a similar manner, due to increased costs and the requirement for rock anchors.

8.0 Schedule

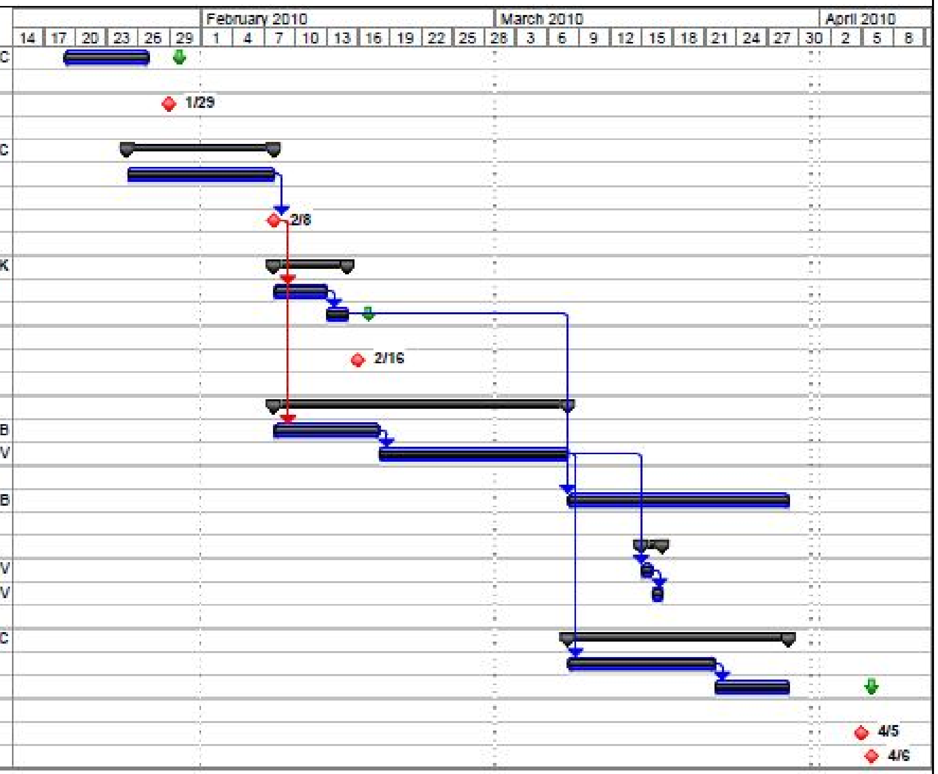
During the course of the project, minor changes were required to the schedule set out in the workplan. The structural design took longer than anticipated, due to difficulties in determining ice loadings. This caused drafting efforts to be delayed by approximately three weeks. It was possible to begin the final report and presentation earlier than anticipated, incorporating information into them as it became available. The previously anticipated time required to carry out the cost estimates were overestimated, allowing more time to complete other tasks. All these items allowed the project to be finished within the deadlines set by QVC, as well as the course deadlines. Figure 13 shows the completed project schedule.

QVC

Rattling Brook Spillway Replacement
Project Schedule
Revision 3

ENGI 8700
Group 1

ID		Task Name	Duration	Start	Finish	Predecessors	Person Responsible
1		Workplan Development	8 days	Tue 1/19/10	Wed 1/27/10		SV, MK, DB, MC
2							
3		Workplan Due	0 days	Fri 1/29/10	Fri 1/29/10		
4							
5		**Re-evaluate Feasibility Study**	**14 days**	**Mon 1/25/10**	**Mon 2/8/10**		**SV, MK, DB, MC**
6		Re-evaluate Feasibility Study	14 days	Mon 1/25/10	Mon 2/8/10		
7							
8		Final Concept Set	0 days	Mon 2/8/10	Mon 2/8/10	6	
9							
10		**Preparation Mid-Term Report**	**7 days**	**Mon 2/8/10**	**Mon 2/15/10**		**SV, MK**
11		Report	5 days	Mon 2/8/10	Sat 2/13/10	8	
12		Slideshow	2 days	Sat 2/13/10	Mon 2/15/10	11	
13							
14		Midterm Progress Presentations	0 days	Tue 2/16/10	Tue 2/16/10		
15							
16		**Detailed Design**	**28 days**	**Mon 2/8/10**	**Mon 3/8/10**		
17		Hydraulic Analysis	10 days	Mon 2/8/10	Thu 2/18/10	8	MC, DB
18		Structural Analysis	18 days	Thu 2/18/10	Mon 3/8/10	17	MC, MK, SV
19							
20		Drafting	21 days	Mon 3/8/10	Mon 3/29/10	12	DB
21							
22		**Cost Estimate**	**1.5 days**	**Mon 3/15/10**	**Tue 3/16/10**		
23		Demolition	0.5 days	Mon 3/15/10	Mon 3/15/10	18	SV
24		Construction	1 day	Mon 3/15/10	Tue 3/16/10	23	SV
25							
26		**Prepare Final Report**	**21 days**	**Mon 3/8/10**	**Mon 3/29/10**		**MK, MC**
27		Report	14 days	Mon 3/8/10	Mon 3/22/10	18	
28		Slideshow	7 days	Mon 3/22/10	Mon 3/29/10	27	
29							
30		Final Report Due	0 days	Mon 4/5/10	Mon 4/5/10		
31		Final Presentation	0 days	Tue 4/6/10	Tue 4/6/10		



Project: Rattling Brook Spillway Replacement - Rev 3
Date: Wed 3/31/10

Task | Progress | Milestone | Summary | Deadline

Page 1

Figure 13 - Project Schedule

9.0 References

Bruneau, S. *Coastal Processes and Structures*. Memorial University of Newfoundland, 2009. http://www.engr.mun.ca/~sbruneau/teaching/8751ocean/8751_12.pdf. Accessed February 4, 2010.

Canadian Dam Association. *Dam Safety Guidelines.* 2007.

Cassidy, J. *Question 79 (ICOLD 20th Congress).* n.d. http://hydrocoop.org/spillways1.htm. Accessed February 9, 2010.

Chanson, H. *Hydraulics of Rubber Dams Overflow: A Simple Design Approach*. University of Queensland, 1998. http://www.uq.edu.au/~e2hchans/reprints/afmc98b.pdf. Accessed February 5, 2010.

Concrete Design Handbook. Cement Association of Canada. Third Edition, 2006.

Design Criteria for Concrete Arch and Gravity Dams. United States Department of the Interior Bureau of Reclamation. US Government Printing Office. 1977.

Falvey, H.T. *Hydraulic Design of Labyrinth Weirs.* ASCE Press, 2003

Handbook of Steel Construction. Canadian Institute of Steel Construction. Ninth Edition, 2007.

Khatsuria, R.M. *Hydraulics of Spillways and Energy Dissipators.* Dekker, 2005. New York.

Munshi, J.A. *Rectangular Concrete Tanks* . Revised Fifth Edition. Portland Cement Association, 1998

Lake Outlet Dams. State of Minnesota, Department of Natural Resources, 1999. http://files.dnr.state.mn.us/waters/publications/lakeoutlet.pdf. Accessed February 9, 2010.

Lake Salinda Dam Improvments. City of Salemin, n.d. http://www.cityofsalemin.com/departments/Lake_Salinda/sal_7.jpg Accessed February 11, 2010.

Mays, L.W.. *Hydraulic Design Handbook*. McGraw-Hill, 1999. New York

Method and Apparatus for stoplog slot/guide sealing surface cleaning. n.d. http://www.freepatentsonline.com/7540059.html. Accessed February 9, 2010.

National Building Codes of Canada. Division B, Appendix C. 2005

Pioneering Rubber Gate Technology. Sumigate, n.d. www.sumigate.com. Accessed February 5, 2010.

Spillway Gates and Inflatable Rubber Dams. Dyrhoff, n.d. www.dryhoff.co.uk. Accessed February 5, 2010.

Smith, C.D. *Hydraulic Structures.* University of Saskatchewan, Saskatoon. 1995.

Smith, Walter. *Rattling Brook Hydroelectric Development Main Spillway Replacement Assessment.* Hatch, 2007.

Thandaueswara, B.S. *Pressure Distribution.* National Programme on Technology Enhanced Learning. http://nptel.iitm.ac.in/courses/IIT-MADRAS/Hydraulics/pdfs/Unit7/7_1.pdf. Accessed March 4, 2010.

US Army Corps of Engineers Shore Protection Manual. 1984.

Wyllie, D.C. *Foundations on Rock*. Principle, Goler Associates, Consulting Engineers. E & FN Spon. New York. 1992.

quidividiconsulting@gmail.com
(709) 699-5127