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Restoration of the Lake Anasagunticook Dam

A Major Qualifying Project Report

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Degree of Bachelor of Science

By

William D. B. Fay

Celeste N. Fay

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Approved:

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Professor Paul P. Mathisen, Advisor

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Professor Malcolm Ray, Advisor

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Professor Mingjiang Tao, Advisor

Abstract

This project analyzes the stability and safety of the Lake Anasagunticook Dam on Whitney Brook in Canton, Maine and investigates alternative designs for repair and replacement of the existing dam. Hydrologic and hydraulic analyses were performed to determine the design flood, operating heights of the river, and appropriate configuration for the dam. A hinge crest gate dam was recommended as the best solution and a final design was completed that included analysis of the structure and foundation.

Executive Summary

This project consists of preliminary design of a new dam for Lake Anasagunticook on Whitney Brook in Canton, Maine. A hydrologic analysis of the Lake Anasagunticook Dam was performed to determine the size of the design flood. A hydraulic analysis was performed to determine the operating heights of the river and determine appropriate dam sizes. The hydraulic and hydrologic analyses were checked with HMR 52, HEC MNS and HEC RAS modeling software.

The tasks required to complete this project include:

- Gathering background information on dam regulation in Maine.
- Performing hydrologic and hydraulic analysis of Whitney Brook, the dam and the downstream area.
- Completing preliminary designs and cost estimates on several design options.
- Based on the preliminary designs and cost estimates, choosing the best design option.
- Completing full structural and stability analysis, and cost estimate on best design.

Several promising design options were analyzed to determine which would result in the safest and most cost-effective design. Cost estimates of each design were based on yearly expected pricing guides for construction. A hinge crest dam was designed in detail. The project examined general theory on dam construction, dam safety regulations and dam design and then applied the knowledge through an analysis of the Lake Anasagunticook Dam.

The project report is intended to assist the Lake Anasagunticook Dam Association as they assess options for the construction of a new dam. The studies included are intended to cover a range of different dam alternatives showing preliminary designs and the advantages and disadvantages of each. The most feasible and affordable alternative was found to be the crest gate design. The crest gate dam was evaluated in more detail, with consideration to structural and foundation design. The crest gate dam is recommended for construction as a possible solution for dam restoration.

Capstone Design

This project is being used to satisfy the WPI Civil Engineering Capstone design requirement. The main requirement of the Capstone design is to solve an open-ended design problem which addresses most of the eight constraints identified by ABET. The constraints are economic; environmental; sustainability; manufacturability; ethical; health and safety; social; and political.

The design included evaluations of alternative solutions and the design of a crest gate dam. Health and safety issues were addressed through the hydraulic and structural analyses. These analyses ensured that during the design flood conditions, the dam structure will not pose a threat to the lives of those downstream and not cause damage to downstream structures. The project has helped to solve a major social issue in town which is the level of the lake. The lake is a major recreational facility in the area and because of the dam problem, the lake level has been lowered significantly. The economic constraint was addressed through the production of cost estimating models. Economics is a very important issue and the different dam designs all have cost estimates for comparison. Ethical concerns were addressed in the choice of dam site and design. The location of the proposed dam was chosen such that all residents who currently have lake front property would keep it as such and no property value losses would be incurred as a result of lost water front property.

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Chapter 1: Introduction

By the 1700‟s, waterpower from dams was used for a variety of tasks and was well established. Almost every New England river and stream of any size had at least one mill, powered by a dam. (Macaulay, 1983)

As long as there have been dams, there has also been the possibility of dam failure. During the 19th and 20th centuries, several major dam failures destroyed whole towns. In 1889, The Johnsontown flood was a result of a dam failure with a death toll of 2,209 (Johnson, 1889). In 1976, the Teton Dam failed, killing 14 people and causing millions of dollars in property damage (Interior, 2006). Many of these dams were constructed of poor material, were poorly designed or were not properly maintained. As dam failure incidents continued concerned citizens and the government created agencies for regulating the care and maintenance of existing dams along with rules for building new dams.

Lake Anasagunticook Dam was originally constructed to power local mills; however today it serves the recreational purpose of maintaining the water level of Lake Anasagunticook. The dam is perched over the town of Canton and a failure of the dam would send the water through the flood plain downtown before entering the Androscoggin River. The dam is currently in poor condition. A failure of the dam could cause loss of life and would certainly cause damage to homes, industrial or commercial facilities, secondary highways or an interruption of relatively important facilities such as the Victorian Villa elderly care facility as well as State Routes 108 and 140. (Ray, 2007)

The goal of this Major Qualifying Project (MQP) is to analyze the existing structure and if necessary to design a new, economically feasible dam that will meet all of the design criteria required by the Maine Emergence Management Agency (MEMA).

The major steps of the project include:

- Performing a literature review to get required background information.
- Studying and modeling the drainage area to estimate the design flood.
- Using the design flood to model the flow of the water over and around the dam, determining the size of the required dam
- Designing several different types of dams including full external and internal structural analysis

- Finding preliminary cost estimates for each design option.
- Determining the best dam option based on all information.

Eventually, the report may aide the Lake Anasagunticook Dam Association in its decisions on how to repair the dam. Finally, this project will be used to satisfy the WPI capstone design requirement. The design and costs for the recommended dam are included in Chapter 4.

Chapter 2: Background

This background section will provide enough basic information to understand the steps involved in determining and interpreting the methodology as well as the results. The background includes a description of the project as well as an overview of the current dam safety orders. Additionally, there is an introduction to hydrology, hydraulics and structural analysis.

2.1 Description of Lake Anasagunticook Dam

The Lake Anasagunticook Dam is located in the Town of Canton, Oxford County, Maine as shown in Figure 1.

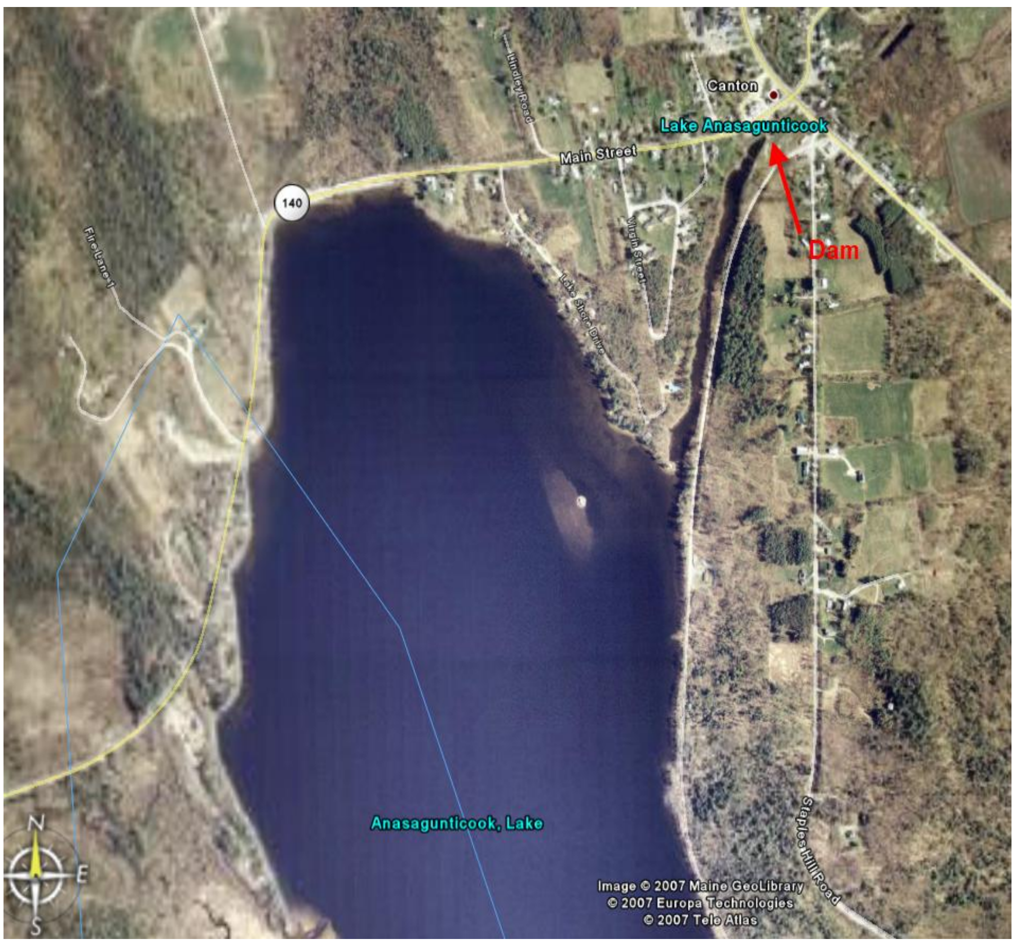

Figure 1– Map of dam location. (Google Earth)

The dam is at latitude 44°26’23”, longitude 70°18’58”, approximately 330 feet southwest of the intersection of Main Street (i.e. Maine State Route 140) and Turner Street (i.e. Maine State Route 108).

The original purpose of the dam was to provide waterpower for local mills. Canton was originally settled between 1790 and 1792. Lake Anasagunticook was initially named Whitney Pond, after a hunter who had been wounded by Indians and accidentally killed by his rescuers. The first dam was built on Whitey brook around 1849. (Lake Anasagunticook Association)

The existing dam at Lake Anasagunticook is approximately 100 years old. The dam is at the outlet of Lake Anasagunticook and impounds 580 acres of surface area. The State of Maine, Maine Emergency Management Office (MEMA) regulates the Lake Anasagunticook Dam. MEMA classifies the dam as a significant hazard, medium size structure. The spillway is a 25-foot wide concrete gated spillway structure with four overflow sluice gates. Additionally, there are remains of a power intake blocked by a fifth gate. Three of the four gates are constructed of wooden leaves and stems while the fourth is constructed of stainless steel. The four gates are powered by a single manual chain fall attached to the steel overhead gantry frame. An overview of the dam site can be seen in Figure 2.

The earthen portion of the dam consists of a left and right embankment. The left embankment is a non-homogeneous mixture of riprap and boulders with a fill of silty-fine sand. Additionally there is a dry masonry rock-block foundation wall. A three to four foot thick layer of gravely sand with cobbles and boulders was placed on top of the embankment. The core is approximately 12 to15 feet thick and both the rock block wall and the core sit on bedrock. The right bank extends 150 feet upstream from the dam with a crest of 398 msl (mean sea level, i.e. stream elevation) to between 404 msl and 406 msl. The surface of this embankment is relatively clear for approximately half of its length however, it becomes overgrown toward the upstream end of the embankment. The embankment surface approximately 40 feet from the stream has a covering of cobbles and boulders. The steep slope, located directly adjacent to the stream is covered in “spotty” riprap (the thickness being undeterminable due to its non-uniformity). The fill at the top of the slope is topsoil over approximately six feet of gravely sand. The gravely sand appears to be a non-homogeneous fill with poor soil characteristics. (Wright - Pierce, 2007) Photos of the existing dam site are shown below in Figures 2 through 11.

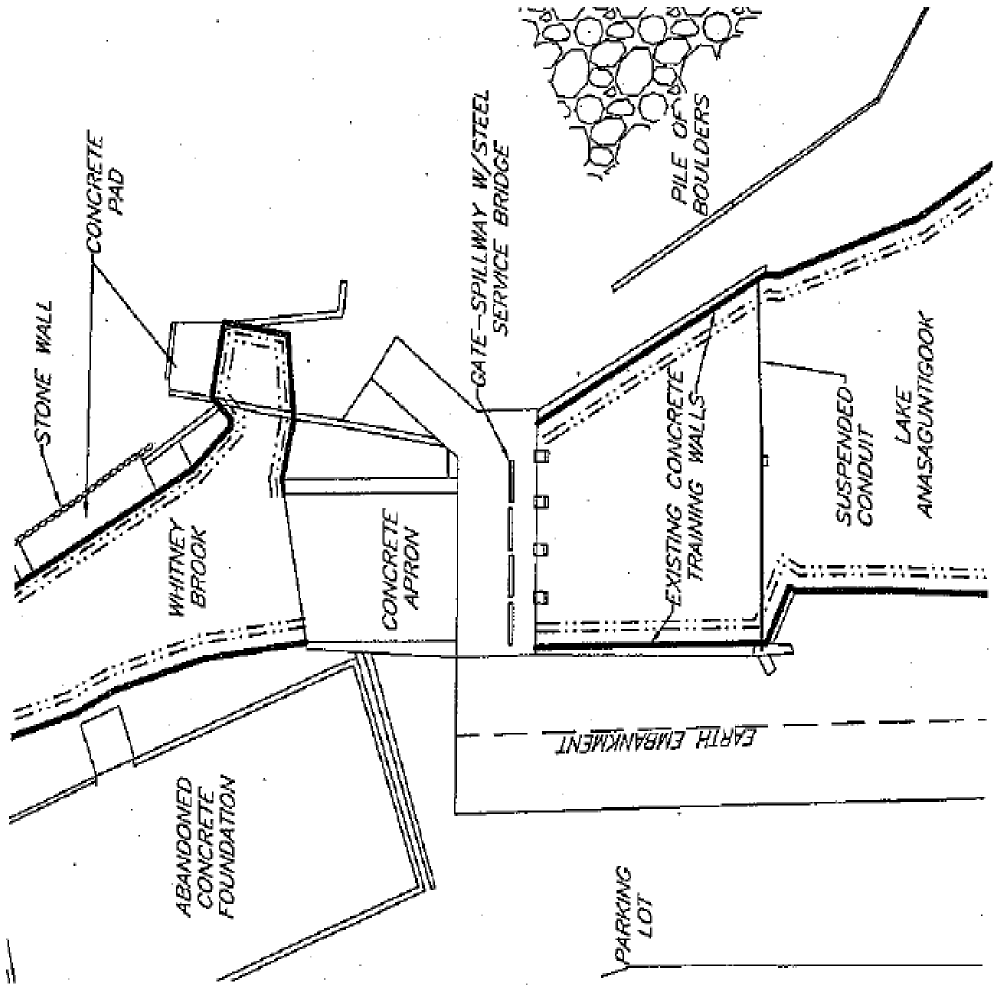

Figure 2 - Overview of the existing dam site. (Wright-Pierce, 2007)

Figure 3 - Upstream of existing spillway.

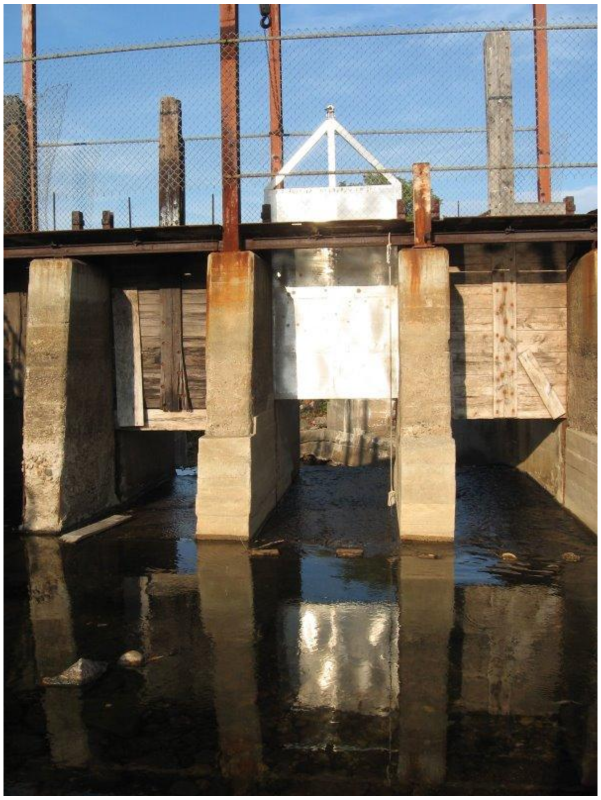

Figure 4 – Upstream side of dam, looking at gates.

Figure 5– Existing left embankment.

Figure 6 – Looking upstream at the existing spillway.

Figure 7 – Existing downstream channel.

Figure 8 - View of the existing left embankment.

Figure 9 – View upstream of spillway including the abandoned bridge foundation piers.

Figure 10 – Overview of the lake.

Figure 11 - Illustration of low water levels in the summer of 2007.

2.2 Authority

Several agencies regulate aspects of dam maintenance, operation and construction. The regulating authority depends on the purpose of the dam. The Lake Anasagunticook dam is under the jurisdiction of MEMA. MEMA uses the United States Army Corps of Engineers (USACE) regulations for the engineering aspects of design and safety of dams. Other agencies or groups who need to be satisfied with the design are the Town of Canton and the Canton Water District as described below.

2.2.1 State of Maine

a. MEMA is responsible for dam safety in Maine. Title 37-B, Chapter 24 of the Maine State Statues gives the authority to the State Dam Safety program and describes how it is set up, regulated, and administered. For regulations and specifications related to dam safety, the statute refers to the United States Army Corps of Engineers' standards. (See http://janus.state.me.us/legis/statutes/37-b/title37-bch24sec0.html)

b. The Maine Department of Environmental Protection (Maine DEP) is responsible for the protection of environmental quality in the State of Maine. Maine DEP is charged with enforcing water level management plans for lakes impounded by dams. In addition, Maine DEP is involved in the permitting process for construction and maintenance of dams. Maine DEP document 06-096 Chapter 450 and 04-061, chapter 11 of the Maine DEP's Administrative Regulations describe the regulation of hydroelectric projects and dams. (See http://www.maine.gov/dep/blwq/docstand/hydropage.htm)

2.2.2Town of Canton

In addition to typical building and zoning requirements, the Town of Canton has a direct regulatory position in the project resulting from the ruling of Superior Court Docket CV-97-55. The court's ruling mandated that the Town review and approve of any applications for local permits required to rehabilitate the dam.
(See http://www.cantonmaine.com/canton/ad20.html)

2.2.2 Canton Water District

The Canton Water District supplies approximately 330 customers with drinking water from Lake Anasagunticook. The supply is threatened by the lowered water levels, so the Canton Water District has a direct interest in the proper operation of the dam and maintenance of appropriate water levels on the lake.

2.2.3 United States Army Corps of Engineers (USACE)

The USACE is used by MEMA as the source of engineering regulations for dam safety. The USACE has over 120 sets of engineering regulations related to civil works alone. The pertinent regulations for this project are as followed:

- ER 1110-1-8100 deals with regulations regarding laboratory investigations and testing.
- ER 1110-2-101 deals with the regulations surrounding the reporting of distress in civil works.
- ER 1110-2-110 deals with regulations regarding the evaluation of civil works projects.
- ER 1110-20112 describes regulations dealing with the required visits to construction sites by design personnel.

- ER 1110-2-1150 describes the regulations for the engineering and design of civil Works Projects.
- ER 1110-2-1156 explains the engineering regulation to dam safety organization, responsibilities, and activities.
- ER 110-2-1302 describes the engineering regulation of civil works cost engineering.
- ER 1110-2-1450 talks about the engineering regulation of hydrologic frequency investigations.
- ER 1110-2-1464 deals with the regulation for hydrologic analysis of watershed runoff.
- ER 1110-2-1806 talks about earthquake design and evaluation of civil works regulation.
- ER 1110-8-2(FR) describes the engineering regulation for the inflow design floods for dams and reservoirs.

The ER in the document title stands for engineering regulation.

(See http://www.usace.army.mil/publications/eng-regs/cecw.html)

2.3 Recent orders at Lake Anasagunticook Dam.

In December 2006, MEMA issued the dam owner a safety order, which updated a similar order from May 5, 2004. At the deadline for compliance on December 31, 2007, the order had not been complied with. The order included the following requirements:

1. "Engage a licensed professional engineer (PE), specializing in dam construction to assist in preparing a remedial action plan
2. Develop a remedial action plan with the assistance of the PE to restore the integrity and structural stability of the dam and to assure that it functions and operates in a manner that will protect public safety, including at a minimum:
    - Evaluation of causes and extent of seepage, settlement and erosion of both earthen embankments and a plan for restoring the integrity and safety of the abutments.

- A plan for removing all new fill material along the left embankment or if the PE determines that the fill is not compromising any structural integrity, a plan for stabilizing and incorporating the fill into the embankment.
- A plan for repairing and resting the four spillway gates such that they are functional and can be completely raised in a timely manner.
- Develop an emergency operational procedure for the spillway gates during a flooding situation.
- Develop a plan for reducing the height of all four spillway gates to increase the flow capacity of the spillway.
- Schedule for completing all elements by Dec. 31, 2007.

3 Complete all work in accordance with local and state permitting rules." (MEMA letter, 2007)

In a letter dated May 8, 2007 MEMA concluded that until the remedial actions discussed above were implemented, the overflow sluice gates at the dam should be left open and clear of water. (MEMA, 2007)

In a letter dated September 24, 2007, from MEMA to the dam owner, MEMA pointed out that none of the previously issued orders had been complied with. As a result, MEMA determined that the current state of the dam poses a potential but real and impending danger to life, limb or property because of flooding or potential and imminent flooding pursuant to 37-B M.R.S.A., Section 1114(2). In January 2008, MEMA referred the issue to the Maine Attorney General's Office in order to enforce the penalties cited in the original dam safety order. (Lake Anasagunticook Association, 2007)

2.4 Hydrology

Hydrology is the study of the movement, distribution and quality of water throughout the earth and thus addresses both the hydrologic cycle and water resources. The hydrology of a dam is focused on determining the amount of water expected during a reoccurring storm (such as the 500 yr. flood) and how quickly the water will reach the dam impoundment.

Hydrology encompasses many variables including climatic and soil characteristics within the drainage basin. The best method of flood determination is to make a model based on site characteristics and weather data from the National Weather Service records. The National Weather Service publishes isograph maps of storm precipitation for the United States. The maps have different return frequencies and duration. (See http://www.nws.noaa.gov/oh/hdsc/studies/pmp.html, 2008) The characteristics of a drainage area such as the size, shape and elevations can be derived from United States Geological Survey (USGS) topographical maps. Electronic USGS topographical mapping programs are available and much easier to use than paper USGS topographical maps. The model used to calculate the possible maximum precipitation (PMP) was HMR-52. HEC-HMS was used to transform the possible maximum precipitation into a possible maximum flood (PMF). (USACE EM 1110-2-1415, 1993)

The results from the HEC-HMS hydrology analysis where checked against the rational method of storm runoff analysis as well as the Wright-Pierce 2007 dam reconstruction study PMF flow.

2.5 Hydraulics

Hydraulics deals with the mechanical and physical properties of liquids. In this case, the liquid is water. The interest here is how the water will act upstream of the dam, at the dam, and downstream of the dam during different flow conditions. The goal is to build the dam such that during the design flood the spillway will be able to pass the total volume of water without overtopping the embankments. The model used for this analysis is HEC-RAS. HEC-RAS is based on basic hydraulic equations for open channel flow.

Open channel flow is based upon analyzing the characteristics of water flow such as the flow rate, the depth and the velocity. The relationships among these different characteristics at different cross sections of the channel, are analyzed using basic flow concepts such as Manning's equation and the Froude number. Manning's equation relates the slope, hydraulic radius and friction of the channel to determine the velocity of the water flowing the channel. The Froude number compares the velocity of the river flow in a cross-section with the critical velocity for the reach. When the Froude number is less than one, the water flow has no opportunity to accelerate past the critical velocity of the channel. It will be in a slow deep state also known as subcritical flow. When the Froude number is greater than one, the flow in the channel has been able to accelerate more than the water downstream of it. It will create shallow turbulent water known as supercritical flow. With this information, flow profiles can be assigned to each cross section of the channel and the flow can be identified by type. This is an important step in determining how the open channel flow is behaving at any particular location along the stream.

Due to the complexity of Whitney Brook's geometry and flow conditions, HEC-RAS was used to calculate the river stage (water surface elevation) for different flood flows. Hydraulic equations where used to check the output from the HEC-RAS to determine if the outputs where accurate.   If the dam design will pass the desired design flood with no over topping, the hydraulic analysis passes. If the dam fails the design flood, as in overtopping over its abutments, the dam fails its hydraulic analysis and a new analysis must be completed.

2.6 Basic Dam Concepts

Dams can be classified into several different categories dependent on their use, their hydraulic design and the materials of which they are constructed. During the early stages of the planning and design process, selection of the size and type of dam should be carefully considered. Generally, preliminary designs and estimates for several types of dams and their appurtenant structures are required before the selection of the most suitable and economical design is made. (Dept. of Interior, 1987) The dam types that are examined in this report are:

- Rock filled gravity dam.
- Existing concrete gravity spillway with earthen embankments and an emergency overflow spillway
- Crest hinge gates (Bascule)
- Rubber inflatable dam

A general background on each of these dams will be discussed in further detail below.

2.6.1 Gravity Dams

A gravity dam is a large solid mass dam, which is dependent on its size and weight to resist overturning and sliding forces. The dam will remain stable for overturning as long as the moment about the toe caused by the water pressure is smaller than the moment caused by the weight of the dam. The dam will resist sliding along the base of the dam as long as the weight of the dam is larger than force of sliding. Finally, as long as the material properties are designed to resist the internal forces, the toe of the dam will resist crushing. Gravity dams are classified as "solid" or "hollow". The solid form is the more widely used of the two, though hollow dams are more economical to construct. Gravity dams can also be classified as having an "overflow" spillway or a "non-overflow" type spillway. A common form of non-overflow gravity dam is the earthen embankment dam, which is made from compacted earth. The existing structure at Lake Anasagunticook has earthen embankments leading up to the concrete spillway on either side. Earthen embankments are discussed in further detail in section 2.6.2. Figure 12 is a cross section of a solid gravity dam. Figure 13 is a cross section of a concrete capped, rock filled gravity dam.

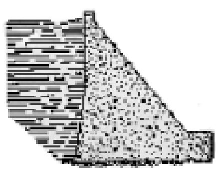

Figure 12 – Solid gravity dam. (Graham, 1997)

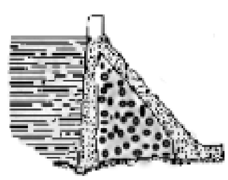

Figure 13 – Concrete capped, rock filled gravity dam. (Graham, 1997)

2.6.2 Earthen Embankments

An earthen embankment is a raised impounding structure made from compacted soil. When designing an earthen embankment, there are generally two types, homogeneous embankments and zoned embankment. A homogeneous embankment is composed of one kind of material (except for slope protection such as riprap). The material used must be impervious to provide an adequate water barrier. In addition, the slopes must be moderately flat for stability and ease of maintenance. A zoned embankment has a central impervious core, flanked by zones of more pervious material called shells. These pervious zones or shells enclose, support, and protect the impervious core.

An earthen embankment must be designed to resist any loading that may develop during the life of the structure. Other than overtopping caused by inadequate spillway capacity, the three most critical conditions that may cause failures of embankments are differential settlement, seepage and shearing stresses. The differential settlement within the embankment or its foundation can be due to shifting in materials, a variation in embankment height or compression of the foundation strata. Differential settlement may cause the formation of cracks through the embankment that are parallel to the abutments. These cracks may concentrate seepage through the dam and lead to failure by internal erosion. Seepage through the embankment and foundation may also cause piping within the foundation of the embankment. This will result in sliding of the embankment or its foundation, which displaces large portions of the embankment. Whether evaluating an existing embankment or designing a new one, the stability of an embankment and its side slopes depend on: construction materials; foundation conditions; embankment height and cross section, normal and maximum water levels and the purpose of the embankment. (Dept. of Interior, 1987)

To properly control seepage in embankment dams, it is important that the different layers of soil that make up the embankment be properly designed. The core of the dam is impervious and designed to provide resistance to the seepage. This creates the upstream reservoir. The outer pervious layers of soil provide stability for the smaller impervious layer. Soils vary greatly in permeability and even ideal soils are porous and cannot completely prevent seepage through the core. There are several factors involved in the overall porosity of the dam. The consistency of the reservoir level, the magnitudes of the permeability of the core material, the amount of pore water pressure and time all affect the rate of seepage and seepage forces in an embankment. (Dept. of Interior, 1987)

2.6.3 Hinge Crest Gates

Hinge crest gates are known by a variety of names including Bascule, Pelican or flap gates – see Figure 14. Generally, the gates are hinged at the base of the dam to a sill. They are raised to retain pool levels and lowered to pass flood flows. The plate is reinforced with vertical and horizontal members and is fitted with hinges. The gates usually seal at the base and sides when raised to retain water. The simplest type of hinge crest gate is the flat plate hinged at the bottom and operated by a hydraulic cylinder connected to the top of each gate section. The hinge crest gate with hydraulic cylinders can be made in longer lengths with multiple sections and total 200 ft. or more in length. Hinge-crest gate dam sills and piers are usually made of reinforced concrete. (USACE EM 1110-2-2607, 1995)) See Appendix F for an example of a hinged crest gate dam.

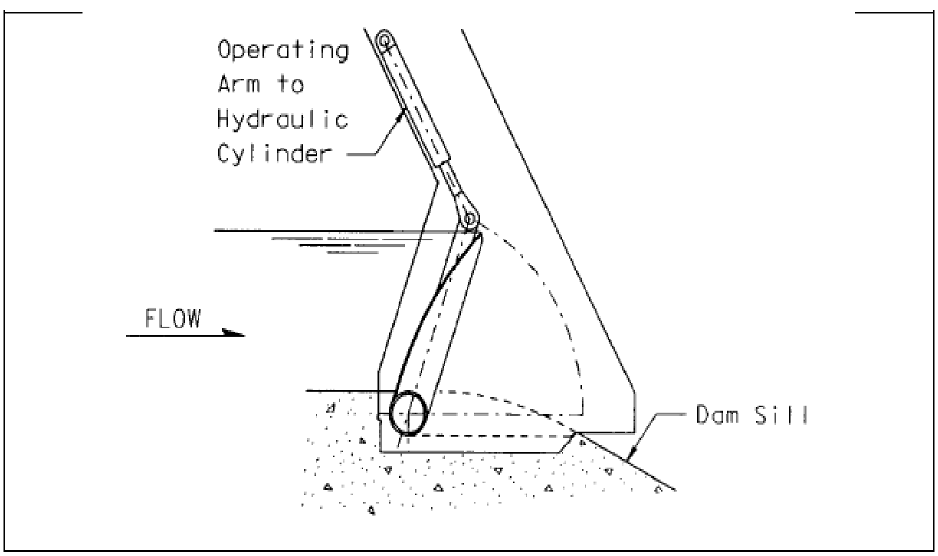

Figure 14 - Hinge crest gate. (USACE EM 1110-2-2607, 1995)

Another form of the hinge crest gate is the Wicket Gate – see Figure 15. Wicket-type gates have been used for over 100 years. The idea is very similar to that of the simple hinge crest gate. The difference is that the gates are held up in position with a prop or strut, which slides in a rack. This allows the cylinder pistons to be retracted. This means that during flood conditions the dam will become very close to an open channel. Wickets are traditionally constructed of steel framing with timber leafs. Wickets, which are hinged at the base, have the advantage of simplicity and cannot be "flipped" up by thrust from the backpressure of the tailwater, then be held partially up by river currents. The advantage of Wicket gates are low initial cost of construction, lighter weight and variability in controlling pool. The disadvantage is the maintenance of the timbers. Again the sill is made of reinforced concrete but piers are not necessary and do not have to be included in the design. The lengths of the sill sections are controlled by cracking and constructability constraints. (USACE EM 1110-2-2607, 1995)

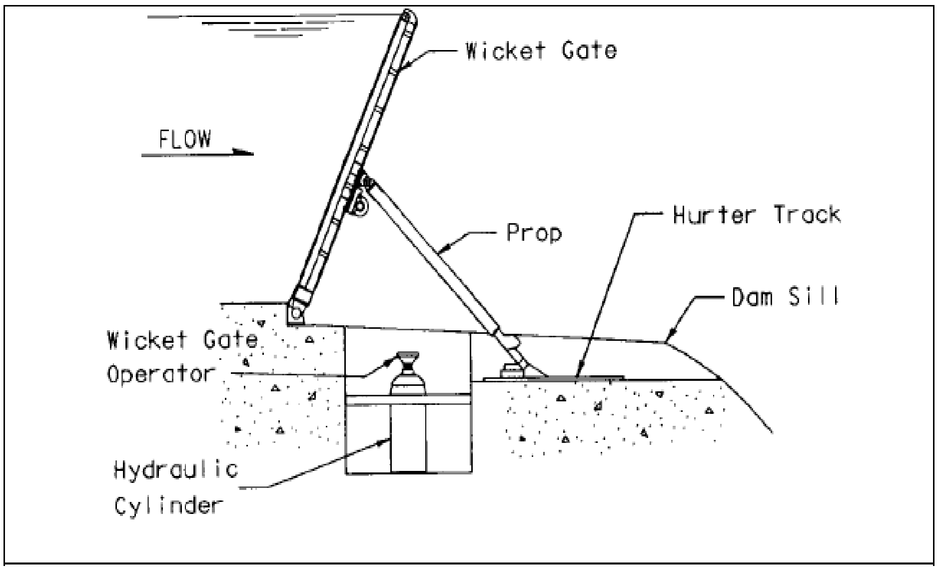

Figure 15 - Wicket gate dam. (USACE EM 1110-2-2607, 1995)

2.6.4 Rubber dam

An expensive, easy-to-install option for a new dam located upstream of the existing dam is a rubber, collapsible spillway such as the one seen in Figure 16.

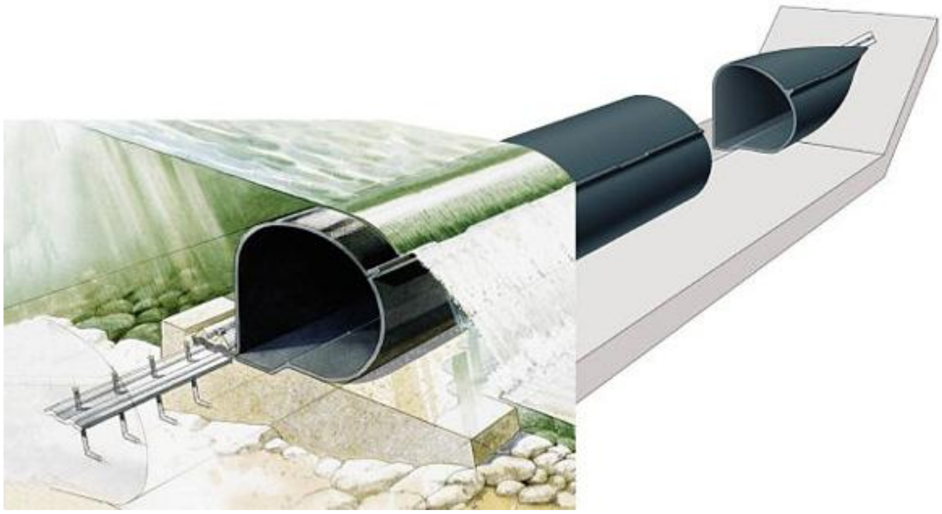

Figure 16 – Bridgestone-Firestone inflatable rubber dam.

The site is very suitable for a rubber dam. The bedrock at the site is located very close to the streambed surface. This is important because a rubber dam is secured by its foundation, which is generally a concrete sill. The rubber dam can collapse automatically with an air pressure blow out plug and reduce the dam's hydraulic profile during a flood to almost nothing as seen in Figure 17. This will help reduce the floodwater elevations and accordingly reduce the dam's necessary hydraulic height to pass flood flows. Over 1000 Bridgestone-Firestone rubber dams have been installed around the world and there are countless other manufactures of rubber dams.

Figure 17 - Rubber dam deflated.

2.7 Structural Analysis

The objective of the structural analysis is to find the materials and size of the dam that will ensure the stability of the dam under a wide range of conditions. The dam is subject to random events such as floods, waves, earthquakes, ice formation and other natural phenomena. The structural analysis can be broken into two portions, external and internal stability. The external forces are those that are directly applied to the dam and include: water pressure; earth pressure; ice pressure; earthquake forces; wind pressure; wave pressure; weight of the dam; weight of the foundation; and reaction of the foundation. The structural analysis begins by evaluating the stability of the preliminary dam section with the external forces applied. The shape and size of the dam are the unknown parameters and will be solved for to ensure stability against the external forces. The internal forces are forces that the materials of the dam must resist. For example if the dam is made of concrete, the forces on the toe of the dam must be calculated internally to ensure that the molecular strength of the concrete is strong enough to withstand the immense pressures of the dam at the toe and not crumble under its own weight. Components of the dam such as timber size or concrete strength are chosen based on internal stress calculations and the limitations of the materials.

A factor of safety is used to provide a design margin over the theoretical design capacity to allow for uncertainty in the design process. The uncertainty may come from calculations, material strengths and material quality. The factor of safety must relate to the strength, stability and durability of the structure with consideration to magnitude of economic and personal loss that would result from its failure. The aim of the engineer must be to reduce the number of uncertainties, in both loading on the dam and the means by which the dam and the foundations withstand such loads. (Graham, 1997)

2.8 Cost Estimating

Construction cost estimating is the determination of probable construction costs of any given project. When deciding between different designs, the cost of a project will play an important role in that decision making process. Many items influence and contribute to the cost of a project and each item must be analyzed, quantified and priced.

Because the estimate is prepared before the actual construction, much study and thought must be put into the construction documents. Generally, the estimate for a dam will include construction materials, labor, machinery and special equipment, permitting, engineering design, administration/management of project and if necessary a temporary dam to enable construction. (Dagostino, 2003)

2.9 Visual Inspection of the Existing Dam

Lake Anasagunticook Dam was inspected on several occasions by both MEMA and Write-Pierce. The dam was visited and inspected on November 17, 2007 by Will Fay and Celeste Fay to survey the project. The following inspection findings are a summary of the important findings by all three parties.

2.9.1 General Findings

The dam was found to be in overall poor condition. The general concerns include seepage, settlement and erosion of the left earthen embankments, the decrease in stability due to the poor quality fill dumped on the top of the embankment, the non-functioning spillway gates, the lack of an Emergency Action Plan (EPA), and the deficient spillway capacity. (MEMA Safety Order, Dec. 4, 2006)

2.9.2 Dam Site

The dam embankments are in poor condition with signs of erosion, seepage and sinkholes. (Figure 8) The June 2006 MEMA dam safety order described the upstream left embankment as having settlement of the embankment along the spillway retaining wall and settlement of embankment along the concrete retaining approach wall. The downstream left embankment has a sinkhole and settlement in the embankment along the outside of the stone retaining wall. A 60 foot long rut along the embankment 5 to10 feet long was found as well as a 15-foot section of collapsed stone retaining wall 90-feet upstream of the spillway. (MEMA Dam Safety Order, 2006) The existing ground surface around the right embankment is relatively clear from the stream to about half way to the abandoned bridge. The other half of the embankment is overgrown with small bushes and trees. The embankment surface approximately 40 feet from the stream has a covering of cobbles and boulders. The steep slope directly adjacent to the stream is covered in “spotty” riprap. The thickness of the riprap was undeterminable due to the non-uniformity of the material. (See Figure 9) The fill at the top of the slope is topsoil with approximately six feet of gravely sand that appears to be a non-homogeneous fill. The MEMA dam safety order described the right embankment upstream as being deficient due to settlement of the embankment at the spillway concrete retaining wall. The downstream right embankment was described as having seepage from the toe area, about 60 feet from the spillway and uncontrolled leakage of approximately 50 to 100 gallons per minute before the lake level was lowered. (MEMA, 2007)

All of the wooden gates on the spillway have been reinforced for strength however, one of the three is still in poor condition. (See Figure 4) The stainless steel gate is in good condition. The gate guides only extend approximately one foot to two feet above the spillway deck meaning that the gates can only be opened between one and two feet or they must be taken completely out. There appears to be minor spalling in the concrete that should be repaired. Overall, the concrete spillway structure appears to be in good condition. (See Figures 3 & 6) The MEMA order stated that the spillway was deficient due to gate overflow restrictions and leaks in the guides. However, it is structurally sound and stable. In addition, it is questionable if the spillway could pass the USACE design flood inflow. (Wright - Pierce)

2.9.4 Downstream Area

Immediately downstream of the dam is a dry laid masonry lined channel approximately 12 feet wide and 10 feet deep. (See Figure 7) There is significant undermining and degradation of the concrete on the right side of the channel which if collapsed would affect the discharge capacity of the spillway. Approximately 175 feet downstream of the dam on the left side is an empty building that would likely be seriously affected by flooding due to a failure of the dam. Approximately 300 feet downstream is the first of several concrete box culverts with roadways passing over them. These box culverts cause water to back up to the dam during high river flows and affect the spillways discharge capacity.

2.9.5 Reservoir Area

The reservoir area of Lake Anasagunticook is approximately 580 acres and it has approximately 9800 acres of drainage area. (See Figure 10) The slopes leading to the pond are mild. The lake is located in a natural bowl with mountainous terrain surrounding the area. The lake is used for recreational purposes and has many seasonal and year round houses along the shoreline. The lake is also the water supply for the 330 customers of the Canton Water District.

Chapter 3: Methodology

The major tasks required to solve the design problem at Lake Anasagunticook and the order in which they are completed are shown in Figure 18.

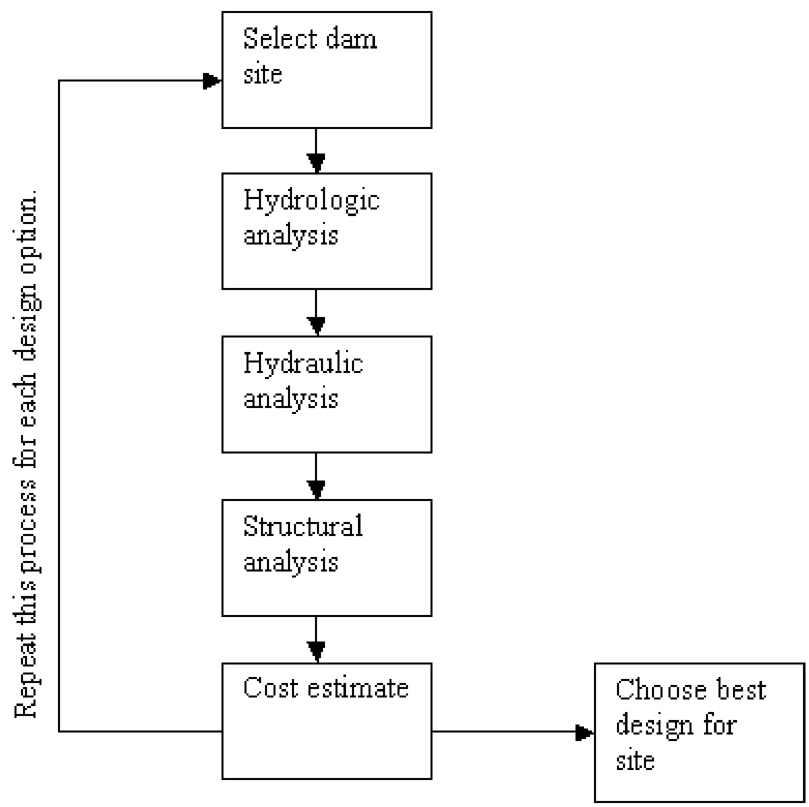

Figure 18 – Flowchart of Design Methodology

In the Figure above, each task represents a piece of information as seen in Figure 19. Selection of the dam site will yield important characteristics of the location that will be required for the hydrologic analysis. The hydrologic analysis will yield the size of the design flood, which is a key piece of information for the hydraulic analysis. The hydraulic analysis will deliver information about river heights and locations of overtopping during the design flood. The hydraulic information is used in the structural analysis to determine the height of water during flooding conditions. The structural design will determine the size, shape and types of materials required to maintain equilibrium. With the information from the structural design, a cost estimate can be completed.

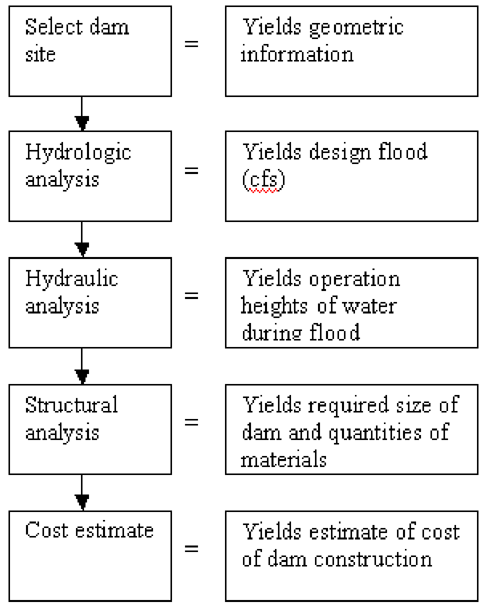

Figure 19 – Equivalent Yield

3.1 Hydrologic Analysis

The hydrologic analysis is required to determine the volumetric flow rate of the design flood. The analysis investigates how certain topography, soil characteristics, storm frequency, and storm duration affect the quantity of the possible maximum flood (PMF) flow for the drainage area   The PMF is used to find a safe design flood for the spillway. The design flood for the spillway matters greatly because it will determine the period of return and determine the statistical probability that a dam will overtop and fail. A flowchart of the steps required to find the PMF is shown in Figure 20.

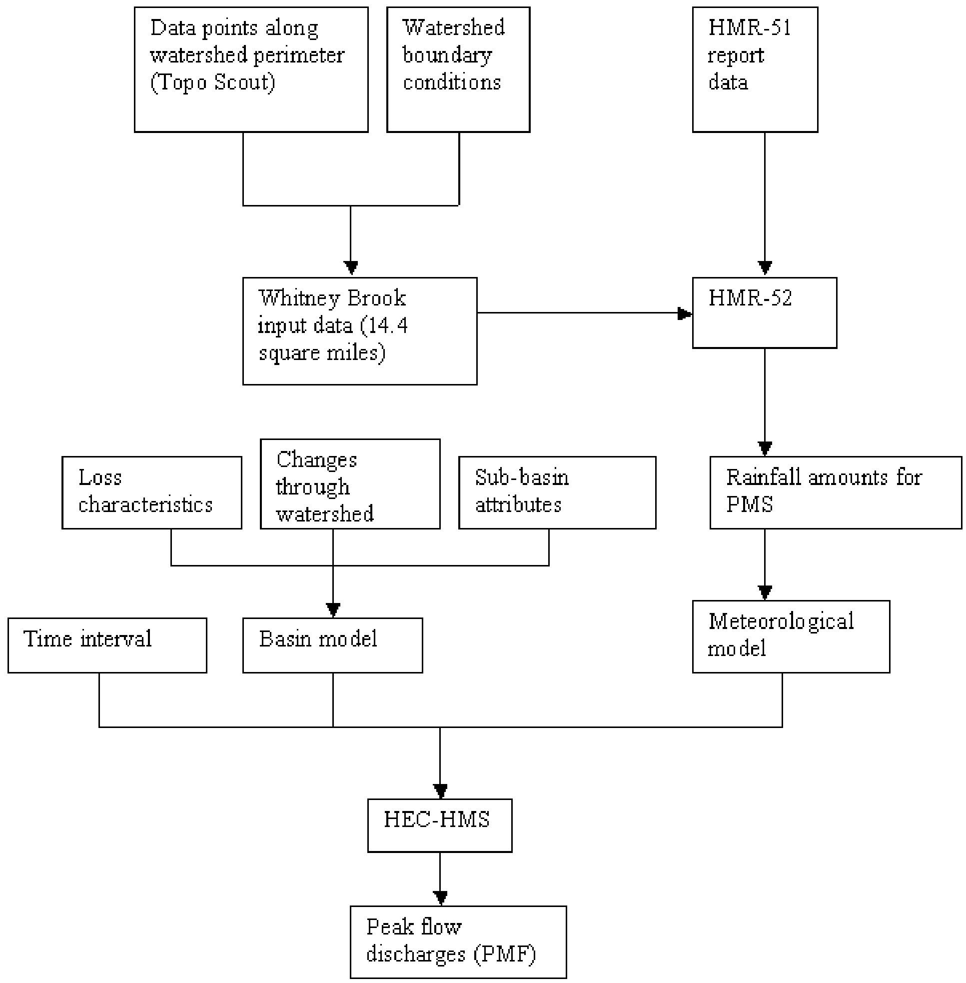

Figure 20 – Flowchart of Hydrology

3.1.1 Drainage Area Characteristics

First, to find the PMF, basic characteristics of the drainage area were found. These characteristics include the surface area of the drainage basin, the slope, the topography, the soil characteristics, and rainfall frequency maps from the NWS. Topo-Scout, a digitalized United States Geological Survey topographical mapping program was used to measure the geometric characteristics of the drainage area. These consist of the slopes, slope lengths, drainage area size, and the orientation of the drainage area. Figure 21 shows the Anasagunticook Lake drainage basin mapped out in Topo-Scout. The program includes detailed maps of the Lake Anasagunticook drainage area with contour lines. Using several of the program's tools and an Excel spreadsheet, necessary information about the drainage area can be gathered.

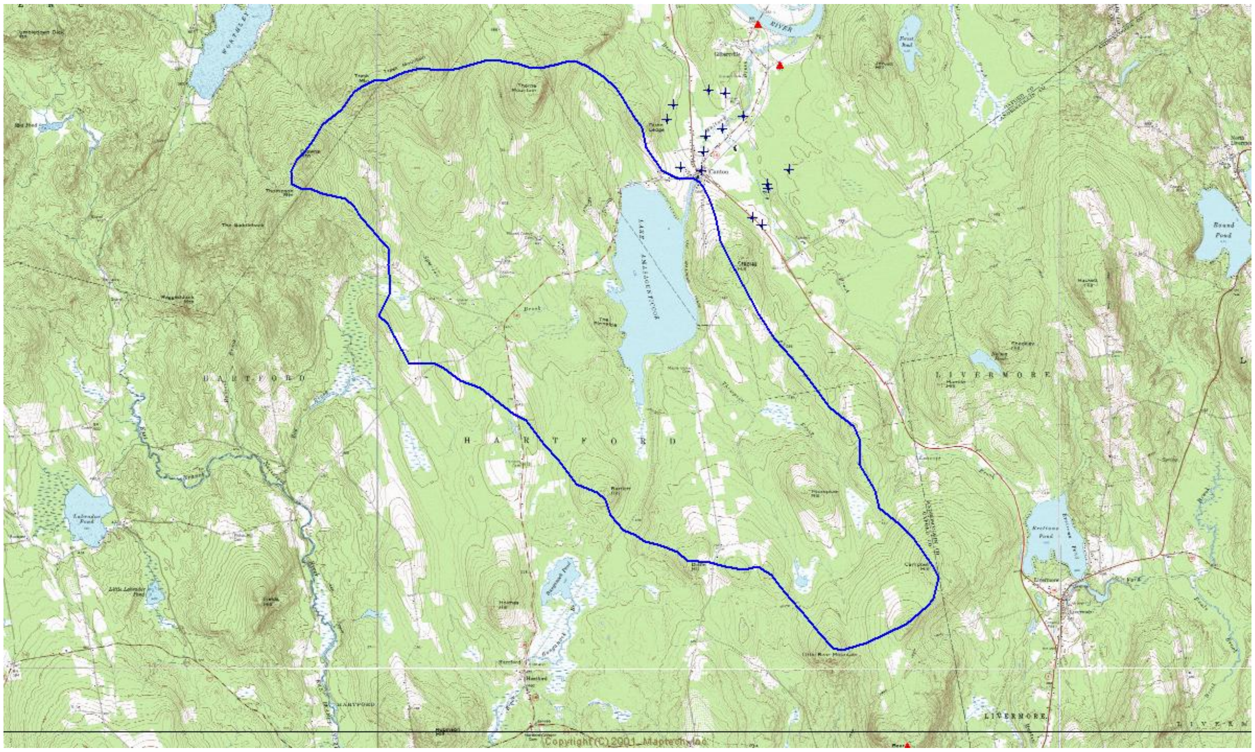

Figure 21 –Lake Anasagunticook Dam Drainage Area (Topo Scout)

Average soil types for the area can be found from the United States Department of Agriculture website. In addition, a list of infiltration rates for different soils was used to assign an average infiltration value for the entire drainage area. Infiltration rate is the rate at which rainfall and runoff is absorbed into the soil.

3.1.2 Determination of the Probable Maximum Precipitation

HMR-52 is a USACE program designed to calculate the probable maximum precipitation of a drainage area. The probable maximum precipitation is the maximum anticipated rainfall a drainage area can be capable of receiving. HMR-52 uses the drainage area characteristics discussed in the previous paragraph to calculate a rainfall graph, also known as a hyetograph, for the possible maximum precipitation. HMR-52 helps engineers compute basin-averaged precipitation for Probable Maximum Storms (PMS). Additionally, it corresponds to the spatially averaged Probable Maximum Precipitation (PMP) for a basin or combination of watershed sub-basins.

To begin, the drainage basin image is printed on a 10 by 10 graph paper with a plot scale, as seen in Figure 21. Arbitrarily, a coordinate axis system is set up with the drainage area basin marker coordinates in inches. This will produce the drainage area in a matrix format that the HMR-52 program can recognize and use to perform calculations. The tabulated coordinates of the Lake Anasagunticook drainage basin are shown in Table 1.

Table 1 – Lake Anasagunticook Drainage Area Division Coordinates

HMR 52 Watershed Shape Factors
Probable Maximum Flood
Anasagunticook Lake Dam
Canton, Maine

Location	X	Y	X-Mile	Y-Mile
1	4.5	7.45	3.788	6.270
2	5.23	5.91	4.402	4.974
3	6.28	4.71	5.286	3.964
4	6.25	3.4	5.260	2.862
5	6.55	3.875	5.513	3.261
6	7.08	3.39	5.959	2.853
7	7.05	2.97	5.934	2.500
8	6.69	2.65	5.631	2.230
9	6.19	2.4	5.210	2.020
10	5.82	2.5	4.899	2.104
11	5.31	3.2	4.469	2.693
12	4.5	3.4	3.788	2.862
13	3.7	3.79	3.114	3.190
14	3.51	4.1	2.954	3.451
15	2.93	4.39	2.466	3.695
16	2.68	4.875	2.256	4.103
17	3.36	5.1	2.828	4.293
18	2	5.2	1.683	4.377
19	1.85	5.375	1.557	4.524
20	1.36	5.6	1.145	4.713
21	1	6.08	0.842	5.117
22	0.64	6.675	0.539	5.618
23	0.2	7.15	0.168	6.018
24	0.22	7.7	0.185	6.481
25	0.42	7.875	0.354	6.628
26	0.7	8.41	0.589	7.078
27	1.6	8.7	1.347	7.323
28	2.33	8.75	1.961	7.365
29	2.84	8.45	2.390	7.112
30	3.12	8.78	2.626	7.390
31	3.325	8.79	2.799	7.398
32	3.65	8.5	3.072	7.154
33	3.9	8.1	3.283	6.818
34	4.18	7.5	3.518	6.313

The drainage basin storm factors were determined next. Hydro-Meteorological Report N. 51 from the National Weather Service is used to obtain depth-area-duration values from the 10, 200, 1000, 5000, 10,000, and 20,000 square mile curves for the drainage area's longitude and latitude A sample storm map from the Hydro-Meteorological report 51 is shown in Figure 22.

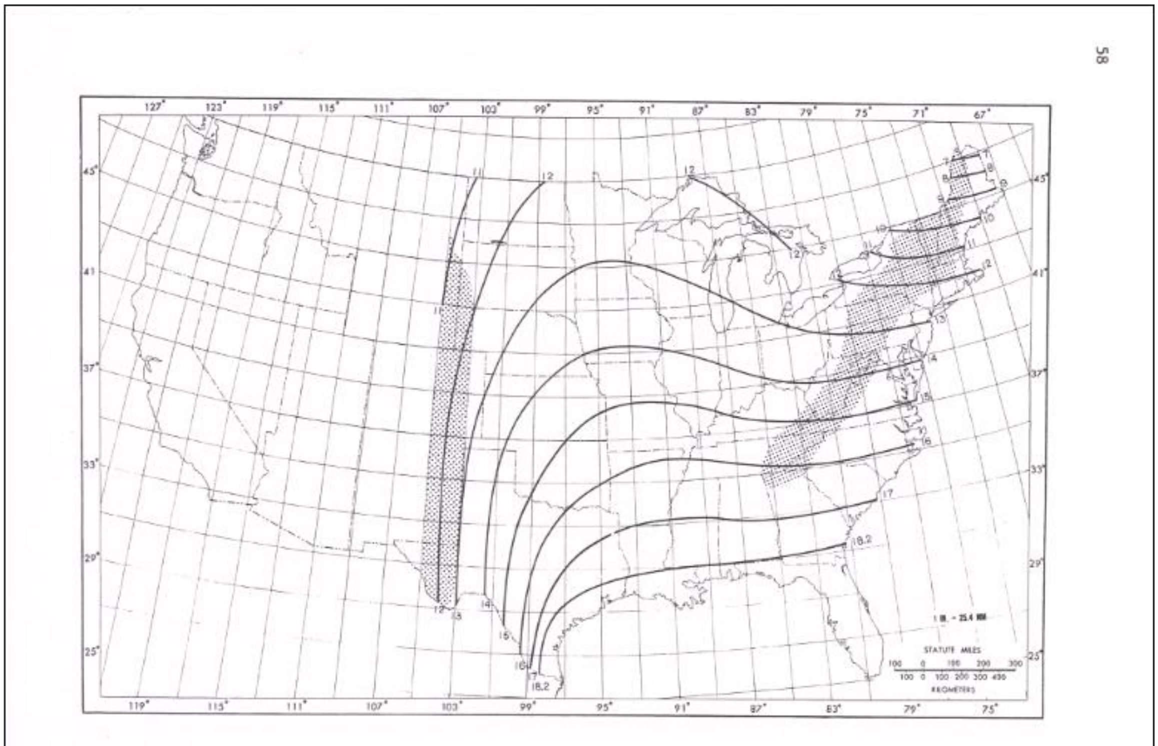

Figure 28.--All-season PMP (in.) for 6 hr 1,000 mi^2 (2,590 km^2).

Figure 22 – Hydro Meteorological Report No. 51, PMP Map (NSW, 1978)

Table 2 – HMR-52 Input data

```
                         H     H  M      M  RRRRRR    5555555    22222
                         H     H  MM    MM  R     R   5         2     2
                         H     H  M M M M   R     R   5               2
                         HHHHHHH  M  M  M   RRRRRR    555555         2
                         H     H  M     M   R  R            5      2
                         H     H  M     M   R   R     5     5    2
                         H     H  M     M   R    R     55555    2222222
1                   HEC PROBABLE MAXIMUM STORM (HMR52) INPUT DATA
PAGE 1

     LINE       ID.......1.......2.......3.......4.......5.......6.......7.......8.......9......10

        1       ID Anasagunticook Lake Dam Study
        2       ID HMR52 Probable Maximum Storm Input Data ***PMS***
        3       ID Whitney Brook, Canton, Maine

        4       BN   TOTAL
        5       ID Drainage Basin Geometry - Calculate Storm Over Uncontrolled Basin
        6       BS     1.0
        7       BX   3.788   4.402   5.286    5.26   5.513   5.959   5.934   5.631    5.21   4.899
        8       BX   4.469   3.788   3.114   2.954   2.466   2.256   2.828   1.683   1.557   1.145
        9       BX   0.842   0.539   0.168   0.185   0.354   0.589   1.347   1.961    2.39   2.626
       10       BX   2.799   3.072   3.283   3.518
       11       BY   6.270   4.974   3.964   2.862   3.261   2.853     2.5   2.230    2.02   2.104
       12       BY   2.693   2.862    3.19   3.451   3.695   4.103   4.293   4.377   4.524   4.731
       13       BY   5.117   5.618   6.018   6.481   6.628   7.078   7.323   7.365   7.112   7.390
       14       BY   7.398   7.154   6.818   6.313
       15       PL       2
       16       ID HYDROMETEOROLOGIAL DATA FROM HMR No. 51
       17       HO     230
       18       HP      10      21      24      26      30      32
       19       HP     200      14      17      19      22      23
       20       HP    1000      10      13      16    18.5    19.5
       21       HP    5000     6.5     9.3    11.5      14      15
       22       HP   10000       5     7.8    10.2    12.3    13.5
       23       HP   20000     3.7     6.5     8.5      11      12
       24       ID STORM SPECIFICATIONS
       25       SA                      12
       26       ST      60    .335             1.0
       27       ZZ
```

The possible maximum storm precipitations, storm frequency, storm length, drainage basin geometry, and the drainage basin orientation were inputted into HMR-52 as shown in Table 2. Then the program was run and a possible maximum precipitation for the drainage basin was computed. A detailed description of the HMR-52 procedure can be found in Appendix A.

3.1.3 Determination of the Possible Maximum Flood

With the possible maximum precipitation outputted from HMR-52, the flow rate of the possible maximum flood can be determined. The possible maximum flood will determine the safe size of the spillway structures for our dam, so that overtopping will not occur.

Once the PMP is known, another USACE program, HEC-HMS is used to calculate the resulting flood hydrograph (graph of flood flow versus time) from the PMP obtained with HMR-52. HEC-HMS is used to simulate the surface runoff response of a river basin to precipitation by representing the basin as an interconnected system of hydrologic and hydraulic components. This program will produce runoff hydrographs for complex watershed networks using unit hydrograph or kinematic wave methods and by incorporating reservoir and channel routing procedures. The program will allow various methods for calculating rainfall hyetographs, basin unit hydrographs and watershed loss rates. A hyetograph is a graphical representation of the amount of precipitation that falls through time.

HEC-HMS can calculate the PMF flow using different methods to mathematically describe how rainfall will flow in a drainage area and then transform itself into stream runoff. The Soil Conservation Service (SCS) method was used in our model. First, the SCS parameters used in the HEC-HMS model needed to be calculated. The drainage basin topography, land use, and soil types were analyzed. These characteristics give the SCS curve number, which is intended to show how rainfall interacts with a drainage basin's physical characteristics. A value was assigned for the Anasagunticook basin’s curve number from Chart 3. Therefore, an approximate average SCS curve number was estimated. This value ranged from 55 to 70 depending upon percentage of urbanization of the watershed and the predominate soil type in the area.

Table 3 – SCS CN Value Charts (Chow, 1957)

Cover Type	Condition	Curve Numbers for Hydrologic Soil Group A	B	C	D
Open Space (lawns, parks, golf courses, cemeteries)	Poor (grass cover < 50%)	68	79	86	89
	Fair (grass cover 50% to 75%)	49	69	79	84
	Good (grass cover > 75%)	39	61	74	80
Impervious Areas (paved areas, roofs, etc.)		98	98	98	98
Urban (Commercial)		89	92	94	95
Urban (Industrial)		81	88	91	93
Residential	1/4 acre lot	61	75	83	87
	1/2 acre lot	54	70	80	85
	1 acre lot	51	68	79	84
	2 acre lot	46	65	77	82
Fallow Land (farm land not actively cultivated)	Bare Soil	77	86	91	94
	Crop Residue Cover	74	83	88	90
Row Crops	Straight Row	67	78	85	89
	Contoured	70	79	84	88
Pasture (continuous grazing)		49	69	79	84
Meadow (protected from grazing, mowed for hay)		30	58	71	78
Brush		35	56	70	77
Woods/Grass Combination (orchard or tree farm)		43	65	76	82
Woods	Poor (heavy grazing or regular burning)	45	66	77	83
	Fair (grazed but not burned; some forest litter covers soil)	36	60	73	79
	Good (protected from grazing; litter and brush adequately cover soil)	30	55	70	77

Then the maximum retention (S) was calculated using Equation 1.

$$S = \frac{1000}{CN} - 10 \qquad (1)$$

Where: CN= Curve Number
S=Retention in inches

Next, the percent slope was determined using the Topo Scout program's profile option. Markers were set along the longest watershed path. The program automatically graphed the path elevation profile and listed its length and elevation change. The distances to the 10% and 85 % stations were calculated. The elevations at each of these stations were determined. The watershed head was calculated from the difference between the elevations of these two stations.

The lag time, is the amount of time required for the water from the farthest reaches of the drainage area to reach the study area. The lag method (Equation 2) was used to determine the time lag (L) which in turn was used to calculate the time of concentration (Equation 3).

$$L = \frac{l^{0.8} * (S+1)^{0.7}}{1900 * Y^{0.5}} \qquad (2)$$

Where:

l= Hydraulic length in feet,
Y = Slope of the watershed in percent,
S = Maximum retention.

$$T_c = \frac{5L}{3} \qquad (3)$$

Where:

L=Length
T_c =Time of Concentration in minutes

Once these parameters are known a HEC-HMS model can be constructed and a PMF flood flow determined. The input data used for the model is seen in Figure 23.

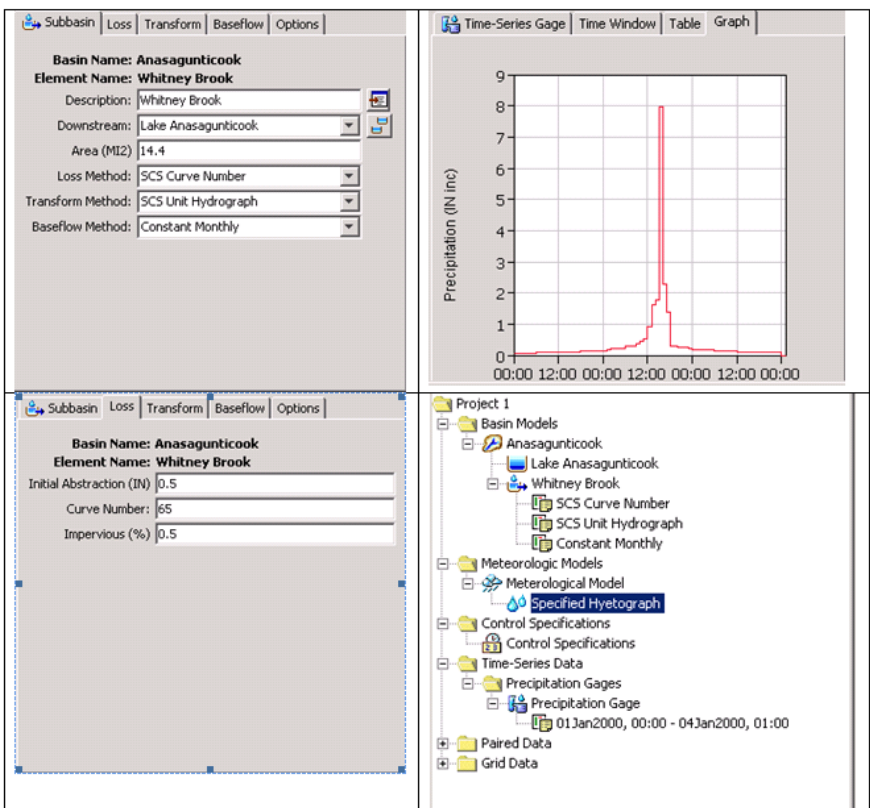

Figure 23 – HEC-HMS Input Data for Lake Anasagunticook

3.1.4 Verification of HEC-HMS Output

The HEC-HMS output was then checked using a rational method of runoff determination. The rational method is an empirical formula developed for the estimation of the peak flow from a storm on a drainage area. The following formula is used.

$$Q = CIA \tag{4}$$

Where: Q=Flow in cfs
C=Runoff coefficient
I=Rainfall intensity in per hour
A=Drainage area in acres

The rainfall intensity comes from the National HMR-52 model. The run-off coefficient C can be looked up in Table 4. Lastly, the drainage area size A is found for both the HMR-52 and the HEC-HMS. The results from this equation are checked against the HEC-HMS results.

Table 4 – Summary of Runoff Coefficients

Ground Cover	**Runoff Coefficient, c**
Lawns	0.05 - 0.35
Forest	0.05 - 0.25
Cultivated land	0.08-0.41
Meadow	0.1 - 0.5
Parks, cemeteries	0.1 - 0.25
Unimproved areas	0.1 - 0.3
Pasture	0.12 - 0.62
Residential areas	0.3 - 0.75
Business areas	0.5 - 0.95
Industrial areas	0.5 - 0.9
Asphalt streets	0.7 - 0.95
Brick streets	0.7 - 0.85
Roofs	0.75 - 0.95
Concrete streets	0.7 - 0.95

The data from the HEC-HMS was also checked against the PMF calculated from the Wright-Pierce 2007 dam redesign. They used a statistical method that compared a drainage area's size to the maximum-recorded flow low for the drainage area. Approximately 25 separate data points were used in the Wright-Pierce model. They were plotted and a trend line was established for the data. The equation of the trend line was found through a regression and the equation was used to calculate the PMF at Anasagunticook Lake.

3.2 Hydraulic Analysis

The purpose of this section is to describe the methodology used to determine the spillway discharge capacity of the current Lake Anasagunticook Dam and the discharge capacity of any replacement options. Additionally, it will determine how the water in the river channel will behave based on several different situations.

Spillway inadequacy and a resulting dam failure are based upon overtopping. If the design storm discharge overtops the freeboard of the dam, there is the potential to damage sections of the dam that are not designed to be overflow sections. In the worst-case scenario, the overtopping flows will cause the dam to catastrophically fail and release a large potentially dangerous surcharge flow into the downstream channel and potentially affect life or property. The tasks associated with the hydraulic analysis are as follows:

- Determination of the design flood
- Determination of the river channel geometry and flow characteristics
- Determination of stream flow versus river stage
- Check model results with hydraulic equations

3.2.1 Design Flood Determination

According to MEMA, dam design specifications have to meet or exceed those recommended by the USACE. The hydraulic analysis begins by classifying the dam into one of three groups. The first group of dams include those that need to pass the full PMF because their failure will cause catastrophic property damage and loss of life downstream. The second group of dams include those that will probably not cause loss of life but may cause catastrophic property damage to downstream land owners need to pass one-half of the PMF. The final group of dams do not pose a significant hazard to life or property, a justified design flood with a suitable return period should be chosen. Once the design flood is selected, the geometry of the downstream channel is found. (USACE ER-1110-8-02)

3.2.2 River Channel Geometry and Flow Characteristics

The downstream channel geometry is important to know how the spillway design flood (SDF) will flow through the channel. A cross section is made at every point in the channel where there was a significant, abrupt change in geometry or at a regular interval of approximately 500 feet. The cross section consisted of a station relative to the horizontal distance in the cross section. At every horizontal station an elevation point three pieces of information were recorded. The distance to the downstream cross section, Manning's values for the riverbanks and the location of the natural river channel in the cross section were recorded for the model. These data points were obtained from the site visit to the Lake Anasagunticook Dam site, the FEMA Flood Insurance Study for Canton and the USGS topographical maps of Canton. During the site visit, the two bridges directly downstream of the Lake Anasagunticook Dam were mapped and surveyed to determine their geometry. Separate cross sections where compiled for each of the structures with the survey data. Also, directly upstream and downstream of each structure, a cross section was made to provide a smooth hydraulic model with no jumps or odd transitions.

3.2.3 Determination of Flow versus River Stage

Due to the complexity of the channel below the Lake Anasagunticook Dam, a hydraulic modeling program was used to determine the river stage (elevation of the water surface elevation in msl) for varying water flows, up to the PMF flood flow. Therefore, a USACE hydraulic program, HEC-RAS was used. HEC-RAS is used to model water flow through complex riverine hydraulic systems and to obtain water surface elevations at specified cross sections. River channel and civil structure geometry is entered into HEC-

RAS and a steady state flow analysis is preformed. The basic computational procedure is based on the one dimensional energy equation.

Flow data from the hydrologic survey (Section 3.1) and physical characteristics such as elevations and lengths were surveyed during the site visit and were used to start the analysis. The geometric cross sections (Section 3.2.2) of the channel and the hydraulic structures were entered into the HEC-RAS program. This data was analyzed through a series of open channel equations. These equations were applied to all the cross sections simutaniously. The results of the equations at each section were compared to identify the flow profile of the channel.

Next the required spillway dimensions were found. The length of the dam will be dependent on the elevations of the embankments and the elevation of the normal water level. The average height of the dam embankments and river banks were determined as well as the average depth of the ledge on the river bottom. The water level order is set such that the elevation of the water needs to be set at a certain level. The difference in the embankment height and the elevation of the lake level order is how many feet of free board (height to over top the dam) that is available to pass the design flood. This reasoning is applicable to a solid gravity dam. However it will change slightly for a crest hinge gate dam.

For the crest hinge gate dam, the height of the dam is dictated by the water pressure on the dam. During normal conditions, the elevation of the dam will be at the water order's recommended lake elevation. However, as the volume of water increases, the dam crest is lowered in order to keep the water surface elevation steady. Depending on the volume of water, the dam will be able to fold down to the channel bottom. This means that during the design flood, the dam will be completely folded over into the channel. At this point, the dam will be approximately level with the bottom of the channel. This design has a huge advantage over the solid gravity dam because during the design flood, the spillway capacity will be much larger than a gravity dam.

Figures 24 and 25 show the geometric and flow inputs for the Lake Anasagunticook HEC-RAS model and Figure 26 is an exapmle output. The top left screen of Figure 24 shows the cross section station information, with elevations and station numbers. The bottom left screen shows the Manning's friction coefficients for each river station cross section. The top right screen shows the down stream reach lenghts from one station to the next. Lastly the bottom right screen shows an output file from the HEC-RAS program. The output is a map of the river vally and the river vally geometry. Figure 25 shows the stream flow imput for the HEC-RAS model. Required flow and the initial river station at the river start are inputted into the model. Figure 26 show the graphical output option for HEC-RAS. A graphical model is constructed and the river stages are represented as the blue surface in the model. The gray blocks represent the dam.

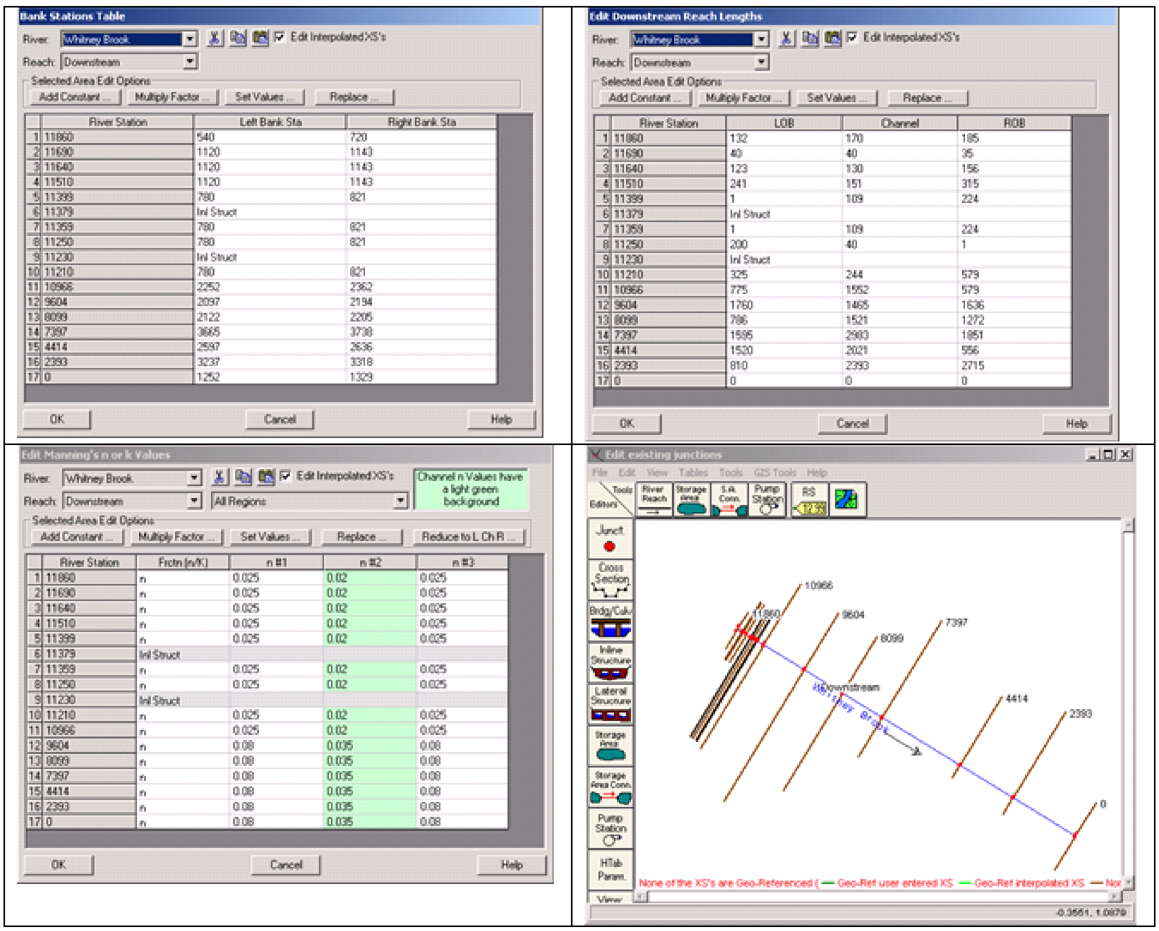

Figure 24 – HEC-RAS Geometry Input.

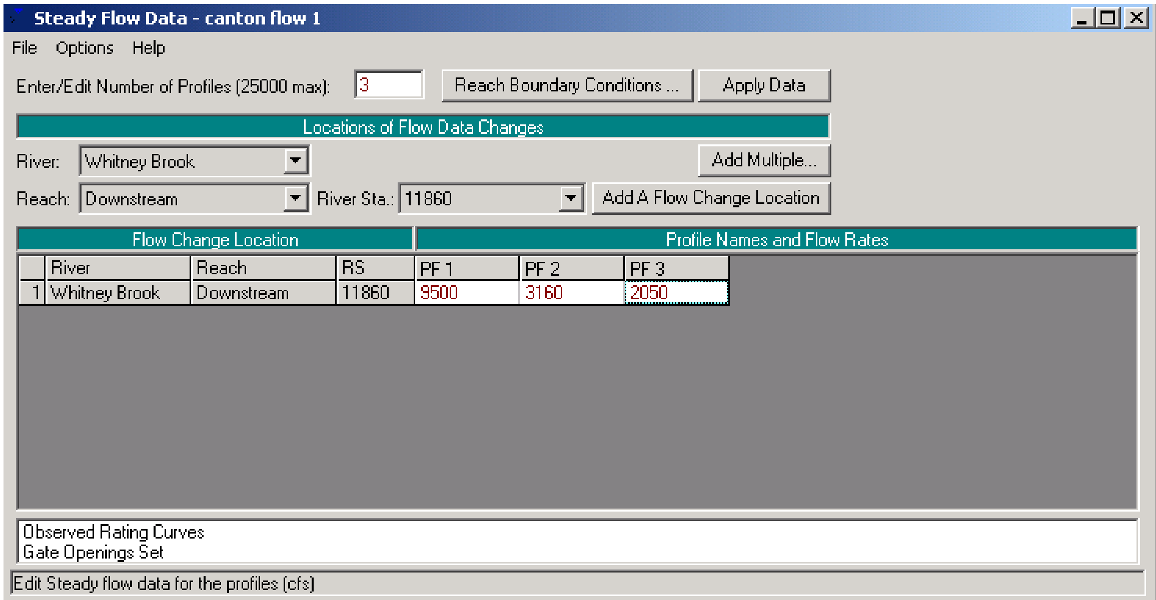

Figure 25 – HEC-RAS Flow Data Input.

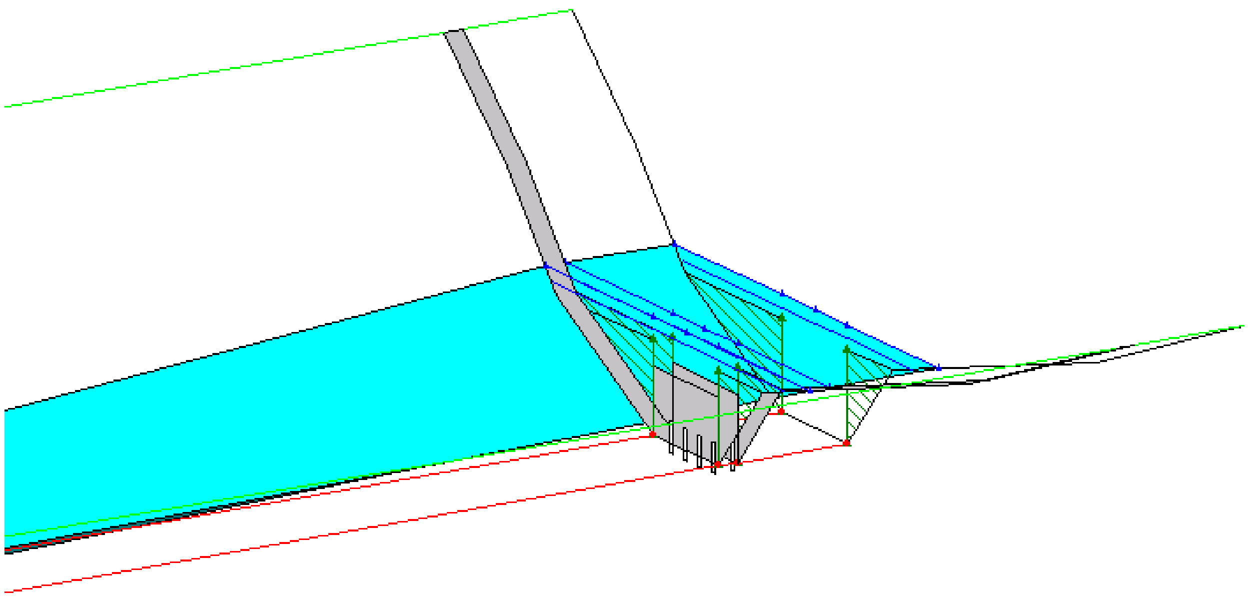

Figure 26 –Example HEC-RAS Profile of Lake Anasagunticook Dam

3.2.4 Check of Hydraulic Model

Lastly, a check was made of the output from HEC-RAS to determine if the results were accurate. Information from the HEC-RAS output sheet where collected and used in Manning's equation (equation 10). A flow was calculated and compared to the flow obtained from HEC-RAS.

Open channel flow is based upon analyzing the characteristics of water flow such as the flow rate, the depth and the velocity. The relationships among these different characteristics at different cross sections of the channel are analyzed using basic flow concepts such as Manning's equation and the Froude number. Manning's equation relates the slope, hydraulic radius and friction of the channel to determine the velocity of the water flowing through the channel. The Froude number compares the velocity of the river flow in a cross-section with the critical velocity for the reach. When the Froude number is less than one the water flow has no opportunity to accelerate past the critical velocity of the channel and will be in a slow deep state also known as subcritical flow. When the Froude number is greater than one, the flow in the channel has been able to accelerate more than the water downstream of it and will create shallow turbulent water also known as supercritical flow. Equation 5 relates flow rate (Q), velocity (V) in and area (A).

$$Q = VA \tag{5}$$

In this situation, the flow rate is a constant and the area is defined by the location of the channel. This means that the velocity of the water will be dependent on the area of the channel at any time. Manning's equation (6) is used to find the normal depth under uniform flow conditions and relates the flow rate, the cross sectional area and the channel slope (S_o).

$$Q = \frac{1.49}{n} R^{2/3} S_0^{1/2} \tag{6}$$

In equation 6, n is Manning's roughness coefficient, which is determined by experimental factors. Any hydraulics textbook has standard charts of Manning's coefficients for various materials. R is the hydraulic radius, defined as the ratio of the wetted area to the wetted perimeter. R is important because it considers the water depth and channel base width. This will yield the normal depth of the water based on the slope, area and flow. The normal depth is important because it can be compared to the critical depth. This is the first step in determining the flow as sub or supercritical. Critical depth is the depth of water in the channel at which the flow will transition from supercritical (shallow, fast flow) to sub-critical (deep, slow flow). The determination of critical depth relates the unit flow rate (q) to gravity (g) as seen in equation 7.

$$y_c = \sqrt[3]{\frac{q^2}{g}} \tag{7}$$

The last equation in determining the flow solves for the Froude number (equation 8). The Froude number is a dimensionless value that describes the ratio of inertial and gravitational forces as described above. It is given by the following formula.

$$N_F = \frac{V}{\sqrt{gD}} \tag{8}$$

The numerator of the fraction is the mean flow velocity and the denominator is the speed of a small gravity surface wave traveling over the water surface. D is the depth of the water. When the Froude number is less than unity, $V < \sqrt{gD}$ then the flow velocity is smaller than the speed of a disturbance wave traveling on the water surface meaning a sub-critical state. When $V > \sqrt{gD}$ it indicates a supercritical state.

The data from the HEC-RAS model was checked with the equations presented in section 3.2.4 and found to be reasonable. The size of the required spillway was then determined and sized as a one-foot unit section.

3.3 Structural Analysis

The purpose of structural analysis is to determine the required geometry and size of the dam while ensuring factors of safety against major instabilities. The structural design of the dam is based on the results of the hydrologic and hydraulic analysis. The determination of structural soundness will depend on the analysis of all loadings, the material properties and the geometric configuration of the dam. An overview of the methodology for each type of dam investigated is shown in Figure 27. The major steps involved in the structural design are:

- Selection of initial design
- Calculation of external forces
- Analysis of external Stability
- Analysis of internal stability

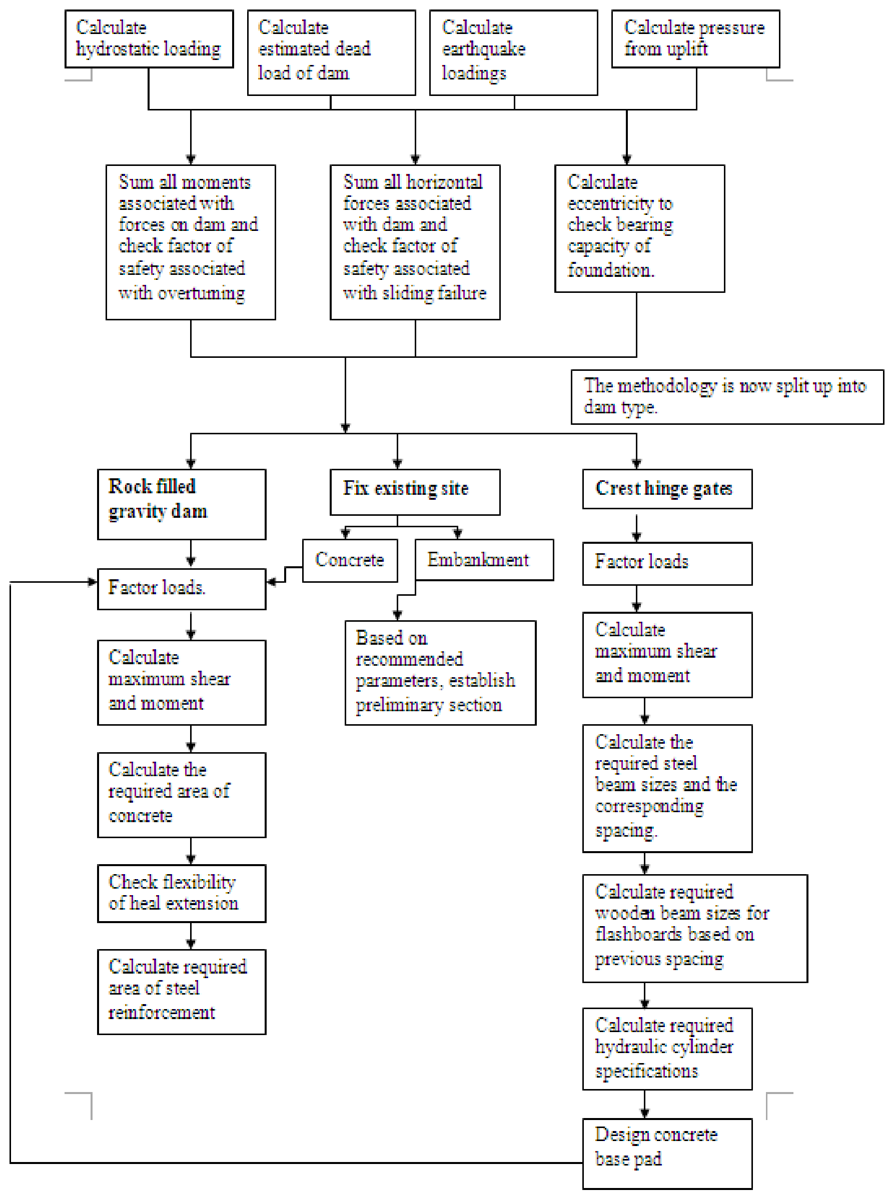

Figure 27 – Structural Analysis Flowchart

The initial design calculations check the size and shape of the assumed dam cross section. Based on the design flood, a preliminary size of the dam is chosen. First, the preliminary dam shape is analyzed for external forces. Then the preliminary design dimensions are analyzed for sliding and overturning. Depending on the results of analysis, the designer may change the size or shape of the dam and reevaluate a more stable dam. The internal forces are then calculated. Internal forces are caused when an external load is transferred through the dam. The external forces and equilibrium are calculated with the same methodology for each type of dam. The internal forces are project specific for each type of dam because of varying dam configurations and materials.

3.3.1External Forces

The external forces are forces that act on the exterior of the dam. These forces include hydrostatic forces, dead loads, earthquake forces, uplift pressure, and ice forces.

3.3.1.1 Hydrostatic Forces

First, the hydrostatic and dead weight loading are calculated. These are the principle external forces acting on the dam. Usually, the hydrostatic pressure is modeled as a triangular (See equation 9) distribution of stresses on the dam.

$$P_h = \frac{1}{2}\gamma h^2 \text{ (Triangular)} \qquad (9)$$

The dead load of the dam causes a downward force on the foundation. Gravity dams rely mostly on their weight for stability. The dead load calculations are based on the shape of the structure and the material’s unit weight. Simplifications of the calculations for triangular (see equation 10) and square (see equation 11) sections are below.

$$W_d = \frac{1}{2}LH\gamma \text{ (Triangular)} \qquad (10)$$

$$W_d = LH\gamma \text{ (Square)} \qquad (11)$$

3.3.1.2 Earthquake Loading

When an earthquake occurs, additional forces are placed on a dam. Recommendations for seismic design and evaluation are provided in the USACE document EM 1110-2-2200. The document includes guidance on using the seismic coefficient method, which provides a simple and direct approach for stability evaluations. Depending on the scenario, different limit conditions have been established for finding the sliding factor of safety and the location of the resultant. (See Figure 28)

$$P_e = C\lambda wh \quad \text{(Pressure due to earthquake)} \tag{12}$$

$$C = \frac{C_m}{2}\left[\frac{y}{h}\left(2-\frac{y}{h}\right)+\sqrt{\frac{y}{h}\left(2-\frac{y}{h}\right)}\right] \tag{13}$$

Where: Y= height from water surface elevation to area of calculation

H=water depth

Cm=Pressure coefficient = (0.7/0.238) from chart on page 165

W=unit weight of water=62.4 lb/cubic foot

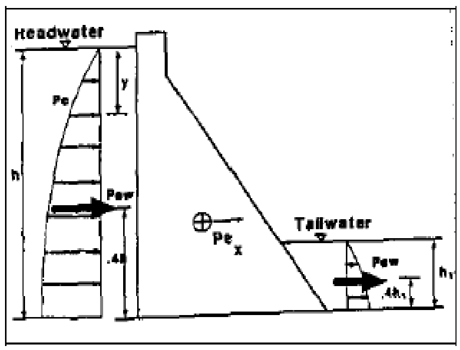

Figure 28 – Distribution of Earthquake Forces (USACE 1110-2-2200, 1995)

3.3.1.3 Uplift Pressures

Uplift pressure resulting from the headwater and tail water exists through the cross section of the dam. They occur at the interface between the dam and the foundation and within the foundation below the base. This pressure is present within the cracks, pores, joints and seams in the concrete and foundation materials. Uplift pressure is an active force that must be included in the stability and stress analysis to ensure structural adequacy. Generally, uplift pressure will be considered as acting over 100 percent of the base. A hydraulic gradient between the upper and lower pool is as seen in Figure 29. The formula for finding the resultant uplift pressure is shown in equation 14.

$$Pu = .\frac{1}{2} l * \Psi * h \tag{14}$$

Where: l=length of cross section

Ψ=Unit weight of Water

h=Hydraulic head

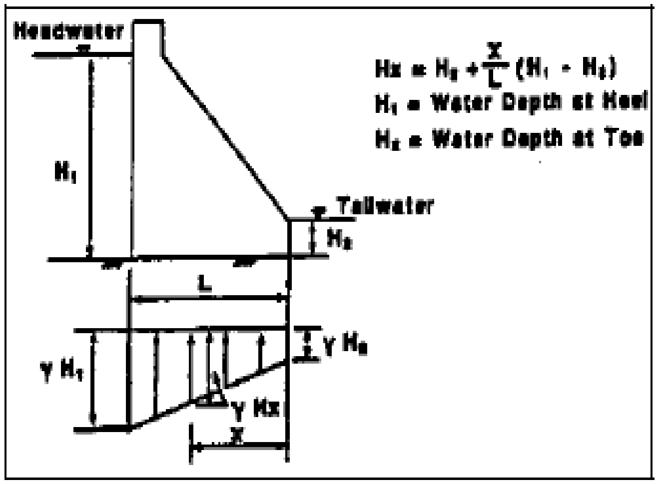

Figure 29 – Distribution of Uplift Forces (USACE 1110-2-2200, 1995)

3.3.2 External Stability

It was previously discussed that for the dam to remain stable it must maintain its equilibrium of forces and moments in all planes. For the dam to resist all overturning forces, equation 15 must be satisfied.

$$FS_{Overturning} = \frac{\sum M_{\mathrm{Re}\,sisting}}{\sum M_{Overturning}} \geq 1.5 \sim 2.0 \qquad (15)$$

Sliding failure is very similar to overturning failure except that instead of dealing with moment forces, it deals with horizontal-direction (shearing) forces. When equation 16 is satisfied, it ensures that the shear stresses applied to the base of the dam are not too large to be resisted.

$$FS_{Sliding} = \frac{\sum F_{\mathrm{Re}\,sisting}}{\sum F_{Sliding}} \geq 1.5 \sim 2.0 \qquad (16)$$

The eccentricity of the dam must be determined to be within mid half of the dam base (see equation 17) and the factor of safety checked (see equation 19) in order to calculate the bearing failure of the foundation.

$$e = \frac{B}{2} - x \qquad (17)$$

Where ……………… $$\frac{\sum M}{b} = H_y(L) + (H_y)Lx - \sum M_R \qquad (18)$$

is used to find x

x=distance from the turning point to the resultant force action location

Check Factor of Safety $$e < \frac{B}{4} \qquad (19)$$

3.3.3 Foundation Bearing Capacity

Compressive strength and shear strength are important factors in dam design. Allowable bearing capacity for a structure is often selected as a fraction of the average foundation rock compressive strength to account for inherent planes of weakness along natural joints and fractures. A more accurate method of determining bearing capacity is detailed below where the bearing capacity is dependent on the eccentricity, footing extension, load per foot and the weight of the footing. The equations to check the bearing capacity of a concrete tee wall are found in equations 20, 21 and 22.

$$B' = B - 2e \qquad (20)$$

Where e=eccentricity

B'= footing extension (effective footing width)

$$q_{eq} = \frac{P + W_f}{B'L'} \qquad (21)$$

Where P=load per foot

W_f =Weight of footing

L'= Unit Length

Check Factor of Safety: $$q_{eq} \leq \frac{q_A}{2} \qquad (22)$$

Where q_A = bearing capacity

3.3.4 New Rock Filled Gravity Dam Internal Stability

After the external analysis was completed and the dam as a whole was found to be stable, the material design begins. The first design alternative is a rock filled, concrete walled, gravity dam. The concrete was designed using the American Concrete Institute specifications. The structure was analyzed as a retaining wall, with the rock acting only as dead weight for the structure.

The first step is to find the loading factors (see equations 23, 24). The larger of the two load combinations will be used.

$$P_u = 1.4(D + F) \quad (23)$$

$$P_u = 1.2(D + F) + 1.6(L) \quad (24)$$

Where D= Dead load

L=Live load

F=Fluid load

With the loading known, the shear and moment must be calculated. Both the moment (equation 25) and shear (equation 26) calculations are based on equilibrium. The moment will be based on the sum of moments being equal to zero while the shear will be the sum of horizontal forces equaling zero.

Moment:

$$M_{uMAX} : \sum M_o = 0 = M_{uMAX} + P_u(L) \quad (25)$$

Where P_u =Factored force

L=Moment arm

M_{uMAX} = Max factored moment

Shear:

$$\sum F_x = 0 = V_{nMAX} - P_u \quad (26)$$

Where V_{nMAX} =Max shear

P_u =Factored force

The stem thickness will be based on the unit width, concrete strength, and wall depth. Additionally, the concrete needs to be reinforced with steel. The diameter and quantity of rebar required will also be calculated. This process is laid out in equations 27 though 34.

$$\frac{V_n}{b} = 2b_w d\sqrt{f'_c} \quad (27)$$

Where: f'_c = Concrete compression strength

b_w =Unit width

d =Effective wall depth

Vn=Shear force

$$T \geq d + d_b + \text{cover} \tag{28}$$

Where: T=Stem thickness

d_b =rebar diameter

cover =3 inches

The shear analysis of the heal extension is calculated as follows:

$$\left(\frac{W_{footing} + W_{water} + W_{rockfill}}{b}\right)1.4 = \frac{V_u}{b} \tag{29}$$

Where W = weight

B = 1 ft. unit section

V_u = factored shear

$$\frac{V_u}{b} = 2b_w d\sqrt{f'c} \tag{30}$$

$$\frac{\phi V_n}{b} = \left(\frac{V_u}{b}\right) \tag{31}$$

Where b_w = stem size

d = footing depth

f'_c = compressive strength of concrete

ϕ = load factor = .85

To ensure stability, check that $V_u \leq \phi V_n$

$$\frac{V_u}{b} \leq \frac{\phi V_n}{b} \tag{32}$$

The heal extension flexural analysis is computed as:

$$\frac{M_u}{b} = 1.4(DL + LL) \tag{33}$$

Where M_u = factored moment

DL=dead load

LL=live load

The final step in concrete design is to calculate the required area of rebar. This is calculated as:

$$\frac{A_s}{b} = \left(\frac{f'_c\, b}{1.176 f_y}\right)\left(d - \frac{\sqrt{2.353 M_u / b}}{\phi f'_c\, b}\right) \qquad (34)$$

When the value of A_s/b is computed, it is compared to the minimum allowable value which is equal to $0.0018A_g$. A_g is the gross area of the footing. Standard rebar size versus gross area should be checked to ensure proper rebar selection. Rebar spacing should also be considered during this stage.

3.4.2 Existing Structure Internal Analysis

The existing dam consists of a concrete, gravity, overflow spillway and two earthen embankments leading up to the spillway. The existing dam cannot pass the ½ PMF therefore, an emergency spillway has been proposed to increase discharge capacity. (See Figure 30)

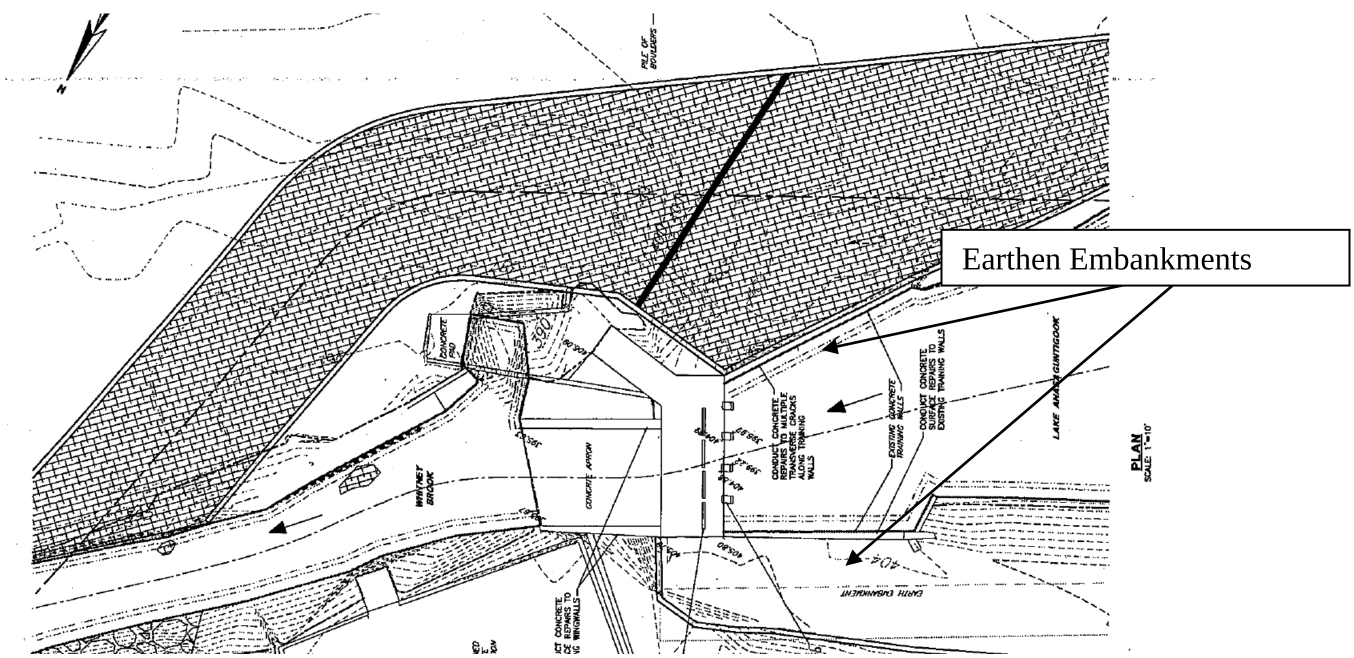

Figure 30 – Overview of Existing Structure with Proposed Emergency Spillway (Wright-Pierce, 2007)

The concrete spillway was designed and checked using the same equations and methodology as the concrete gravity dam in section 3.4.1. Earthen embankments will be discussed in detail below. The emergency spillway is designed for erosion similarly to an embankment design and is lined with riprap. The size of the spillway was determined through the hydraulic modeling discussed in section 3.2.

The design criteria for embankment dams include:

- Safety against overtopping during flooding.
- Slope stability.
- Reduction of seepage.
- Reduction of slope erosion.

Safety against overtopping is a design parameter dependent on the hydraulic analysis. If the hydraulic analysis is completed accurately, the dam height and width should be such that during the design flood, the water will not overtop the embankments.

Slope stability is finding the equilibrium of an embankment under loading from internal and external forces. Slope stability embankment design begins with the determination of pore water pressure. Pore water pressure is the pressure of groundwater held within gaps in soil (pores) and is calculated as the hydrostatic pressure (equation 35).

$$u = \rho g h \qquad (35)$$

Where ρ = the liquid density

g = gravity

h = height of water

For slope stability analysis, the ordinary method of slices will be used (see Figure 31). In this method, the normal force on the base of the slice is calculated by summing forces in a direction perpendicular to the bottom of the slice. Once the normal force is calculated, moments are summed about the center of the circle to compute the factor of safety (see equation 36). The factor of safety ensures adequate performance of slopes throughout their design lives. Two of the most important considerations that determine appropriate magnitudes for the factor of safety are uncertainties in the conditions being analyzed, which include shear strengths, and the consequences associated with in Table 5.

Table 5 – Minimum Required Factor of Safety (USACE EM1110-2-1902)

Required Factors of Safety		
Analysis condition	**Min. F. S.**	**Slope**
End of Construction	1.3	upstream/downstream
Long-term	1.5	downstream
Max surcharge pool	1.4	downstream
Rapid drawdown	1.3	upstream

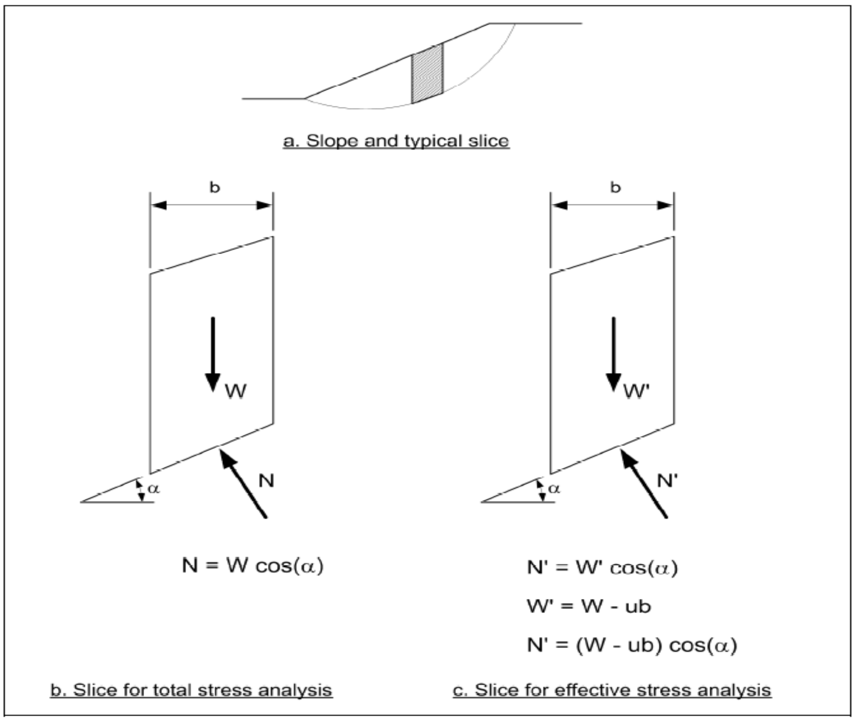

Figure 31 – Typical Slice and Forces for Ordinary Method of Slices (USACE EM1110-2-1902)

The factor of safety when using the Ordinary Method of Slices

$$F = \frac{\sum\left[c'\Delta\ell + \left(W\cos\alpha - u\Delta\ell\cos^2\alpha\right)\tan\phi'\right]}{\sum W\sin\alpha} \qquad (36)$$

Where $c'\ and\phi'$ = shear strength parameters for the center of the base of the slice

W= weight of the slice

α = inclination of the bottom of the slice

u = pore water pressure

$\Delta\ell$ = length at the bottom of the slice

In the case where water loads act on top of the slice (See Figure 32), the expression for the factor of safety (see equation 37) must be modified to the following:

$$F = \frac{\sum c'\Delta\ell + \left[W\cos\alpha + P\cos(\alpha-\beta) - u\Delta\ell\cos^2\alpha\right]\tan\phi'}{\sum W\sin\alpha - \frac{\sum M_p}{R}} \quad (37)$$

Where P = resultant water force acting perpendicular to the top of the slice

β = inclination of the top of the slice

M_p = moment about the center of the circle produced by the water force acting on the top of the slice

R = radius of the circle

(USACE, EM 1110-2-2300)

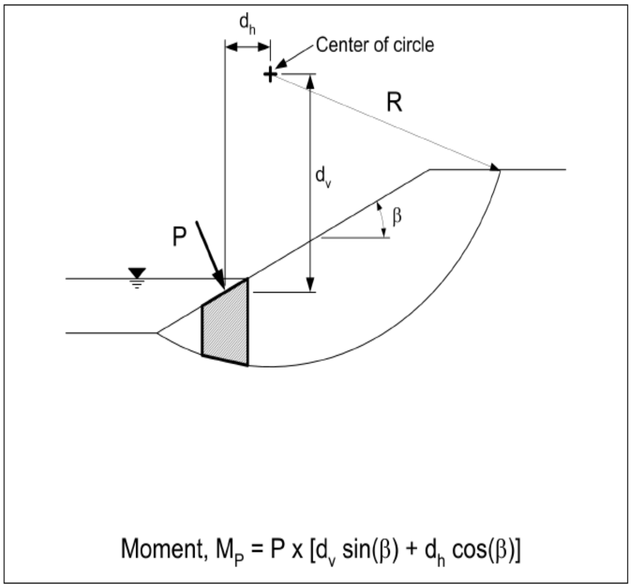

Figure 32 – Slice for Ordinary Method of Slices with External Loads (USACE EM1110-2-1902)

Table 6 gives recommended slopes for small, heterogeneous earthfill dams on stable foundations similar to the Lake Anasagunticook Dam. Table 6 will give a good slope value to start the slope stability analysis.

Table 6 – Recommended slopes for small, zoned earthfill dams on stable foundations (Dept. or Interior, Design of Small Dams)

Case	Detention/ Storage?	Type	Rapid drawdown?	Shell soil classification	Core soil classification	Upstream slope	Downstream slope
A	either	min. core	not critical	Not critical, Rock fill/gravel	Not critical, GC, FM, SC	2 to 1	2 to 1
B	either	max. core	no	Not critical, Rock fill/gravel	GC, GM	2 to 1	2 to 1
					SC, SM	2.25 to 1	1.25 to 1
					CL, ML	2.5 to 1	2.5 to 1
					CH, MH	3 to 1	3 to 1
C	storage	max. core	yes	Not critical, Rock fill/gravel	GC, GM	2.5 to 1	2 to 1
					SC, SM	2.5 to 1	2.25 to 1
					CL, ML	3 to 1	2.5 to 1
					CH, MH	3.5 to 1	3 to 1

Slope protection from erosion can be achieved through the placement of riprap, which is composed of angular granite rock of high specific gravity and excellent quality. Protection for the downstream slope should consist of well-maintained low vegetative cover. Table 7 gives the thickness and gradation limits of riprap on a 3:1 slope. For 2:1 slopes, the nominal thickness required (except for the 36-inch thickness) should be increased by 6 inches and the corresponding gradation used. For a slope between 3:1 and 2:1, the nominal thickness of the riprap should be interpolated between the known values. (Dept. Interior, Design of Small Dams)

Table 7 – Thickness and Gradation Limits of Riprap on 3:1 Slopes (Dept of Interior, Design of Small Dams)

Reservoir fetch, miles	Nominal thickness, inches	Gradation, percentage of stones of various, weights (pounds)			
		Maximum size	At least 25% greater than	45-75% from-to	Not more than 25% less than-
1 and less	18	1000	300	10-300	10
2.5	24	1500	600	30-600	30
5	30	2500	1000	50-1000	50
10	36	5000	2000	100-2000	100

3.4.3 New Crest Gates

With the external loads known, the internal structural design of a crest gate dam can be started. First, the main supporting members of the crest gate are sized. Then the gate steel is designed using the AISC Steel Construction Manual. The first step is to find the loading factors (see equations 38, 39, 40). The larger of the three load combinations will be used. The load combinations come from the USACE EM 1110-2-2702.

$$P_u = 1.4(H) + 1.2(D) + 1.6(C) \tag{38}$$

$$P_u = 1.2(D) + 1.4(H) + 1.6(C + I + M) \tag{39}$$

$$P_u = 1.2(H + D) + 1.4(H) + 1.6(C + M) + 1.0(E) \tag{40}$$

Where: D= Dead load

H=Hydrostatic Load

C=Ice Dead Load

E=Earthquake Force

M=Mud Dead Load

I=Impact Loading

Pu=Factored Loading

With the loading known, the shear and moment must be calculated for the main vertical members. Both the moment and shear calculations are based on statics equilibrium. The moment will be based on the sum of moments (equation 41) being equal to zero while the shear (equation 42) will be the sum of horizontal forces equaling zero.

Moment:

$$M_{uMAX} : \sum M_o = 0 = M_{uMAX} + P_u(L) \qquad (41)$$

Where P_u =Factored force

L=Moment arm

M_{uMAX} = Max factored moment

Shear:

$$\sum F_x = 0 = V_{nMAX} - P_u \qquad (42)$$

Where V_{nMAX} =Max shear/ft

P_u =Factored force/ft

3.4.3.1 Steel Design

The steel beam sizing is based on the maximum factored moment and shear values from the external force analysis on the structure. The AISC Manual of Steel Construction gives the allowable bending stress and shear stress of a steel member. The required section modulus for a beam of sufficient size to resist the forces from the dam was determined. A suitable beam was then chosen from the AISC manual. The shear capacity of the beam was checked against the allowable shear capacity and determined if it is acceptable. Equations 43 though 49 show the progression of the steel member design.

Moment:

$$F_b = .66F_y \tag{43}$$

$$S = \frac{I}{C} \tag{44}$$

$$S_{req} = \frac{M_u}{F_b} \tag{45}$$

$$S_{req} \leq S \tag{46}$$

Shear:

$$F_v = 0.40F_y \tag{47}$$

$$A_{req} = \frac{V_u}{F_v} \tag{48}$$

$$A_{req} \leq A \tag{49}$$

Where:

S = Section Modulus (From AISC Manual)

S_{req}= Required Section Modulus

I= Moment of Inertia

C =Distance from NA to Extreme Fiber

M_{u}= Factored Moment

F_y =Yield strength of Steel

F_b=Allowable Bending Stress

F_v = Allowable Shear Stress

A_{req} =Required Area of Steel

A = Actual Area of Steel (AISC Manual)

V_u = Factored Shear

3.4.3.2 Wood Design

Once the main supporting beams were selected, the wooden beam sizes for the interconnecting wooden webs had to be selected. The American Wood Council's Manual for Wood Construction was used for the design. The same sized beam was uniformly used throughout the project. First, a wood type was chosen. The bending design value (equations 50-53) and shear design value (equations 54-57) were found in the AWC Wood Construction Manual. The appropriate ASD strength factors were applied to the bending design value and shear design value to get factored values. The required section modulus and required area of the wood beam were found. A beam with a section modulus and area larger than the required design values was found in the AWC Wood Construction Manual.

Bending:

$$M' \geq M \tag{50}$$

$$M' = F_b' S \tag{51}$$

$$F_b' = F_b C_m C_d C_l C_t C_{fu} C_i C_f C_r \tag{52}$$

$$S_{req} = \frac{M}{F_b{}'} \tag{53}$$

Shear:

$$V' \geq V \tag{54}$$

$$V' = \frac{2}{3} F_v{}' A \tag{55}$$

$$F'_v = F_v C_d C_m C_t C_i \tag{56}$$

$$A_{req} = {V} \Big/ {\tfrac{2}{3} F'_v} \tag{57}$$

Where:

M' = Adjusted Moment Capacity

M = Bending Moment

F_b' = Adjusted Bending Design Value

S = Section Modulus

S_{req} =Required Section Modulus

F_b= Bending Design Value

V' = Adjusted Shear Capacity

V = Shear Force

F_v =Shear Design Value

F'_v = Adjusted Shear Design Value

A =Area

A_{req} =Required Area

C_d= Load Duration Factor

C_m= Wet Service Factor

C_t =Temperature Factor

C_l = Beam Stability Factor

C_f = Size Factor

C_{fu} = Flat Use Factor

C_i = Incising Factor

C_r = Repetitive Member Factor

3.4.3.2 Hydraulic Cylinder Design

The hydraulic cylinders for the gate actuation were chosen out of the Prince Hydraulics catalogue. Standard, off the shelf cylinders were selected based on allowable operating pressure, bore size, and bore stroke. The maximum force per cylinder was calculated (equation 58). Then the required spacing between hydraulic cylinders was calculated (equation 59).

$$P_{max} = H_p \frac{D}{2}^2 \pi \tag{58}$$

$$D_{max} = \frac{P_u/_b}{P_{max}} \tag{59}$$

Where:

P_{max} =Maximum Allowable Load per Cylinder

H_p =Maximum Hydraulic Cylinder Pressure

P_u =Maximum External Loading

D_{max} =Maximum Cylinder Spacing

3.4.3.2 Gate Sill Design

The last part of the crest gate design, is the design of the concrete sill and substructure for the gate superstructure. The design closely follows the design of the continuous footing for the rock filled gravity dam. The methodology described in the second half of section 3.4.1 can be followed to produce an adequate concrete slab for the crest gate.

3.5 Dam Locations

The location of the proposed new dams maters greatly. If the natural topography of the river channel upstream of the existing structure is used, the long embankments parallel with the stream can be reduced in size or eliminated. Currently the existing structure is in a relatively broad flood plain with almost no natural containment from the riverbanks. This led the designers of the existing dam to add an extensive embankment to the left stream bank upstream of the dam. An upstream location will also allow for an easier hydraulic passing of the design flood flow. Instead of using embankments to contain the design flood, the natural channel can be used to reduce construction costs. Four potential upstream locations of the dam were identified for further analysis:

1. 175 feet upstream of the current dam at an old bridge crossing of the stream,
2. 280 feet upstream of the current dam at a natural constriction point in the channel,
3. 750 feet upstream of the current dam at a natural bluff in the right and left embankments, and
4. 1500 feet upstream of the current dam were another natural bluff occurs.

The USGS map shown in Figure 33 shows that the 420 foot contour line appears just upstream of the old bridge foundation. This contour can be used to contain the water instead of using an artificial embankment. The current ground elevation at the dam site for the flood plain is between 398 and 402 mean sea level. By using the 420 foot contour, a 20 foot high natural embankment can be exploited at almost no cost.

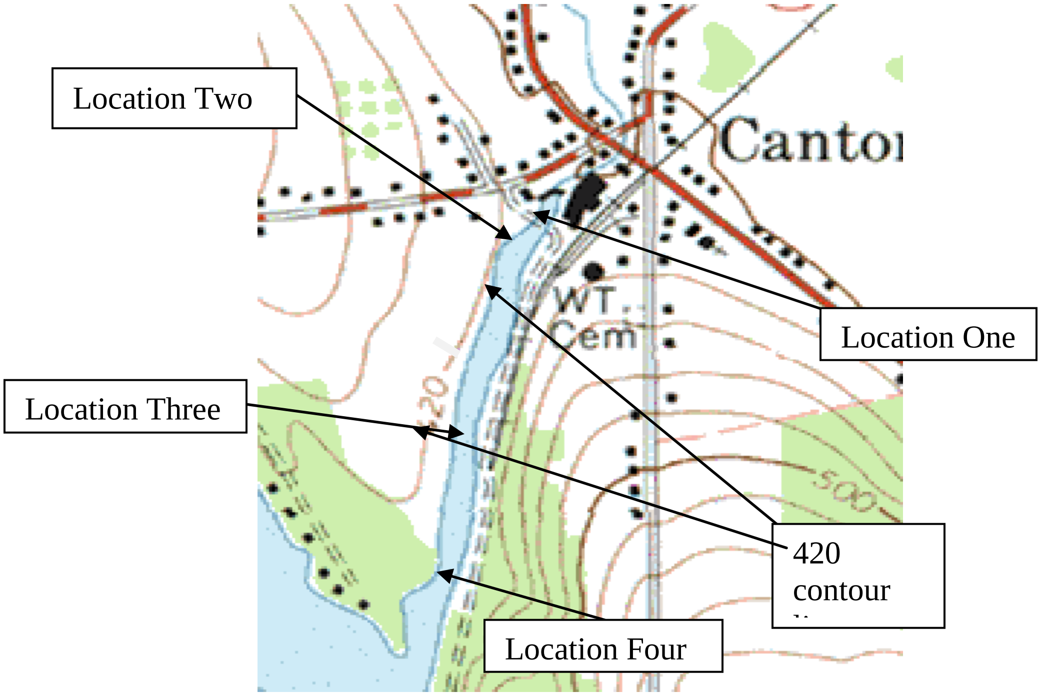

Figure 33 – Proposed New Dam locations.

3.6 Cost Analysis

The cost estimate for this project was complex. An estimate had to be made for each design option, which meant a thorough investigation of costs for each option. General cost categorizations are listed in Table 8.

Table 8 – Overview of Major Cost Divisions and their Corresponding Division Numbers (Dagostino, 2003)

Division Name	Division Number
General Requirements	1000
Site work	2000
Concrete	3000
Metals	5000
Equipment	11000
Special Construction	13000

There are many other categories however, they are not applicable to this project and have been omitted. RS Means publishes annual guides by division number that gives a range of unit costs for a variety construction costs as well as a number of adjustments to compensate for varying systems. (Dagostino, 2003)

Chapter 4: Results

The most important aspect of the dam design is safety. The USACE considers dams to have low probability of failure, but high impact of failure, due to the large amount of released energy upon failure. In Canton, purpose of the dam is to impound water for recreational purposes, therefore, to serve its purpose the dam must keep the water height at a constant water level of 402 msl. Although the dam must keep the lake level at approximately 402 msl in normal operating conditions, the dam must also be able to safely pass the design flood over the spillway without overtopping the embankments. In most projects, where there is more than one design option, the final decision will be determined by constructability and cost

4.1 Hydrologic Results

A hydrologic analysis of the Anasagunticook Dam drainage area was performed to determine the possible maximum flood that can be produced by the drainage area. It is necessary to know the possible maximum flood, in order to design a spillway with an adequate discharge capacity. The possible maximum flood is dependent on many specific characteristics of a drainage area. These specific characteristics were determined and used in two computer models. HMR-52 was used to output the possible maximum precipitation from the possible maximum storm. Then the possible maximum precipitation was inputted to HEC-HMS to determine the possible maximum flood that is produced from the possible maximum rainfall. A map of the drainage area is seen in Figure 34.

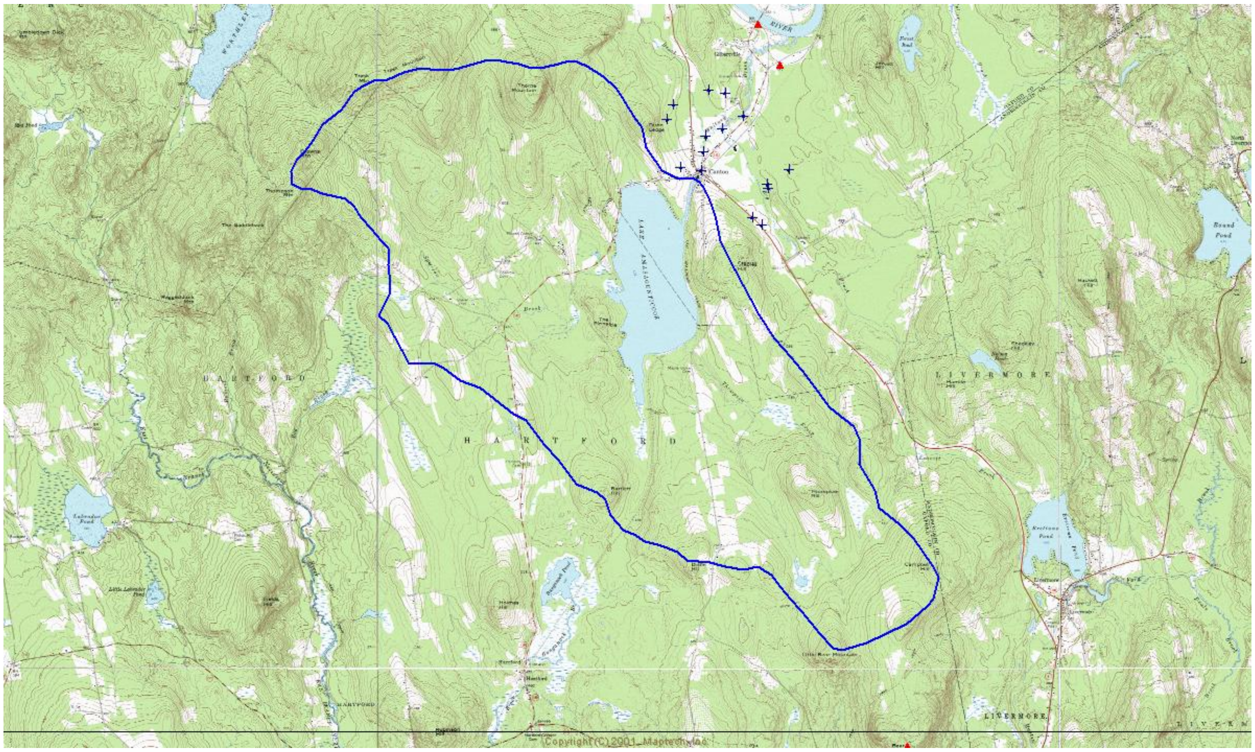

Figure 34 – Lake Anasagunticook Dam Drainage Area (Topo Scout)

MEMA adopted the United States Army Corps of Engineers' (USACE) standards for dam safety. These include the design specifications for inflow design floods, required spillway capacities and dam breach outflows. A ½ PMF event was initially chosen because it is the USACE's most conservative spillway design standard that is applicable to the Anasagunticook Lake Dam.

The probable maximum precipitation was found using the procedure outlined in section 3.1.2 of this report. The output of HMR-52 shown in Table 9 is the incremental possible maximum rainfall. This is shown in tabulated form for the possible maximum storm. Note the peak rainfall happens in day two with a gradual increase leading up to the maximum hourly rainfall and then a gradual hourly decrease.

Table 9 – HMR-52 Output Table

PROBABLE MAXIMUM STORM FOR TOTAL

DAY 1

	TIME	PRECIPITATION		TIME	PRECIPITATION		TIME	PRECIPITATION		TIME	PRECIPITATION	
		INCR	TOTAL		INCR	TOTAL		INCR	TOTAL		INCR	TOTAL
	0100	.07	.07	0700	.09	.52	1300	.11	1.07	1900	.14	1.74
	0200	.07	.15	0800	.09	.61	1400	.11	1.17	2000	.14	1.88
	0300	.07	.22	0900	.09	.70	1500	.11	1.28	2100	.14	2.02
	0400	.07	.29	1000	.09	.78	1600	.11	1.39	2200	.14	2.16
	0500	.07	.36	1100	.09	.87	1700	.11	1.49	2300	.14	2.30
	0600	.07	.44	1200	.09	.96	1800	.11	1.60	2400	.14	2.44
6-HR TOTAL		.44			.52			.64			.84	

DAY 2

	TIME	PRECIPITATION		TIME	PRECIPITATION		TIME	PRECIPITATION		TIME	PRECIPITATION	
		INCR	TOTAL		INCR	TOTAL		INCR	TOTAL		INCR	TOTAL
	0100	.19	2.63	0700	.30	3.96	1300	.90	6.78	1900	.30	26.08
	0200	.19	2.82	0800	.29	4.25	1400	1.62	8.40	2000	.28	26.36
	0300	.20	3.02	0900	.31	4.57	1500	2.78	11.18	2100	.27	26.63
	0400	.21	3.22	1000	.36	4.93	1600	8.95	22.12	2200	.25	26.88
	0500	.21	3.44	1100	.43	5.35	1700	2.27	24.40	2300	.24	27.12
	0600	.22	3.66	1200	.52	5.87	1800	1.39	25.78	2400	.23	27.36
6-HR TOTAL		1.22			2.21			19.91			1.57	

DAY 3

	TIME	PRECIPITATION		TIME	PRECIPITATION		TIME	PRECIPITATION		TIME	PRECIPITATION	
		INCR	TOTAL		INCR	TOTAL		INCR	TOTAL		INCR	TOTAL
	0100	.17	27.52	0700	.12	28.47	1300	.10	29.18	1900	.08	29.74
	0200	.17	27.69	0800	.12	28.60	1400	.10	29.27	2000	.08	29.82
	0300	.17	27.85	0900	.12	28.72	1500	.10	29.37	2100	.08	29.90
	0400	.17	28.02	1000	.12	28.84	1600	.10	29.47	2200	.08	29.97
	0500	.17	28.19	1100	.12	28.96	1700	.10	29.56	2300	.08	30.05
	0600	.17	28.35	1200	.12	29.08	1800	.10	29.66	2400	.08	30.13
6-HR TOTAL		1.00			.73			.58			.48	

1

The HMR-52 data was then used in the HEC-HMS possible maximum flood calculation. The input data described in section three was put into the HEC-HMS model and a PMF output was calculated according to the procedure outlined in section 3.1.3.

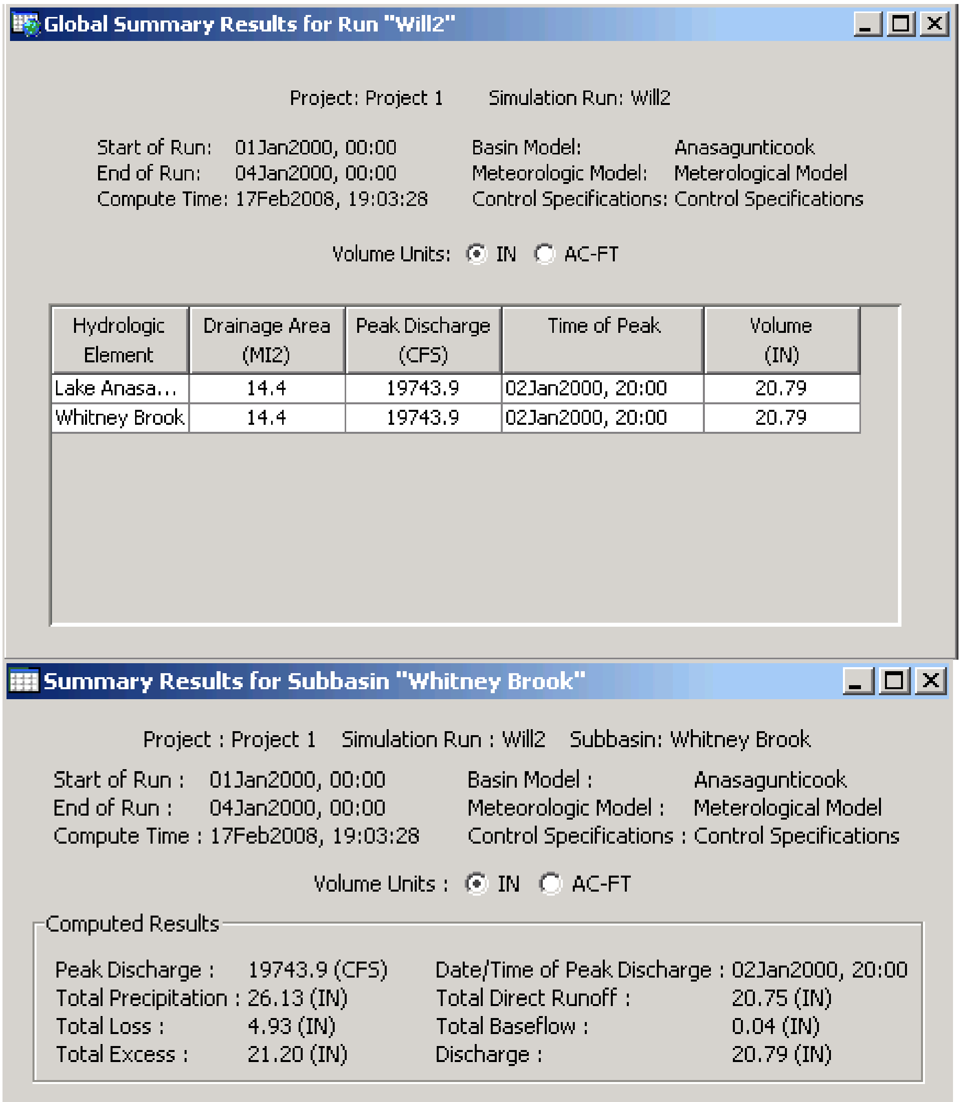

Figure 35 – HEC-RAS PMS Tabular Output.

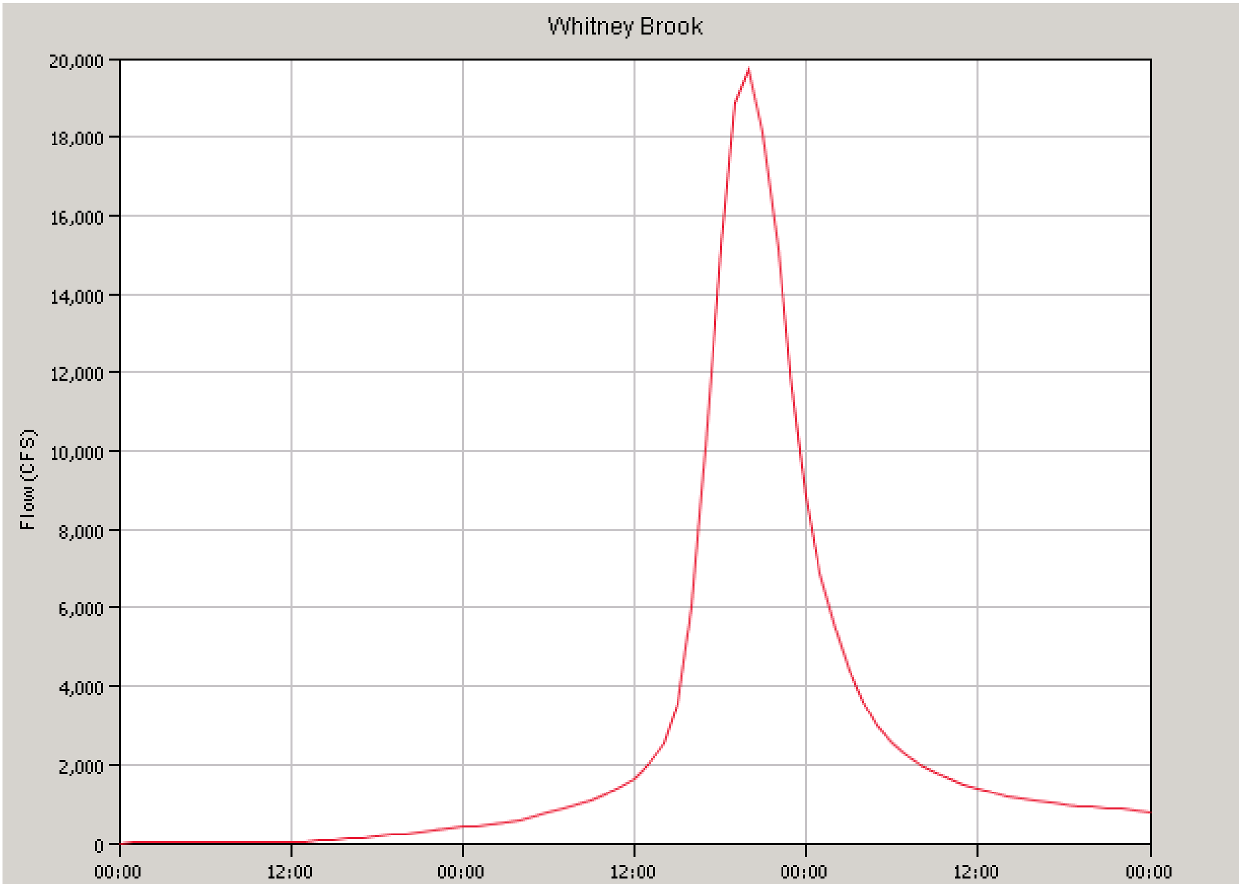

Figure 36 – HEC-HMS Hydrograph PMF Output.

The output from the HEC-HMS is shown in Table 9 and Figure 35. Figure 35 shows the tabulated global summary of results. Figure 36 is a graphical representation of the possible maximum flood. The peak flow rapidly rises after the peak rainfall hour and then rapidly diminishes as the hourly rainfall amounts decreases. The hydrograph slowly levels out during the last twelve hours of the storm. This is because the soil in the drainage area is saturated and any rainfall received is directly transposed into stream runoff. The peak stream flow of 19,500 cfs compares well to the Wright-Pierce estimate of 19,075 cfs and the USACE's 1979 estimate 0f 22,875 cfs for the PMF flow. Therefore, the value of the PMF for Lake Anasagunticook Dam is 19,500 cfs. Our approach used to obtain the PMF provided a basis to define a number of alternative design floods and values. The final values of all determined floods are summarized in Table 10.

Table 10 – Summary of Floods Determined from the PMF.

Flood	Value (cfs)
PMF	19,500
½ PMF	9,500
1/6 PMF	3,160

Normally, a flow of 1.2 PMF (9,500 cfs) would be used for the design. For this case, definition of the design flood also revised the consideration of hydraulic modeling and a clear picture of the nature of the potential flooding. This meant that the hydraulic analyses were performed in an iterative manner. For example, based on the hydrologic models and formulas, the ½ PMF was determined to be 9,500 cfs. Next, the 9,500 cfs flow condition was modeled in HEC_RAS and it became obvious that the dam could not be designed for such a large flood because of the natural constrictions in the valley of Whitney Brook. This meant that we had to re-visit the design flood for the analysis. Because of these constraints, the flow characteristics would exceed the natural capacity of the channel, even if the dam were not in place.

Due to the hydraulic characteristics of the existing dam site, the ½ PMF events could not be accommodated with an economically feasible spillway design. At the dam site, during the ½ PMF event the water level is 3.8 feet over the right embankment. Therefore, a 500 year return period flood was chosen for the Anasagunticook Lake Dam spillway design flood. The 500 year flood flow value was obtained from the Federal Emergency Management Agency's (FEMA) Flood Insurance Study (FIS) for Canton, Maine. Table 11 shows the FEMA FIS flood flows for Whitney Brook.

The 500 year return period was chosen because it fit the USACE spillway design flood specifications and provided a 0.005% (1/500 years) chance of a yearly return. Additionally, other New England States (e.g. Massachusetts and New Hampshire) use the 500 year return period for the design of a new spillway for intermediate hazard dams and the USACE's specifications are comparatively very large.

Table 11 – FEMA Flood Values

FLOODING SOURCE AND LOCATION	DRAINAGE AREA (sq. miles)	PEAK DISCHARGES (cfs)			
		10-YEAR	50-YEAR	100-YEAR	500-YEAR
ANDROSCOGGIN RIVER					
At downstream corporate limits	2,470	52,600	73,800	85,200	113,000
WHITNEY BROOK					
At confluence with Androscoggin River	23.7	1,060	1,700	2,030	2,980
Above confluence of Childs Brook	15.1	710	1,150	1,380	2,050

Finally, the USACE specification says that when a dam's spillway fails from hydraulic inadequacy, the dam may not cause loss of life or cause catastrophic flood damage. During the 1/2 PMF there is approximately three feet of incremental flooding already in the Town of Canton. The water from the Androscoggin River downstream of the dam area backs up Whitney Brook into Canton and consequently floods homes. It can be assumed that any person living in the flood plain will have already evacuated the town at the peak flood time and the flood zone will be clear of inhabitants. This also means that by the time the dam is at its maximum capacity and could possibly fail, the town will already be evacuated and water damage already incurred to the downstream property. If the dam were to fail during the ½ PMF event, the discharge from the dam is just incremental flooding on top of the Androscoggin flood and will not cause any "extra" damage as a result. This means that during the ½ PMF, the failure of the dam will not cause loss of life or property as any damage will already be done. With this in mind, it would not be the most practical or economical solution to design for the ½ PMF as the natural mountainous valley will have already filled upon dam failure. A more practical solution would be to design the dam for the 500 year flood, as mentioned in the previous paragraph. The value of the 500 year flood for Whitney brook is 2,050 cfs.

4.2 Hydraulic Results

For the given dam, the design spillway capacity was determined using the USACE's HEC-RAS as described in Chapter 3.3 of this report. The hydraulic analysis determinins if a dam can pass the design flood without failure or overtopping.

Three flood flows were used in the analyisis of the Anasagunticook Lake dam's spillway capacity. The ½ PMF, the 1/6 PMF, and the 500 year flood were analized. The

½ PMF was used as the most conservative standard offered for the design of the Ansagunticook Lake Dam, the 1/6 PMF was the largest flow that the channel above the dam could pass without overtopping the existing bank elevation of 404 msl, and the 500 year flood was choosen accourding to the reasoning in section 4.1. The existing spillway was analized, a gravity dam with a fixed spillway hieght was analized, and a collapsible crest gate type dam was analized for the different flood flows. The channel geometry for the three models was keept the same with different dam geometrys being subsituted into the model.

The HEC-RAS output for the 500 year flood for the collapsible dam is shown in Table 12 and Figure 37. The 500 year flood was choosen as the design flood due to design restraints discussed above in section 4.1. The HEC-RAS results for the other senarios can be found in Appendix C of this report.

Table 12 – HEC-RAS Tabular output of Lake Anasagunticook Dam HEC-RAS

Reach	River Sta	Profile	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chl
			(cfs)	(ft)	(ft)	(ft)	(ft)	(ft/ft)	(ft/s)	(sq ft)	(ft)	
Downstream	11860	PF 1	9500.00	396.00	408.75		409.16	0.000239	5.23	1941.40	276.75	0.29
Downstream	11860	PF 2	3160.00	396.00	405.58		405.69	0.000098	2.63	1206.11	187.78	0.18
Downstream	11860	PF 3	2050.00	396.00	404.17		404.24	0.000078	2.13	963.30	162.46	0.15
Downstream	11690	PF 1	9500.00	395.00	407.84	407.84	409.01	0.001534	12.88	1738.10	671.30	0.64
Downstream	11690	PF 2	3160.00	395.00	403.81	403.81	405.49	0.002160	11.82	392.06	125.09	0.71
Downstream	11690	PF 3	2050.00	395.00	402.30	402.30	404.04	0.002490	11.16	234.04	84.72	0.74
Downstream	11640	PF 1	9500.00	388.00	403.36		404.52	0.001349	12.07	1573.51	460.68	0.55
Downstream	11640	PF 2	3160.00	388.00	398.42	398.42	400.53	0.002362	12.24	330.22	104.22	0.68
Downstream	11640	PF 3	2050.00	388.00	395.80	394.57	398.02	0.003324	11.96	171.37	22.95	0.77
Downstream	11510	PF 1	9500.00	387.00	402.57	402.57	404.25	0.001736	13.82	1328.24	389.38	0.62
Downstream	11510	PF 2	3160.00	387.00	397.23	397.23	399.66	0.002700	12.92	294.19	82.85	0.72
Downstream	11510	PF 3	2050.00	387.00	395.83		397.51	0.002157	10.43	204.45	45.32	0.63
Downstream	11399	PF 1	9500.00	386.50	398.30	392.88	398.35	0.000070	2.95	5825.39	1046.06	0.15
Downstream	11399	PF 2	3160.00	386.50	397.33	391.42	397.34	0.000013	1.18	4852.38	961.73	0.06
Downstream	11399	PF 3	2050.00	386.50	397.00	391.00	397.00	0.000006	0.82	4539.22	932.97	0.04
Downstream	11379		Inl Struct									
Downstream	11359	PF 1	9500.00	386.00	397.73		397.80	0.000090	3.30	5268.47	996.92	0.17
Downstream	11359	PF 2	3160.00	386.00	397.23		397.24	0.000013	1.21	4774.40	952.83	0.06
Downstream	11359	PF 3	2050.00	386.00	396.93		396.93	0.000006	0.84	4491.63	926.65	0.04
Downstream	11250	PF 1	9500.00	385.00	397.72	392.89	397.78	0.000088	3.34	5291.29	995.42	0.17
Downstream	11250	PF 2	3160.00	385.00	397.23	391.28	397.23	0.000012	1.22	4812.10	952.62	0.06
Downstream	11250	PF 3	2050.00	385.00	396.93	389.36	396.93	0.000006	0.85	4530.52	926.55	0.04
Downstream	11230		Inl Struct									
Downstream	11210	PF 1	9500.00	383.00	390.61	390.61	392.26	0.002556	13.92	1207.44	377.33	0.89
Downstream	11210	PF 2	3160.00	383.00	388.11	388.11	389.26	0.002326	10.16	481.20	217.02	0.80
Downstream	11210	PF 3	2050.00	383.00	387.28	387.28	388.32	0.002392	9.14	320.26	169.94	0.78
Downstream	10966	PF 1	9500.00	380.00	388.38		388.87	0.000490	6.79	2662.25	1134.46	0.41
Downstream	10966	PF 2	3160.00	380.00	385.78		386.14	0.000414	4.87	733.68	347.29	0.36
Downstream	10966	PF 3	2050.00	380.00	384.79		385.02	0.000338	3.88	531.56	111.92	0.31

Each of the columns in Table 12 represents a piece of information from the hydraulic model. The HEC-RAS model is inputted with data such as the channel configuration and the slope of the streambed. Once the model is set up, multiple flows of water can be tested and the results of how the water will flow are recorded both in a tabular form and in a graph. In Figure 37, three flows are tested at once, the ½ PMF, the 1/6 PMF and the 500 year flood which respectively correspond to 9,500 cfs, 3,160 cfs and 2,050 cfs. With the known streambed elevations and the known channel cross sections, the program will tabulate the elevation of the water depending on the given flood.

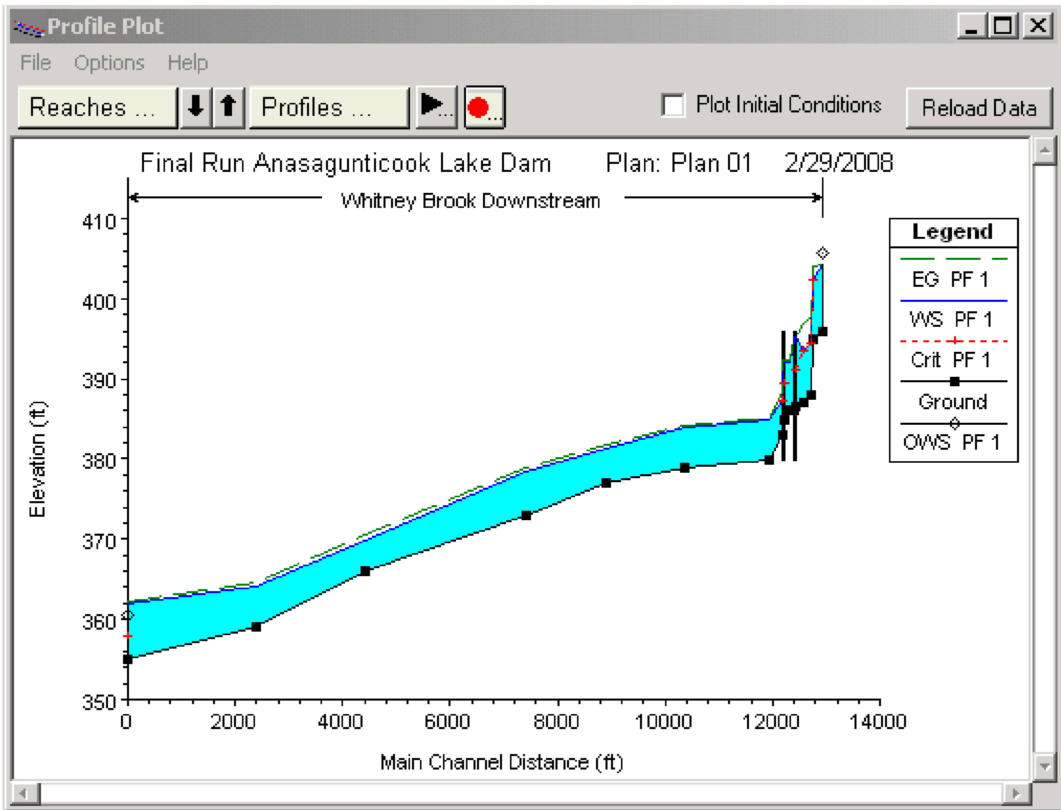

Figure 37 – HEC-RAS graphical output of Lake Anasagunticook dam 500 year flood for crest gate configuration.

Figure 37 shows the model of the 500 year flood flowing over the open crest gates. By looking at the predicted elevation of the water and comparing it to the known heights of the embankments, it can be determined if water will be overflowing the embankments. It can be a bit difficult to precisely determine the elevations of the water from the graphical output however, the numbers are very clear in Table 12. This case shows that there will be 1.7 feet of extra spillway capacity before overtopping occurs making the design suitable. If the other flows modeled are analyzed, the crest gate design will actually pass the 1/6 PMF event with 0.2 feet of extra spillway capacity. However, it will not pass the ½ PMF flow. During the ½ PMF flood, Whitney Brook is at elevation 407.84 msl or 3.84 feet above the abutments of the dam. This is with almost no obstruction in the channel. The collapsible crest gate design will pass the 500 year flood with no overtopping of the structure and will fulfill the design requirements of the Anasagunticook Lake Dam site.

4.3 Preliminary dam design results

The lower two proposed sites (i.e. locations 1 and 2) for a new dam are located on the existing dam property parcel, while the upper two sites are not located on the existing property parcel. The lower two sites do not have as good of a potential for using the natural 420 msl contour line as the steep terrain begins to spread out. However the land to construct a new dam will not have to be purchased or taken if a new structure is place on the existing property. The upper two locations have the benefit of having the 420 msl contour less spread out which will reduce the civil works costs but increase the property acquisition cost and would cut off lake access to several downstream property owners.

If a new dam site upstream of the existing dam is selected, the existing dam will have to be removed so that it is not obstructing flow. While the demolition will involve some cost, this extra cost will be outweighed by the benefit of eliminating or reducing the length of the embankments. Considering the land acquisition cost, water rights of the current land owners, natural topography and construction costs, the best location of a new dam upstream of the existing dam is at location 1 or 2. There is no major difference between locations 1 and 2 so in an effort to keep the area as similar to the existing state as possible, the dam will be located at location 1.

When comparing the three dam designs, it is relatively easy to identify that the rock-filled gravity dam will not be feasible for several reasons. The rock-filled gravity dam will never change size or height regardless of the volume of water flowing over it. This means that it is much more difficult to justify changing the design flood from the ½ PMF to the 500 year flood, as is the case with the crest gates. It was previously discussed that the water level must be kept at 402 msl and the top of the right embankment is 404 msl leaving only a two foot area for water to safely flow. For the rock-filled gravity dam, with the design flood being 9500 cfs (1/2 PMF) and the area to pass the water only being two feet high, the required dam length is approximately 1200 feet long. A 1200-foot long dam is not a reasonable choice for this site. There is not enough room on the property to excavate a foundation. Additionally, the $500,000 costs for the excavation is prohibitive.

To fix the existing dam site involves several problems of its own. The first is that to accurately go through the analysis of an embankment, as described in the methodology section, has been deemed impractical. The lack of uniformity in the soil and excessively poor condition of the embankment makes it impossible to precisely asses its ability to perform as a water retaining structure. The embankment will have to be removed and rebuilt as it is impossible to accurately assign values of particle size and shape as well as other important characteristics required for analysis. The concrete spillway itself is in fair condition and will need a lot of superficial work, such as fixing the spalling concrete. The gates contained in the spillway need a lot of work to function adequately. The gate structure will need to be redesigned such that the gates are operable. Lastly, the existing dam in its current state will not pass the ½ PMF and an emergency spillway has to be constructed to add flood capacity. Generally, emergency spillways are not a problem, however in this case, the difference between the design flood flow of 9,500 cfs and the capacity of the gravity spillway of 1,057 cfs is 8,843 cfs. This is a very large flow for an emergency spillway to have to pass and therefore, it has to be very large and extremely well reinforced.

The rubber dam was an interesting early idea that seemed ideal for the site. The basic information was placed in section two because it is an unusual idea that could work well at another site, however it was deemed impractical at this site. A price was requested for the Lake Anasagunticook dam but the quote came back at well over $750,000, which was too costly. Additionally, it was found that there is a problem with cutting of the rubber from both vandals and bottles compressed under the deflated rubber during a storm. Because of this information, the dam was found to be impractical and no further investigation into design continued.

The crest gates have been chosen as the best design for this site for several reasons. When flood flows are present, the crest gates are designed to open and lay flat against the channel bottom. As discussed in the hydraulics section, if a complete analysis of the hydraulics is completed from Lake Anasagunticook to the Androscoggin River, the level of the water (at the 1/2 PMF) will be at elevation 407.84 msl. As seen in Figure 38, at the location where the crest gate dam will be constructed, the left and right embankment elevations are respectively 406 msl and 404 msl. The left embankment is not natural and is higher than the natural topography. The right embankment is natural and is more like the elevations found in Canton center. In the event that the flood waters reached an elevation of 407.84, there would be 3.84 feet of water flowing over the right embankment and at least as much in town.

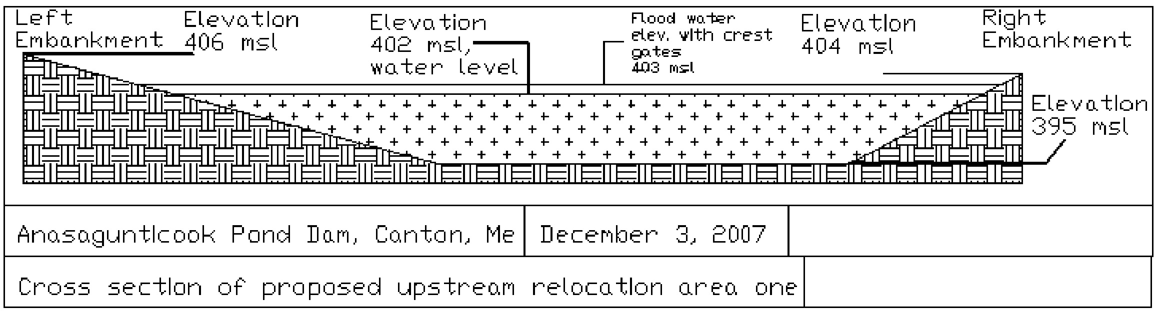

Figure 38 – Channel cross section at crest gate location. The flood water elevation is for the 500 year flood

Unless the resources are present and it is deemed practical to widen the river channel from Lake Anasagunticook all the way to the Androscoggin River, then there is no solution to the flooding situation. It is a natural problem that has existed since before the dam was originally constructed. The crest gates are an optimum solution because they will not contribute to the flooding problem by adding another constriction in the river channel. Instead, the crest gates fold down to the channel bottom maximizing the flow capacity of the channel and will never pose a threat to the town from a sudden release of water. Additionally, crest gates are very simple to design and construct.

The design calculations for the concrete gravity dam and the crest gates are located in Appendix E. The final crest gate design is discussed and analyzed in more detail in the following sections.

4.4 Final dam analysis results

In section 4.3, it was determined that the best dam design for the site is the crest gates. Summaries of the design results pertinent to construction as well as the specific results of the cost estimate are given below.

4.4.1 Structural analysis

The structural analysis of the dam was based on the results from the hydrologic and hydraulic analyses as well as the guidelines presented by the USACE. There were some old dam inspection reports that were utilized in analyzing the existing site, however they could not be used for the crest hinge gates. To help with the general layout and design criteria of the hinge crest gates, the design calculations of the Collinsville dam in Wilbraham, MA were used (See Figure 39).

Figure 39 – The crest gate dam is visible because of repairs. (Wilbraham, Ma)

To complete the structural analysis as described in section 3.4.3 some assumptions had to be made. These assumptions included:

- The structure is supported by Granite ledge, which does not have any major cracks, fissures or fault lines running through it.
- During normal conditions, the crest gates are at a 60 degree angle from the horizontal.
- During the design flood, the gates are completely horizontal.
- Uplift forces act linearly over the base of the dam.
- The slope at the base of the dam is zero.

The first step in design is determining the various external loadings associated with the dam. Once these were found, the factored loading combinations were found and the internal analysis began. With the maximum loading known, the shear and moment values were calculated to size the main steel gate vertical members for internal forces. The bending and shear values will be the basis for the wood slats' selection. The hydraulic cylinders for the gate actuation were selected based on the operating pressures, bore sizes and bore stroke. The maximum cylinder force was found to determine the required spacing between hydraulic cylinders. The concrete pad that the crest gates tie into is designed to ACI standards. The overview of the crest gates shown in Figure 40 is looking upstream. The left and right embankments are shown as earth mounds on either side of the 125 feet of wooden slats. Each wooden slat section is 6 feet wide and separated from its neighbor by a steel I beam. The dam is seven feet tall and sits on a concrete slab, two feet thick that is anchored to the stream bedrock. Each of the wooden slat section is held up by a hydraulic cylinder actuating on the steel beam. This is easier to see in Figure 41, which shows the side view of a set of slats held up by the hydraulic cylinder. Additionally in Figure 41, the dimensions and layout of the concrete foundation slab are clearer.

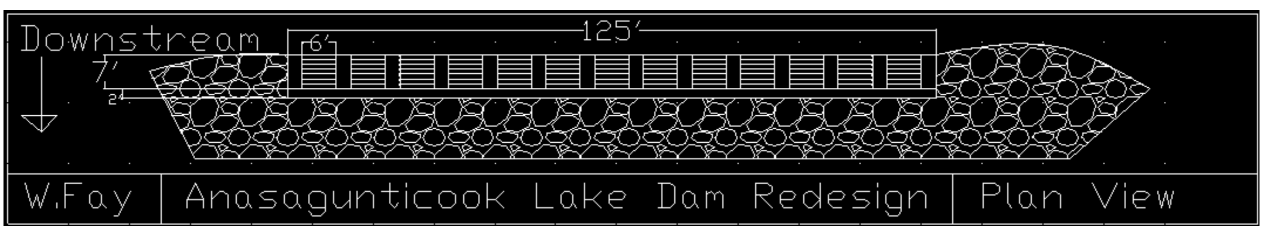

Figure 40 – Overview of Crest Gates

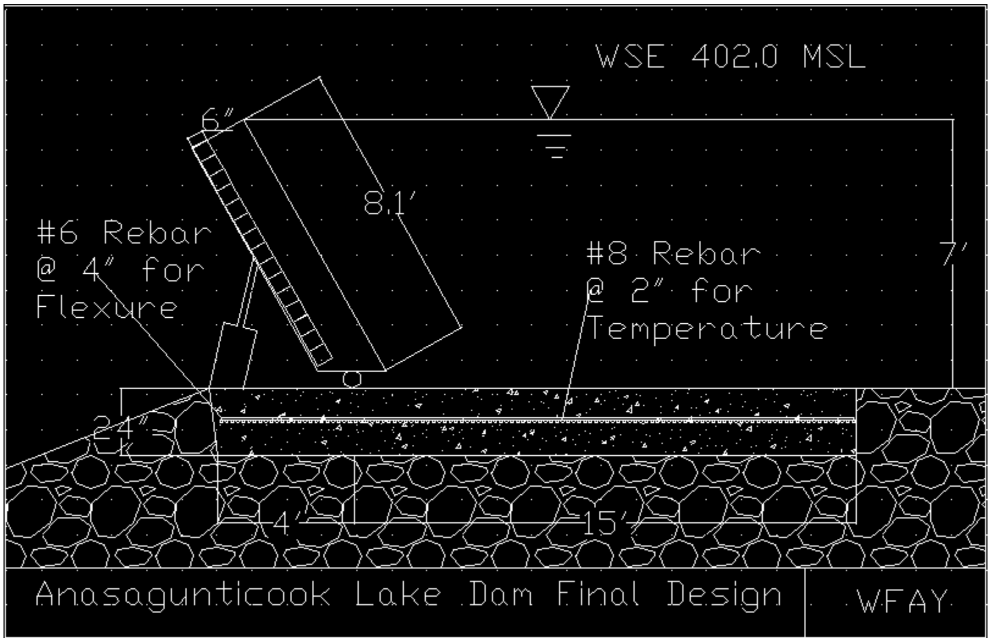

Figure 41 – Side view of crest gate design.

4.4.1.1 External Forces

The external forces looked at the dam as a whole and ensured that the shape and size will be able to withstand all anticipated forces. Figure 42 shows a free body diagram of the external forces acting on the dam. Table 13 summarizes the values of the various external horizontal and vertical forces acting on the dam cross section. Table 14 summarizes the values of the various external moments acting around the dam section.

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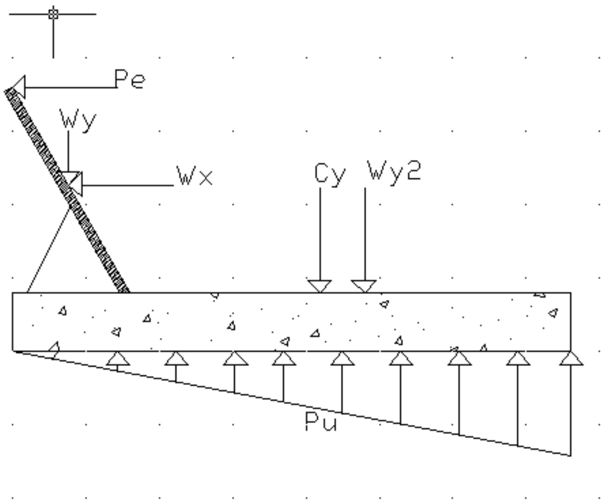

Figure 42 –External forces on dam.

Table 13 – Summary of Loads on Dam

Load	**X direction**	**Y direction**
Hydrostatic loading (Wx,Wy,Wy2)	**885#/ft**	**3058 #/ft**
Dead load (Cy)		**4287 #/ft**
Earthquake (Pe)	**37.2 #/ft**	
Uplift (Pu)		**4161 #/ft**

Table 14 – External Stability Continued.

FORCE	Failing	Resisting	Factor of Safety
Overturning	43 927 ft-lb	129 638 ft-lb	OK
Sliding	1 421 lb	9 735 lb	OK
Eccentricity	3.162 ft	3.175ft	OK
Bearing Force	1092lb/ft^2	2500lb/ft^2	OK

4.4.1.2 Internal stability

These values represent the forces that must be resisted in order to maintain stability

1. Maximum factored load = Pu=5555 lb
2. Maximum moment = $M_{u\max} = 133K - ft$
3. Maximum shear = $V_{n\max} = 38.2K$

4.4.1.2.1 Sizing steel beam

Using the maximum moment, the required cross sectional area of the beam was calculated to be 47.8 cubic inches. Based on that area, a W21X112 beam, which has a cross sectional area equal to 48.8 cubic inches was chosen. Next, using the maximum shear value, the required cross sectional area of the steel beam was calculated and compared to the area of the W21X112. For shear, the required area is 1.94 square inches meaning that the W21X112 with an area of 48.8 cubic inches will be more than adequate to cover the area required to withstand the maximum moment and the maximum shear as seen in Table 15.

Table 15 – Comparison of required areas for W21X112

	REQUIRED AREA (square inches)	ACTUAL AREA (square inches)
Moment	47.8	48.8
Shear	1.94	48.8

4.4.1.2.2 Sizing wooden slats

Again using the maximum shear and moment, the wooden beam sizes were picked. To resist the maximum moment or bending resistance, it was calculated that a minimum of 20.93 cubic inches is required and for the shear, a minimum of 16.1 cubic inches is required. Based on these numbers and common beam sizes, 6" by 6" beams were found adequate to resist the shear and moment as seen in Table 16.

Table 16 – Comparison of required areas for 6" by 6" beam

	REQUIRED AREA (square inches)	ACTUAL AREA (square inches)
Moment	20.93	36.0
Shear	16.1	36.0

4.4.1.2.3 Sizing hydraulic cylinders

The hydraulic cylinders were checked using USACE equations. It was found Pmax is equal to 39.26 k and Dmax is 6.21 feet. This means that when the cylinders are picked, they should be able to take a compression of 39.26 kips over whatever the total area of the cylinder is. When constructed to ensure that the cylinders are adequate, they should be spaced no more than every 6 ft.

4.4.1.2.4 Design of concrete pad

Finally, the concrete pad was designed to AIC standards. The analysis began by solving for the required depth of the footing. The required area of the footing was found to be 19.14 inches however for constructability purposes, this was increased to a standard 24 inches. The required steel reinforcement in the slab was calculated by solving for the required minimum cross sectional area of the steel. The minimum required area of steel turned out to be 1.132 square inches. Again, for standardization and constructability purposes, #6 bar with a cross sectional area of .44 square inches, spaced every 4 inches was the final conclusion.

Table 17 is a summary of the different internal design components as well as the final sizes and where pertinent, the required spacing.

Table 17 – Summary of design components

Component	Size	Spacing
Concrete Pad	24 inch thick	
Metal beams	W21X112	6 ft
Wood slats	6”X6”X6ft	
Hydraulic cylinders	Can take at least 39.2 kips over total area of cylinder	6 ft
Rebar	#6	4 inch

4.4.2 Cost Estimate

The cost estimate for the crest gate dam was based on RS Means values of heavy construction. Many of the components required calculations to estimate the costs and included finding volumes and quantities of materials. Some of the values such as the duration of a particular part of construction had to be estimated. The chart of expenses which makes up the estimate is in Figure 43. The chart contains several pieces of information about each item in the estimate including the unit, cost per unit, number of units and where applicable (such as renting equipment by the day) the required time. In some cases, such as in the case of over head, the cost per unit is a percentage of the total project and is added at the end of the project. The items included in the estimate include, but are not limited to: welders, concrete pumps, demolition, de-watering, coffer dams, grading, excavation and building supplies. The estimated cost of the crest gate design is $111,000.

Crest Gate Design

Division	Cost/Unit	# of units	Unit	Time	Total	notes
Division 1 - General requirements						
Subdivision:	The 25% O&P is already included in the RS means					
1310 Project Management/Coordination	cost estimating data					
620 Overhead and Profit (25% of total)	25%					
700 Field Personel	995	10	week		9950	
1500 Temporary facilities and controls						
50 Generator	1250	2	each		2500	
50 Welder	920	1	each		920	
1590 Equipment rental						
100 concrete pump	704	2	day		1408	
100 vibrators	11.3	5	each	2 days	113	
100 Concrete batch truck	43.05		day	2 day	86.1	
400 Restrooms	23.75	80	day		1900	
1560 Barriers and Encolsures						
250 temporary fencing	6.75	440	linear foot		2970	
1580 Project signs						
700 Signs	18	10	each		180	
Division 2 - Site construction						
Subdivision:						
2220 Site demolition						
310 Demolition					10000	
120 Disposal of material	$9.00	40	cy		1000	
2240 Dewatering						
500 Dewatering	7.3		cy		5000	
2260 Excavation support/protection						
200 Coffer dams	35	300	sf		10500	
2310 Grading						
100 Finsih grading	$2.37	1500	sy		3555	
2315 Excavation & Fill						
110 Channel excavation	$6.15		cy		3000	
Division 3 - Concrete:						
Subdivisions:						
3060 Basic Concrete materials						
200 Cement	$75	169	cy		12700	
3200 Concrete Reinforcement						
100 #6 bar	$0.45	7011	lb		4900	
100 #8 bar	$0.45	720	lb		900	
Division 5 - Metals						
Subdivisions:						
5120 Structural Steel						
640 structural steel members (I beams)	$120		Lf		17000	
Division 6 - Wood and Plastic						
Subdivisions:						
6100 Wood						
555 Beams	$2,100	1.68	1000 feet		12600	
Division 15 - Mechanical						
Hydraulic cylinders	$450	20	each		9000	
Total					$111,000	

Figure 43 – Summary of crest gate costs.

Chapter 5: Summary and Recommendations

In summary, this project looked at various design options to either fix or replace the deteriorating and undersized dam at Lake Anasagunticook in Canton, Maine. The final dam design was based upon the results of the hydrologic analysis of the drainage area, which predicted the volumes of flood waters that the dam would be exposed to, the hydraulic analysis that determined the optimum dimensions of the dam, the structural analysis that ensured both external and internal stability and the cost estimate. When all assessments had been completed and compared, the crest gate design proved to be the best for the site.

The hydrology of the drainage area was modeled using the USACE HMR-51, HMR-52 and HEC-2 programs. The value of the ½ PMF was found to be 9500 cfs. Due to the natural valley constrictions, the ½ PMF flood value was determined unreasonable and the 500 year flood was used to design the crest gates. The 500 year flood was found from FEMA flood maps and is 2,050 cfs.

The hydraulics of the site was modeled using the USACE HEC-RAS program. The results of the HEC-RAS model were checked using basic hydraulic modeling equations. The hydraulic analysis produced the required information to size the dam ensuring adequate spillway capacity during flood conditions. As seen in Figure 38, with the 500 year flood passing over the lowered crest gates, the height of the water in the channel is at 403.02 msl leaving approximately one foot of embankment exposed before overtopping begins.

The external stability analysis of the crest gates found that if the dam were 125 feet long and seven feet tall with a concrete pad 19 feet wide, it would be able to resist the forces at the site. The internal analysis revealed that W21X112 beams spaced every 6.5 feet fitted with 16 vertically stacked 6” by 6” wood beams all the way across the dam would be satisfactory. The 19 foot wide, 125 foot long concrete pad was required to be 24 inches thick and reinforced with #6 rebar spaced every 4 inches.

The cost estimates were based on RS Mean values for heavy construction and included profit and overhead. The final cost of the crest gate dam was $111,000, which was relatively inexpensive compared with the estimates of $261,000 to fix the existing site and $690,000 for the gravity dam. The crest gates was chosen as the best design for the site because it has the advantage of collapsing to the channel bottom during floods as well as being both cost effective and constructible.

The goals expressed in the capstone design section of this project were met. The dam structures were designed following capstone guidelines to meet the needs of the all parties including the community. A real world, open ended problem in the field of civil engineering was researched and analyzed.

Appendix A – HMR-52 PMP detailed calculation procedure

The HMR-52 PMP calculation methodology is as follows. First the drainage area size is determined using the Use Topo Scout. See Figure 21 with the drainage basin delineated by markers and boundary lines.

Then the probable maximum precipitation has to be determined. This is started by plotting the drainage basin image on 10 by 10 graph paper with a plot scale. Arbitrarily a coordinate axis system is set up with the drainage area basin marker coordinates in inches. This will produce the drainage area in a matrix format that the HMR-52 program can recognize and perform calculations with.

Next HMR-52 is downloaded and installed from the USACE website with a new worksheet open. The program is set up with a “card” system. Each card represents a space in a matrix that the HMR-52 program uses to organize and find data used for calculations. The BS card is set to a scale of 1.0 and then the shape factors Bx and By in miles from the graphed map described in the above section are inputted. Set the Pl card to one in order to plot and check basin shape. This output is compared to the Topo Scout plot to determine if the data was imputed correctly. Finally the card HO is set to 1.0 which is defined by the user’s manual for the storms orientation to the drainage area.

The HP factors have to be determined next. Hydro-Meteorological Report N. 51 from the National Weather Service is used to obtain depth-area-duration values from the 10, 200, 1000, 5000, 10,000, and 20,000 square mile curves.

Next, the SA card is set to 12 inches on field three. The model will use 12 six hour periods for computing the maximum precipitation on the drainage basin. Leave field one set to zero. The model calculates several storm area sizes and selects the area size which produces the maximum precipitation on the drainage basin for the specified number of six hour periods.

After, use the ST card to set the temporal distribution of the PMS for intervals less than six hours. Set field one to 60 minutes. This is the time interval to be used for the temporal distribution of the PMS. Set field two to 0.318. This is the ratio of one hour to six hour precipitation for isohyetal A of the 20,000 sq. mi. storm, for Maine, from Figure 39 of HMR No. 52. Note that the normal range for the U.S. east of the 105th meridian is 0.27 to 0.35. Leave field three blank so that the previously established arrangement of 6-

hour increments of PMS will be used. Set field four to 1.0. It is better to scale the predicted PMF precipitation increments in HEC-HMS using the HEC-HMS PMF ratio multiplier

Lastly, remember to end with the ZZ end of file card to close the data collection portion of HMR-52. The input file used for HMR-52 is inserted.

The possible maximum flow was found using HEC-HMS which can calculate PMF flow using different methods of mathematically describing how rainfall will flow in a drainage area. The Soil Conservation Service (SCS) method was used in our model.

SCS parameters are drainage area characteristics that the SCS uses to calculate the PMF. First the SCS parameters used in the HEC-HMS model needed to be calculated. The drainage basin topography, development amount and soil types were assigned a curve number (CN) value determined from the data in chart #. Therefore an approximate average SCS curve number (CN) was estimated. This value ranged from 55 to 70 depending upon percentage of urbanization of the watershed and the predominate soil type in the area.

The HEC-HMS modeling procedure is a s follows.

First HEC-HMS 3.10 was downloaded and installed from the USACE, HEC website at http://www.hec.usace.army.mil/software/hec-hms/download.html Next click on the create new project icon. On the drop down menu, fill in the project name and description. The default unit system should be set to U.S. customary. After exiting the new project drop down menu, the project name screen will come up.

Next on the main menu bar, select components, basin model manager and click new. Input a name and description in the basin model drop down menu. Then click create new. Note the basin model folder appears in the work area view port.

Repeat this procedure for the meteorological model manager, controls specification manager, time series data manager, and the paired data manager. Do not use the grid data manager.

Once all the components are created, open the basin model folder and click on the basin icon. The basin model, gridded screen, view port opens. Notice the basin model tool bar has been activated. Move the cursor over the gridded area and click near the center of the grid. A “create sub basin element” drop down menu appears. Fill in the name and a description of the basin. Click the create button. Note that the basin icon is now located on the basin model view port. In the bottom left corner of the screen, in the data entry view port, under unit system, change the units to U.S. customary.

Go back to the basin model folder in the navigation view-port. Note the newly named basin icon for the newly created sub basin appears in the folder. Expand the folder. Notice the data entry view port, on the bottom left of the screen, has changed.

There are four tabs in this menu, on the data input view port. They are titled sub-basin, loss, transform, and options. Fill in the basin description and the drainage area in square miles. We are using the SCS method. Click on the arrow and find SCS curve number and click on it. (not, girded SCS curve number). Click on the transform method tab. Click on SCS unit hydrograph. We are not using a base flow method, so click on none. Click on the loss tab and fill in the basin averaged SCS number. Input the desired percent impervious. Then click on the transform tab. Fill in the time lag in minutes.

In the navigation window, click on the meteorological model and then click on the cloud icon. In the data entry window, there are three tabs called meteorology model, basins and options.

Click on the meteorology model tab. Fill in the description. Under precipitation, click on the down area and click on specified hyetograph. Remember, we are using the 72 hour, incremental rainfall, PMF hyetograph that we developed with HMR-52. We are not using evapo-transpiration, so specify none. We are not using the snowmelt option, so specify none. Make sure the system units are set to U. S. customary. Click the basins tab.

Click on include sub basins and click yes. Leave the options tab on its default settings. These should be replace missing "yes" and total override "no".

In the navigation window, click on the specified hyetograph, a rain drop icon. In the data entry window, click on the down arrow. Switch it from "none" to "gage data".

In the navigation window, open the control options folder and click on the clock icon. In the data input window, fill in the description. For illustrative purposes, assume the rainfall event starts on New Years Eve. Fill in the start date as 01JAN2008. Use a start time of 00:00 hours. Since the HMR 52 hydrograph is 72 hours or three full days, the end date is the zero hour of the fourth day. This makes the end date 04JAN2008 and the ending hour must be 00:00. The time interval is in hours. Enter these parameters in the provided boxes. Click the down arrow on the time interval field and click on hours.

In the navigation window, open the time series data folder and click the rain gauge icon. In the data input window, fill in the description. In the data source, click the down arrow and select "manual entry". In the units field, click the down arrow and select one hour. Fill in the latitude and longitude fields for degrees, minutes and seconds. Use whole numbers ie: no decimals.

In the navigation window, open the time series data folder, click the rain gauge icon and than click on the table icon. The table icon should have "01JAN2008, 00:00-04JAN2008, 00:00 following it.

In the data input window, click on the table tab. Fill in the 72 incremental precipitation fields with there corresponding inches of incremental rain from the HEC-HMR 52 output. These fields start with 01JAN2008, 00:00 and end with 04JAN20008, 00:00.

In the navigation window, open the paired data folder, click the graph icon. In the data input window, there are three tabs, paired data, table and graph. Click on the paired data tab. Fill in the description. In the data source field, click the down arrow and select manual entry. In the units field, click the down arrow and select FT:CFS. Click on the table tab. In the first elevation field, input the elevation of the overflow weir. In the corresponding CFS field, input zero (0). In the next elevation field, input the elevation of the top of the abutments. In the corresponding CFS field, input the hydraulic capacity of the spillway.

On the main tool bar, at the top of the screen, click the compute button. Select "create simulation run". A create simulation run drop down menu appears. Fill in the name of the run. Click next, the drop down changes to a list of basin models. In the previous steps, you have created and named a basin model. This basin model and possibly others will be listed. Click on your model to select it and click next. The drop down menu changes to a list of meteorological models. Your meteorological model should be listed. Click on your model and click next. The drop down menu changes to a list of control specifications. Your control specifications should be listed. Click on your control specifications and click finish.

On the main tool bar, at the top of the computer, click the compute button, than select "run". A sub menu appears with a list of all the created simulation runs. Select your run by clicking on it.

On the main tool bar, at the top of the computer, click the compute button. At the bottom of the drawdown menu, select "compute run". After the run is complete, click the close button.

On the bottom of the navigation window are three tabs. They are components, compute and results. Until now, we have been exclusively using the components tab. Click on the results tab. Click on the name of your run. The results of the run are listed as summary, outflow, incremental precipitation, excess precipitation, precipitation loss, direct run off, and base flow.

The summary is a table listing the numerical results. The other results are graphs which are displayed in the area of the data input window. This finishes the HEC-HMS run and HEC-HMS will have computed your PMF.

Appendix B – HMR-52 output

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 ****************************************
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 *                                      *                                                    *
*
 *    PROBABLE MAXIMUM STORM  (HMR52)    *                                                    *    U.S.
ARMY CORPS OF ENGINEERS     *
 *             NOVEMBER 1982            *                                                    *  THE
HYDROLOGIC ENGINEERING CENTER  *
 *           REVISED  APRIL 91          *                                                    *
609 SECOND STREET         *
 *                                      *                                                    *
DAVIS, CALIFORNIA 95616       *
 *  RUN DATE 09/29/2007    TIME 22:34:17 *                                                    *  (916)
551-1748 OR (FTS) 460-1748   *
 *                                      *                                                    *
*
 ****************************************
***************************************



                                             H     H  M     M  RRRRRR   5555555   22222
                                             H     H  MM   MM  R     R  5        2     2
                                             H     H  M M M M  R     R  5              2
                                             HHHHHHH  M  M  M  RRRRRR   555555        2
                                             H     H  M     M  R   R          5     2
                                             H     H  M     M  R    R   5     5   2
                                             H     H  M     M  R     R   55555   2222222
1                                      HEC PROBABLE MAXIMUM STORM (HMR52) INPUT DATA
PAGE  1

           LINE
ID.......1.......2.......3.......4.......5.......6.......7.......8.......9......10

             1           ID  Anasagunticook Lake Dam Study
             2           ID  HMR52 Probable Maximum Storm Input Data ***PMS***
             3           ID  Whitney Brook, Canton, Maine

             4           BN   TOTAL
             5           ID  Drainage Basin Geometry - Calculate Storm Over Uncontrolled Basin
             6           BS     1.0
             7           BX   3.788   4.402   5.286    5.26   5.513   5.959   5.934   5.631    5.21
4.899
             8           BX   4.469   3.788   3.114   2.954   2.466   2.256   2.828   1.683   1.557
1.145
             9           BX   0.842   0.539   0.168   0.185   0.354   0.589   1.347   1.961    2.39
2.626
            10           BX   2.799   3.072   3.283   3.518
            11           BY   6.270   4.974   3.964   2.862   3.261   2.853     2.5   2.230    2.02
2.104
            12           BY   2.693   2.862    3.19   3.451   3.695   4.103   4.293   4.377   4.524
4.731
            13           BY   5.117   5.618   6.018   6.481   6.628   7.078   7.323   7.365   7.112
7.390
            14           BY   7.398   7.154   6.818   6.313
            15           PL       2
            16           ID  HYDROMETEOROLOGIAL DATA FROM HMR No. 51
            17           HO     230
            18           HP      10      21      24      26      30      32
            19           HP     200      14      17      19      22      23
            20           HP    1000      10      13      16    18.5    19.5
            21           HP    5000     6.5     9.3    11.5      14      15
            22           HP   10000       5     7.8    10.2    12.3    13.5
            23           HP   20000     3.7     6.5     8.5      11      12
            24           ID  STORM SPECIFICATIONS
            25           SA                      12
            26           ST      60    .335             1.0
            27           ZZ
1****************************************
***************************************
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 *                                      *                                                 *
*
 *    PROBABLE MAXIMUM STORM  (HMR52)    *                                                 *   U.S.
ARMY CORPS OF ENGINEERS     *
 *             NOVEMBER 1982             *                                                 *  THE
HYDROLOGIC ENGINEERING CENTER  *
 *           REVISED  APRIL 91           *                                                 *
609 SECOND STREET          *
 *                                       *                                                 *
DAVIS, CALIFORNIA 95616       *
 *  RUN DATE 09/29/2007    TIME 22:34:17 *                                                 *  (916)
551-1748 OR (FTS) 460-1748   *
 *                                       *                                                 *
*
 ****************************************
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```

Anasagunticook Lake Dam Study
HMR52 Probable Maximum Storm Input Data ***PMS***
Whitney Brook, Canton, Maine
Drainage Basin Geometry - Calculate Storm Over Uncontrolled Basin
HYDROMETEOROLOGIAL DATA FROM HMR No. 51
STORM SPECIFICATIONS

PMP DEPTHS FROM HMR 51

AREA	DURATION				
(SQ. MI.)	6-HR	12-HR	24-HR	48-HR	72-HR
10.	21.00	24.00	26.00	30.00	32.00
200.	14.00	17.00	19.00	22.00	23.00
1000.	10.00	13.00	16.00	18.50	19.50
5000.	6.50	9.30	11.50	14.00	15.00
10000.	5.00	7.80	10.20	12.30	13.50
20000.	3.70	6.50	8.50	11.00	12.00

STORM AREA	PMP DEPTHS FOR 6-HOUR INCREMENTS											
10.	21.14	2.35	1.67	1.29	1.06	.89	.77	.68	.61	.55	.50	.46
25.	19.47	2.45	1.64	1.24	.99	.83	.71	.63	.56	.50	.46	.42
50.	18.08	2.53	1.62	1.19	.94	.78	.67	.58	.51	.46	.42	.38
100.	16.03	2.67	1.56	1.11	.86	.70	.60	.52	.46	.41	.37	.34
175.	14.39	2.77	1.51	1.04	.80	.65	.54	.47	.41	.37	.33	.30
300.	12.97	2.91	1.52	1.04	.79	.64	.53	.46	.40	.36	.32	.30
450.	11.93	3.03	1.55	1.05	.79	.64	.54	.46	.40	.36	.32	.30
700.	10.81	3.16	1.58	1.06	.80	.64	.54	.46	.41	.36	.33	.30
1000.	9.92	3.24	1.60	1.07	.81	.65	.54	.47	.41	.36	.33	.30
1500.	9.04	3.13	1.56	1.04	.79	.63	.53	.45	.40	.35	.32	.29
2150.	8.27	3.03	1.51	1.02	.77	.62	.52	.44	.39	.35	.31	.28
3000.	7.56	2.92	1.48	1.00	.75	.60	.51	.43	.38	.34	.31	.28
4500.	6.69	2.79	1.43	.97	.73	.59	.49	.42	.37	.33	.30	.27
6500.	5.89	2.80	1.41	.95	.72	.58	.48	.41	.36	.32	.29	.27
10000.	4.95	2.85	1.40	.94	.71	.57	.47	.41	.36	.32	.29	.26
15000.	4.21	2.76	1.39	.93	.71	.57	.47	.41	.36	.32	.29	.26
20000.	3.68	2.69	1.38	.93	.70	.57	.47	.41	.36	.32	.29	.26

1

BOUNDARY COORDINATES FOR   TOTAL

X	3.8	4.4	5.3	5.3	5.5	6.0	5.9	5.6	5.2	4.9
Y	6.3	5.0	4.0	2.9	3.3	2.9	2.5	2.2	2.0	2.1
X	4.5	3.8	3.1	3.0	2.5	2.3	2.8	1.7	1.6	1.1
Y	2.7	2.9	3.2	3.5	3.7	4.1	4.3	4.4	4.5	4.7
X	.8	.5	.2	.2	.4	.6	1.3	2.0	2.4	2.6
Y	5.1	5.6	6.0	6.5	6.6	7.1	7.3	7.4	7.1	7.4
X	2.8	3.1	3.3	3.5						
Y	7.4	7.2	6.8	6.3						

SCALE = 1.0000 MILES PER COORDINATE UNIT

BASIN AREA = 14.4 SQ. MI.

BASIN CENTROID COORDINATES, X = 3.0, Y = 5.0

1

VARYING STORM AREA SIZE AND FIXED ORIENTATION

STORM AREA	ORIENTATION	BASIN-AVERAGED INCREMENTAL DEPTHS FOR 6-HR PERIODS												SUM OF DEPTHS FOR 12 PEAK 6-HR PERIODS
10.	314.	19.91	2.21	1.57	1.22	1.00	.84	.73	.64	.58	.52	.48	.44	30.13
25.	314.	19.64	2.50	1.65	1.24	.99	.83	.71	.63	.56	.50	.46	.42	30.13
50.	314.	18.96	2.65	1.64	1.19	.94	.78	.67	.58	.51	.46	.42	.38	29.18
100.	314.	17.78	2.86	1.60	1.11	.86	.70	.60	.52	.46	.41	.37	.34	27.59
175.	314.	16.93	3.03	1.55	1.04	.80	.65	.54	.47	.41	.37	.33	.30	26.43
300.	314.	16.17	3.22	1.57	1.04	.79	.64	.53	.46	.40	.36	.32	.30	25.81
450.	314.	15.47	3.38	1.59	1.04	.79	.63	.53	.46	.40	.36	.32	.29	25.26
700.	314.	14.66	3.51	1.61	1.04	.78	.63	.53	.45	.40	.35	.32	.29	24.57
1000.	314.	14.06	3.59	1.61	1.03	.78	.62	.52	.45	.39	.35	.31	.29	24.01
1500.	314.	13.54	3.40	1.52	.98	.74	.59	.49	.42	.37	.33	.30	.27	22.94
2150.	314.	12.93	3.20	1.43	.91	.69	.55	.46	.40	.35	.31	.28	.25	21.76
3000.	314.	12.15	2.95	1.32	.85	.64	.51	.43	.37	.32	.29	.26	.24	20.33
4500.	314.	11.93	2.86	1.28	.82	.62	.50	.42	.36	.32	.28	.25	.23	19.88
6500.	314.	11.55	2.90	1.27	.81	.61	.49	.41	.35	.31	.27	.25	.23	19.44
10000.	314.	10.90	2.99	1.27	.80	.60	.48	.40	.35	.30	.27	.24	.22	18.83
15000.	314.	10.27	2.92	1.26	.79	.60	.48	.40	.35	.30	.27	.24	.22	18.11
20000.	314.	9.65	2.87	1.26	.79	.60	.48	.40	.35	.30	.27	.24	.22	17.45

FIXED STORM AREA SIZE AND VARYING ORIENTATION

SUM OF DEPTHS
ORIEN-
FOR 12 PEAK
STORM AREA TATION BASIN-AVERAGED INCREMENTAL DEPTHS FOR 6-HR PERIODS
6-HR PERIODS

10.	140.	19.90	2.21	1.57	1.22	1.00	.84	.73	.64	.58	.52	.48	.44	30.11
10.	150.	19.55	2.17	1.54	1.20	.98	.83	.72	.63	.57	.51	.47	.43	29.59
10.	160.	18.95	2.10	1.50	1.16	.95	.80	.69	.61	.55	.50	.45	.42	28.68
10.	170.	18.22	2.02	1.44	1.12	.91	.77	.67	.59	.53	.48	.44	.40	27.59
10.	180.	17.57	1.95	1.39	1.08	.88	.74	.64	.57	.51	.46	.42	.39	26.60
10.	190.	17.04	1.89	1.35	1.05	.85	.72	.63	.55	.49	.45	.41	.37	25.80
10.	200.	16.61	1.85	1.31	1.02	.83	.70	.61	.54	.48	.44	.40	.37	25.15
10.	210.	16.32	1.81	1.29	1.00	.82	.69	.60	.53	.47	.43	.39	.36	24.71
10.	220.	16.16	1.80	1.28	.99	.81	.69	.59	.52	.47	.42	.39	.36	24.47
10.	230.	16.14	1.79	1.28	.99	.81	.68	.59	.52	.47	.42	.39	.36	24.45
10.	240.	16.29	1.81	1.29	1.00	.82	.69	.60	.53	.47	.43	.39	.36	24.67
10.	250.	16.59	1.84	1.31	1.02	.83	.70	.61	.54	.48	.43	.40	.37	25.13
10.	260.	17.03	1.89	1.35	1.04	.85	.72	.63	.55	.49	.45	.41	.37	25.78
10.	270.	17.60	1.96	1.39	1.08	.88	.75	.65	.57	.51	.46	.42	.39	26.65
10.	280.	18.28	2.03	1.44	1.12	.92	.77	.67	.59	.53	.48	.44	.40	27.67
10.	290.	19.05	2.12	1.50	1.17	.95	.81	.70	.62	.55	.50	.46	.42	28.84
10.	300.	19.62	2.18	1.55	1.20	.98	.83	.72	.63	.57	.51	.47	.43	29.70
10.	310.	19.88	2.21	1.57	1.22	.99	.84	.73	.64	.57	.52	.47	.44	30.09
10.	309.	19.87	2.21	1.57	1.22	.99	.84	.73	.64	.57	.52	.47	.44	30.07
10.	139.	19.90	2.21	1.57	1.22	1.00	.84	.73	.64	.58	.52	.48	.44	30.12

1

PROBABLE MAXIMUM STORM FOR TOTAL

STORM AREA = 10. SQ. MI., ORIENTATION = 314., PREFERRED ORIENTATION = 230.

STORM CENTER COORDINATES, X = 3.0, Y = 5.0

ISOHYET	AREA (SQ.MI.)	AREA WITHIN BASIN (SQ.MI.)	DEPTHS (INCHES) FOR 6-HOUR INCREMENTS OF PMS 1	2	3	4	5	6	7	8	9	10	11	12
A	10.	10.	21.14	2.35	1.67	1.29	1.06	.89	.77	.68	.61	.55	.50	.46
B	25.	14.	13.53	1.50	1.08	.84	.69	.58	.50	.44	.40	.36	.33	.30
C	50.	14.	10.15	1.13	.80	.62	.51	.43	.37	.33	.29	.26	.24	.22
D	100.	14.	8.03	.92	.65	.50	.41	.35	.30	.27	.24	.22	.20	.18
E	175.	14.	6.34	.70	.50	.39	.32	.27	.23	.20	.18	.17	.15	.14
F	300.	14.	5.07	.56	.40	.31	.25	.21	.19	.16	.15	.13	.12	.11
G	450.	14.	4.02	.47	.33	.26	.21	.18	.15	.14	.12	.11	.10	.09
H	700.	14.	2.96	.33	.23	.18	.15	.12	.11	.10	.09	.08	.07	.06
I	1000.	14.	2.11	.23	.17	.13	.11	.09	.08	.07	.06	.06	.05	.05
J	1500.	14.	1.27	.16	.11	.08	.07	.06	.05	.04	.04	.04	.03	.03
K	2150.	14.	.42	.07	.05	.04	.03	.03	.02	.02	.02	.02	.02	.01
L	3000.	14.	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00
M	4500.	14.	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00
N	6500.	14.	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00

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 O 10000.       14.       .00     .00     .00     .00     .00     .00     .00     .00     .00     .00
.00     .00
 P 15000.       14.       .00     .00     .00     .00     .00     .00     .00     .00     .00     .00
.00     .00
 Q 25000.       14.       .00     .00     .00     .00     .00     .00     .00     .00     .00     .00
.00     .00
 R 40000.       14.       .00     .00     .00     .00     .00     .00     .00     .00     .00     .00
.00     .00
 S 60000.       14.       .00     .00     .00     .00     .00     .00     .00     .00     .00     .00
.00     .00

 AVERAGE DEPTH          19.91    2.21    1.57    1.22    1.00     .84     .73     .64     .58     .52
.48     .44
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                          *****         +++++++++++                  +++*****
                         *         +++++                                     ****+++++********
                       *      +++++                                                  ++++     *
                      *    +++                                                           +++   *
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TIME INTERVAL = 60. MINUTES
1-HR TO 6-HR RATIO FOR ISOHYET A AT 20000 SQ. MI. = .335

DEPTH VS. DURATION

ISOHYET	5MIN	10MIN	15MIN	30MIN	1-HR	2-HR	3-HR	6-HR	12-HR	18-HR	24-HR	30-HR	36-HR	42-HR	48-HR	54-HR	60-HR	66-HR	72-HR
A	1.36	2.73	4.09	7.89	12.49	15.23	17.42	21.14	23.49	25.16	26.45	27.51	28.40	29.17	29.86	30.47	31.02	31.52	31.99
B	.25	.49	.74	1.49	2.99	5.93	8.64	13.53	15.04	16.12	16.96	17.64	18.23	18.73	19.17	19.57	19.93	20.25	20.56
C	.18	.37	.56	1.12	2.24	4.45	6.48	10.15	11.28	12.08	12.70	13.20	13.63	14.00	14.33	14.62	14.89	15.13	15.35
D	.15	.29	.44	.88	1.77	3.52	5.12	8.03	8.95	9.60	10.10	10.52	10.86	11.17	11.43	11.67	11.89	12.08	12.26
E	.12	.23	.35	.70	1.40	2.78	4.05	6.34	7.05	7.55	7.94	8.25	8.52	8.75	8.96	9.14	9.31	9.46	9.60
F	.09	.19	.28	.56	1.12	2.22	3.24	5.07	5.64	6.04	6.35	6.60	6.82	7.00	7.17	7.31	7.44	7.57	7.68
G	.07	.15	.22	.44	.89	1.76	2.56	4.02	4.49	4.82	5.08	5.29	5.47	5.62	5.76	5.88	5.99	6.09	6.19
H	.05	.11	.16	.33	.65	1.30	1.89	2.96	3.29	3.52	3.70	3.85	3.98	4.08	4.18	4.27	4.34	4.41	4.48
I	.04	.08	.12	.23	.47	.93	1.35	2.11	2.35	2.52	2.65	2.75	2.84	2.92	2.99	3.05	3.10	3.15	3.20
J	.02	.05	.07	.14	.28	.55	.80	1.27	1.43	1.54	1.63	1.69	1.75	1.80	1.85	1.89	1.92	1.95	1.99
K	.01	.02	.02	.05	.09	.18	.26	.42	.49	.54	.58	.61	.64	.66	.68	.70	.72	.73	.75
L	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00
M	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00
N	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00
O	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00
P	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00
Q	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00
R	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00
S	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00
AVERAGE	1.18	2.37	3.55	6.86	10.95	13.72	15.99	19.91	22.12	23.69	24.91	25.91	26.75	27.48	28.12	28.70	29.22	29.70	30.13

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PROBABLE MAXIMUM STORM FOR TOTAL

DAY 1

TIME	PRECIPITATION INCR	PRECIPITATION TOTAL	TIME	PRECIPITATION INCR	PRECIPITATION TOTAL	TIME	PRECIPITATION INCR	PRECIPITATION TOTAL	TIME	PRECIPITATION INCR	PRECIPITATION TOTAL
0100	.07	.07	0700	.09	.52	1300	.11	1.07	1900	.14	1.74
0200	.07	.15	0800	.09	.61	1400	.11	1.17	2000	.14	1.88
0300	.07	.22	0900	.09	.70	1500	.11	1.28	2100	.14	2.02
0400	.07	.29	1000	.09	.78	1600	.11	1.39	2200	.14	2.16
0500	.07	.36	1100	.09	.87	1700	.11	1.49	2300	.14	2.30
0600	.07	.44	1200	.09	.96	1800	.11	1.60	2400	.14	2.44

6-HR TOTAL	.44	.52	.64	.84

DAY 2

TIME	PRECIPITATION INCR	PRECIPITATION TOTAL	TIME	PRECIPITATION INCR	PRECIPITATION TOTAL	TIME	PRECIPITATION INCR	PRECIPITATION TOTAL	TIME	PRECIPITATION INCR	PRECIPITATION TOTAL
0100	.19	2.63	0700	.30	3.96	1300	.90	6.78	1900	.30	26.08
0200	.19	2.82	0800	.29	4.25	1400	1.62	8.40	2000	.28	26.36
0300	.20	3.02	0900	.31	4.57	1500	2.78	11.18	2100	.27	26.63
0400	.21	3.22	1000	.36	4.93	1600	10.95	22.12	2200	.25	26.88
0500	.21	3.44	1100	.43	5.35	1700	2.27	24.40	2300	.24	27.12
0600	.22	3.66	1200	.52	5.87	1800	1.39	25.78	2400	.23	27.36
6-HR TOTAL	1.22			2.21			19.91			1.57	

DAY 3

TIME	PRECIPITATION INCR	PRECIPITATION TOTAL	TIME	PRECIPITATION INCR	PRECIPITATION TOTAL	TIME	PRECIPITATION INCR	PRECIPITATION TOTAL	TIME	PRECIPITATION INCR	PRECIPITATION TOTAL
0100	.17	27.52	0700	.12	28.47	1300	.10	29.18	1900	.08	29.74
0200	.17	27.69	0800	.12	28.60	1400	.10	29.27	2000	.08	29.82
0300	.17	27.85	0900	.12	28.72	1500	.10	29.37	2100	.08	29.90
0400	.17	28.02	1000	.12	28.84	1600	.10	29.47	2200	.08	29.97
0500	.17	28.19	1100	.12	28.96	1700	.10	29.56	2300	.08	30.05
0600	.17	28.35	1200	.12	29.08	1800	.10	29.66	2400	.08	30.13
6-HR TOTAL	1.00			.73			.58			.48	

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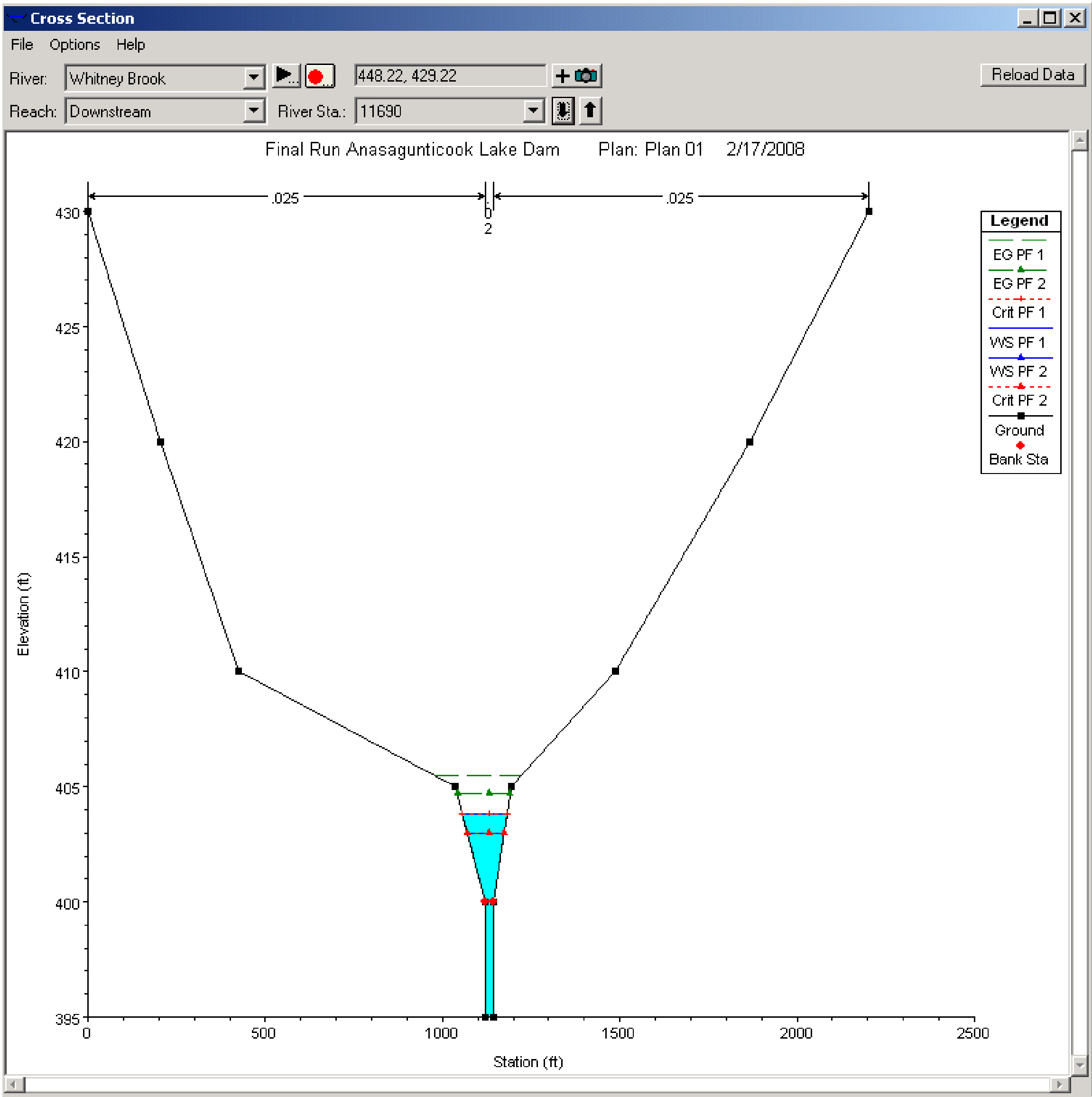
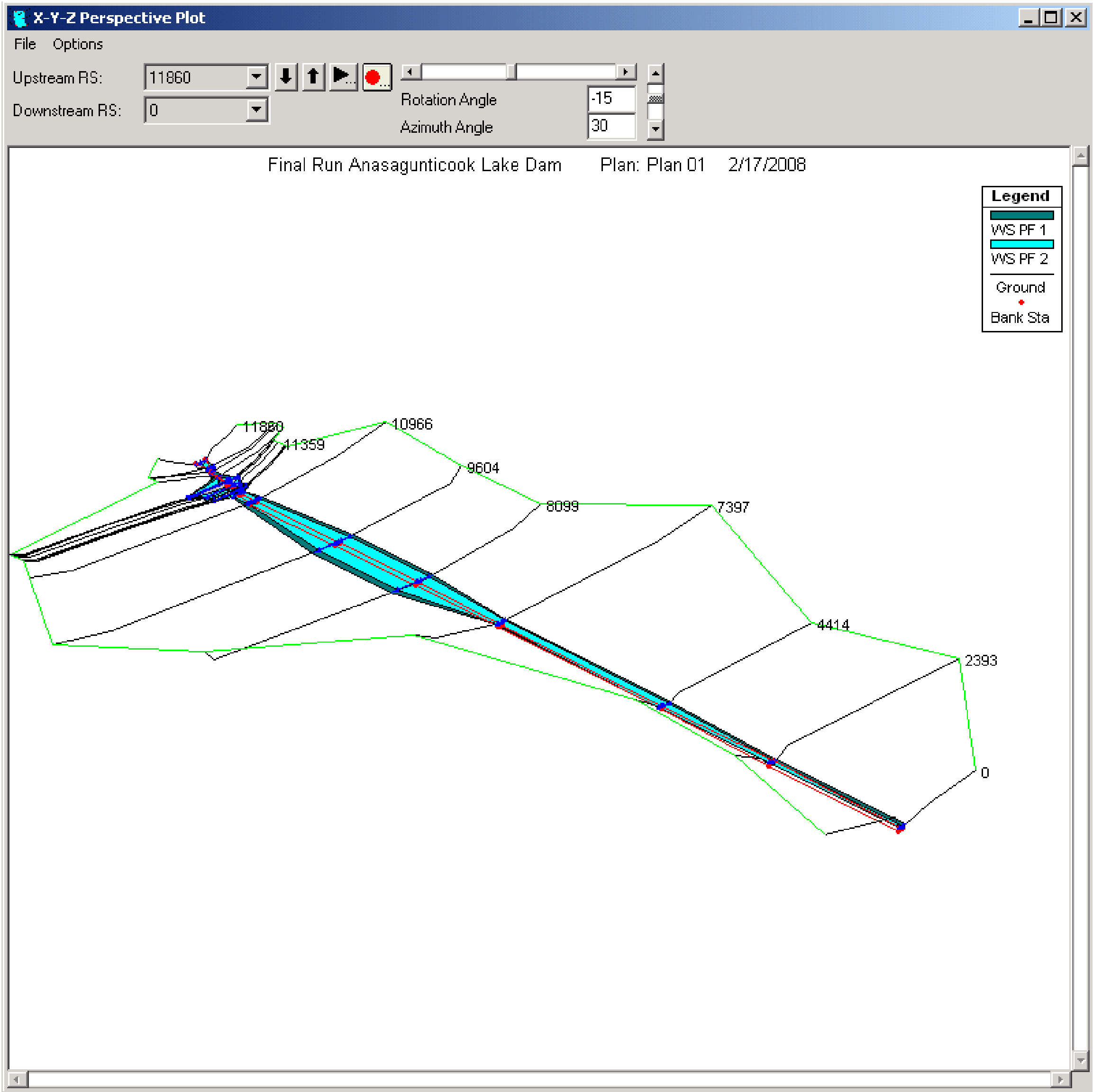
Appendix C – HEC-RAS output data

Collapsible Dam

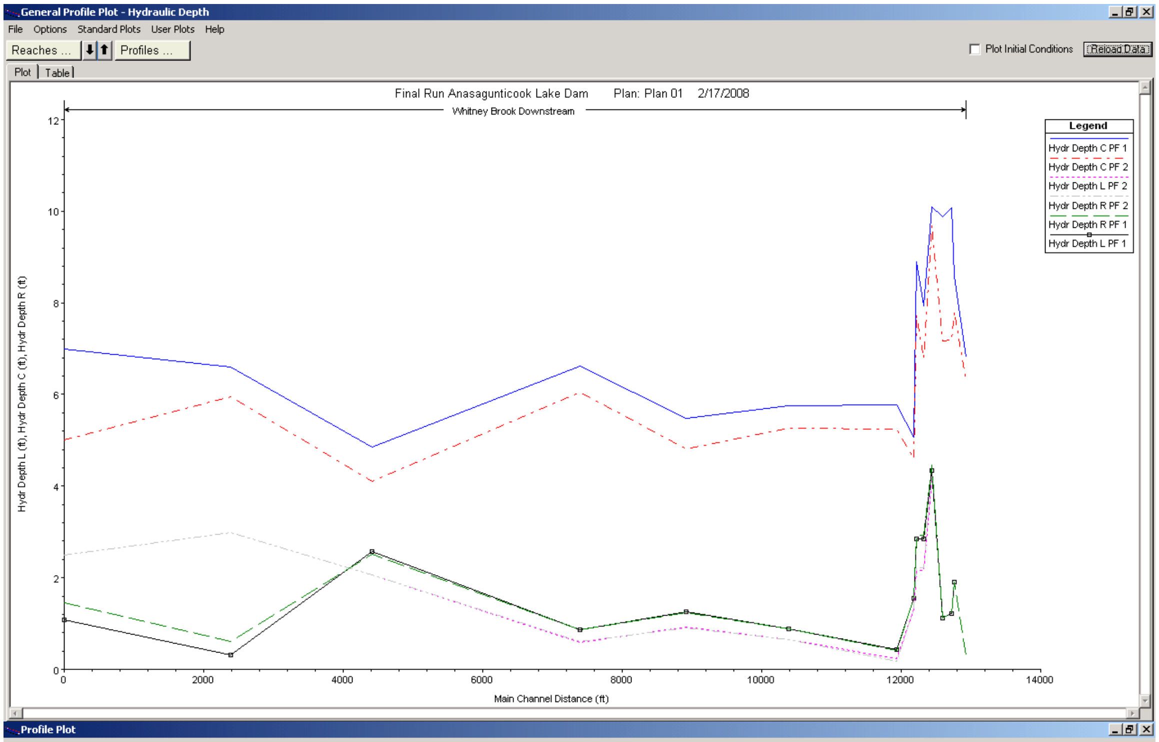
HEC-RAS Plan: Plan 01 River: Whitney Brook Reach: Downstream

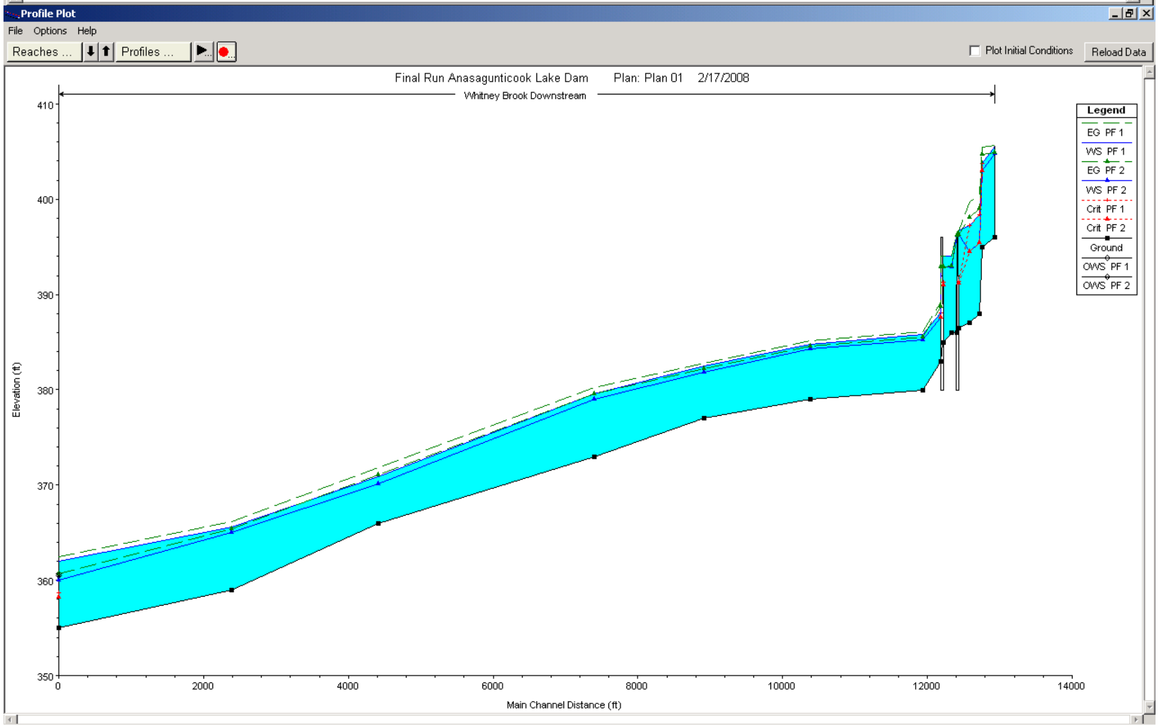
Reach	River Sta	Profile	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chl
			(cfs)	(ft)	(ft)	(ft)	(ft)	(ft/ft)	(ft/s)	(sq ft)	(ft)	
Downstream	11860	PF 1	3160.00	396.00	405.58		405.69	0.000098	2.63	1206.11	187.78	0.18
Downstream	11860	PF 2	2500.00	396.00	404.82		404.91	0.000086	2.33	1071.55	169.71	0.16
Downstream	11690	PF 1	3160.00	395.00	403.81	403.81	405.49	0.002160	11.82	392.06	125.09	0.71
Downstream	11690	PF 2	2500.00	395.00	403.02	403.02	404.71	0.002284	11.40	301.46	103.88	0.72
Downstream	11640	PF 1	3160.00	388.00	398.42	398.42	400.53	0.002362	12.24	330.22	104.22	0.68
Downstream	11640	PF 2	2500.00	388.00	395.49	395.49	399.09	0.005569	15.22	164.21	22.87	1.00
Downstream	11510	PF 1	3160.00	387.00	397.23	397.23	399.66	0.002700	12.92	294.19	82.85	0.72
Downstream	11510	PF 2	2500.00	387.00	394.48	394.48	398.09	0.005581	15.24	164.09	22.87	1.00
Downstream	11399	PF 1	3160.00	386.50	396.68	391.43	396.69	0.000018	1.35	4244.84	905.11	0.08
Downstream	11399	PF 2	2500.00	386.50	396.26	391.19	396.27	0.000014	1.18	3876.28	868.96	0.07
Downstream	11379		Inl Struct									
Downstream	11359	PF 1	3160.00	386.00	394.03		394.08	0.000103	2.73	2171.23	674.49	0.17
Downstream	11359	PF 2	2500.00	386.00	392.89		392.97	0.000179	3.24	1461.18	575.66	0.22
Downstream	11250	PF 1	3160.00	385.00	394.01	391.28	394.06	0.000095	2.75	2198.46	672.84	0.16
Downstream	11250	PF 2	2500.00	385.00	392.86	390.97	392.94	0.000159	3.24	1482.85	572.88	0.21
Downstream	11230		Inl Struct									
Downstream	11210	PF 1	3160.00	383.00	388.11	388.11	389.26	0.002326	10.16	481.20	217.02	0.80
Downstream	11210	PF 2	2500.00	383.00	387.64	387.64	388.74	0.002371	9.62	386.11	190.61	0.79
Downstream	10966	PF 1	3160.00	380.00	385.78		386.14	0.000414	4.87	733.68	347.29	0.36
Downstream	10966	PF 2	2500.00	380.00	385.25		385.54	0.000370	4.32	592.72	188.03	0.33
Downstream	9604	PF 1	3160.00	379.00	384.75		385.10	0.001381	5.06	1092.09	701.98	0.37
Downstream	9604	PF 2	2500.00	379.00	384.27		384.58	0.001292	4.62	794.26	536.02	0.35
Downstream	8099	PF 1	3160.00	377.00	382.48		382.83	0.001673	5.40	1259.10	727.40	0.41
Downstream	8099	PF 2	2500.00	377.00	381.80		382.20	0.002023	5.43	826.08	552.01	0.44
Downstream	7397	PF 1	3160.00	373.00	379.61		380.23	0.001831	6.40	585.13	191.88	0.44
Downstream	7397	PF 2	2500.00	373.00	379.05		379.54	0.001578	5.60	489.43	151.47	0.40
Downstream	4414	PF 1	3160.00	366.00	370.86		371.78	0.006476	9.80	656.50	221.82	0.78
Downstream	4414	PF 2	2500.00	366.00	370.10		371.10	0.008296	9.90	495.16	201.54	0.86
Downstream	2393	PF 1	3160.00	359.00	365.63		366.17	0.001700	5.91	546.18	109.31	0.41
Downstream	2393	PF 2	2500.00	359.00	364.99		365.41	0.001497	5.18	485.23	82.00	0.37
Downstream	0	PF 1	3160.00	355.00	362.00	358.73	362.51	0.001387	5.79	599.60	130.60	0.39
Downstream	0	PF 2	2500.00	355.00	360.00	358.19	360.65	0.002730	6.49	390.00	79.00	0.51


Cross Section
File Options Help
River: Whitney Brook
448.22, 429.22
Reload Data
Reach: Downstream
River Sta.: 11690
Final Run Anasagunticook Lake Dam Plan: Plan 01 2/17/2008
.025
.025
Legend
EG PF 1
EG PF 2
Crit PF 1
WS PF 1
WS PF 2
Crit PF 2
Ground
Bank Sta
Elevation (ft)
Station (ft)



X-Y-Z Perspective Plot
File
Options
Upstream RS:
11860
Downstream RS:
0
Rotation Angle
-15
Azimuth Angle
30
Final Run Anasagunticook Lake Dam Plan: Plan 01 2/17/2008
Legend
WS PF 1
WS PF 2
Ground
Bank Sta
11860
11359
10966
9604
8099
7397
4414
2393
0

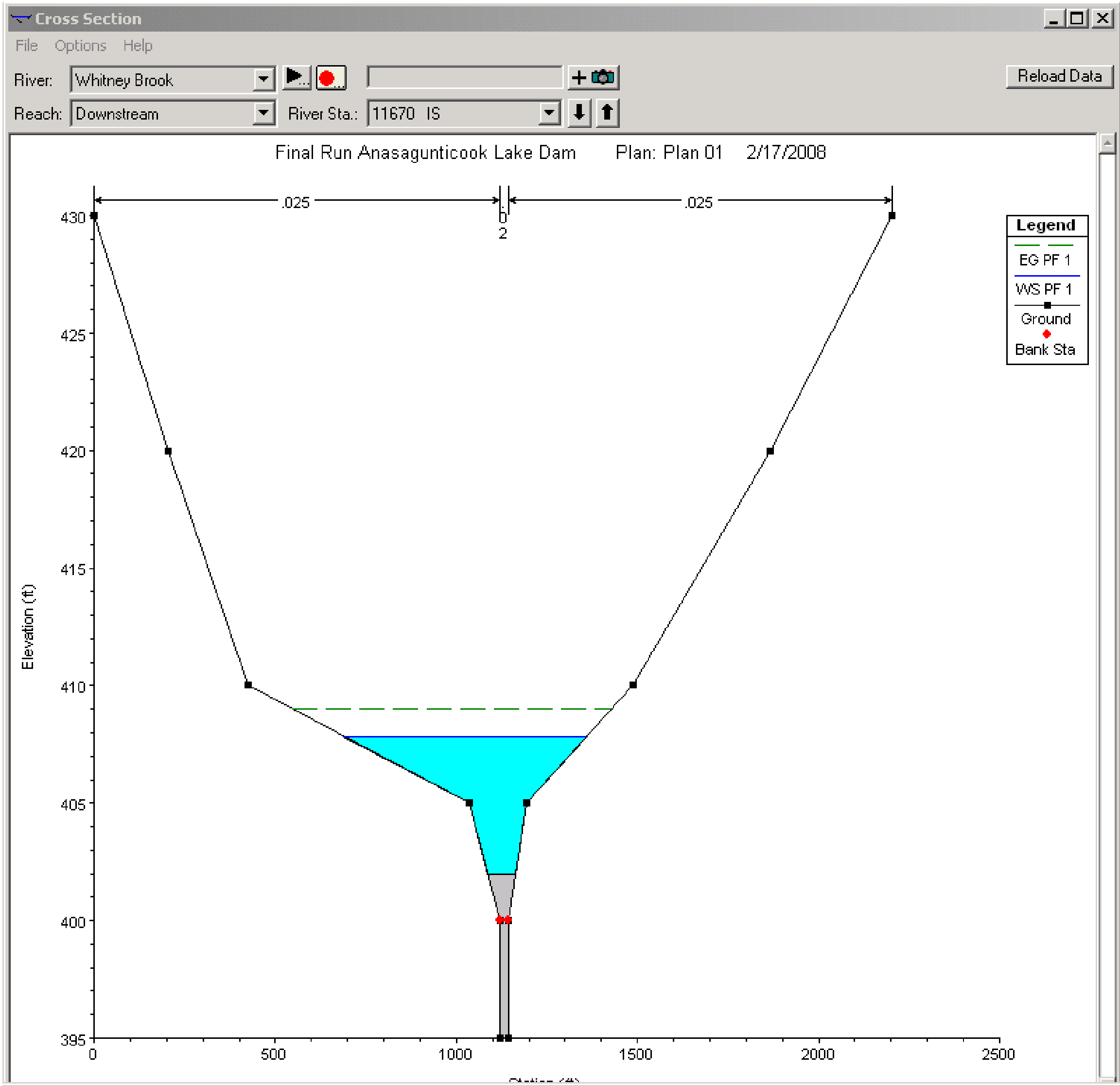

General Profile Plot - Hydraulic Depth
Final Run Anasagunticook Lake Dam Plan: Plan 01 2/17/2008
Whitney Brook Downstream
Legend
Hydr Depth C PF 1
Hydr Depth C PF 2
Hydr Depth L PF 2
Hydr Depth R PF 2
Hydr Depth R PF 1
Hydr Depth L PF 1
Hydr Depth L (ft), Hydr Depth C (ft), Hydr Depth R (ft)
Main Channel Distance (ft)

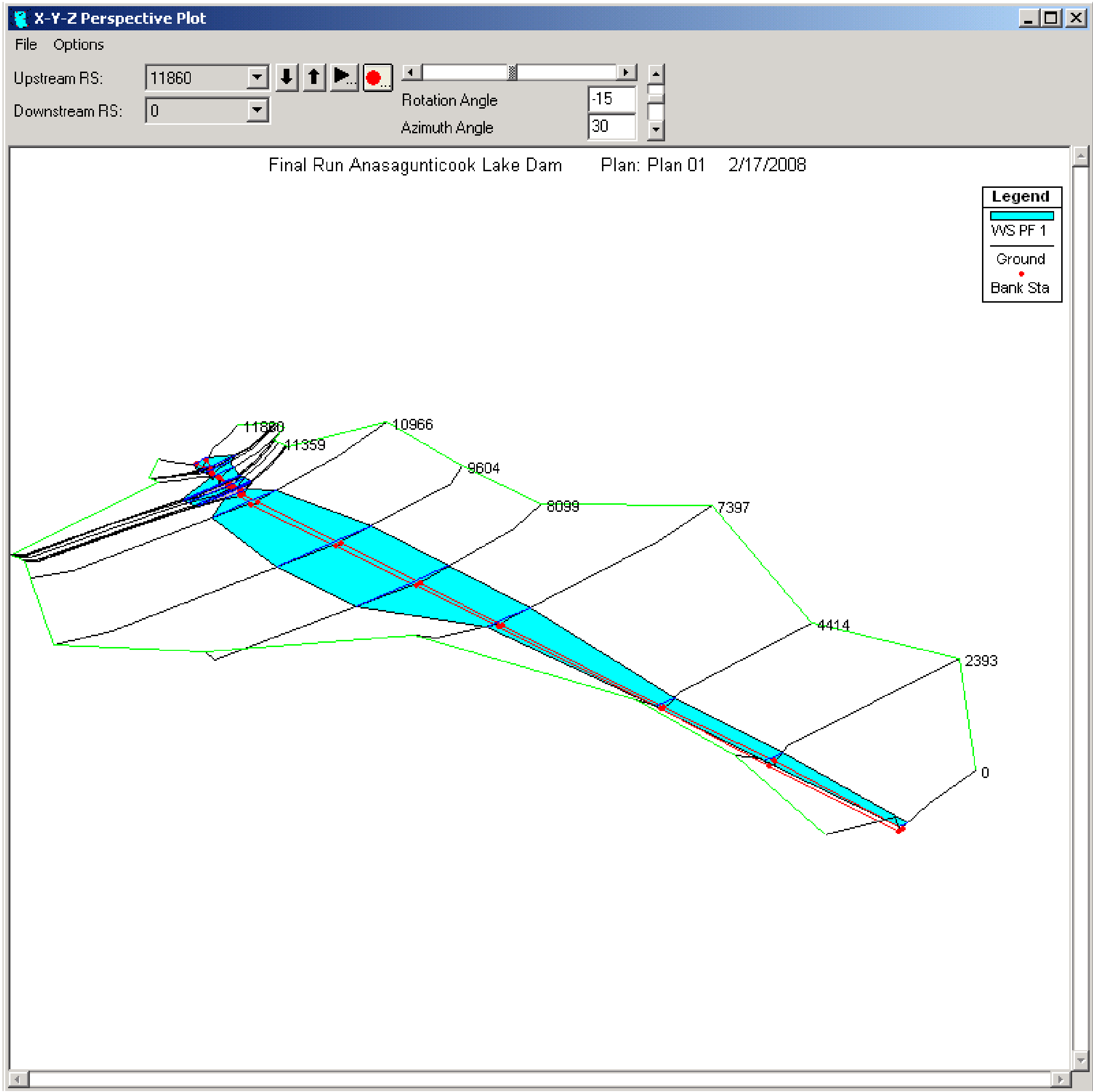

Profile Plot
Final Run Anasagunticook Lake Dam Plan: Plan 01 2/17/2008
Whitney Brook Downstream
Legend
EG PF 1
WS PF 1
EG PF 2
WS PF 2
Crit PF 1
Crit PF 2
Ground
OWS PF 1
OWS PF 2
Elevation (ft)
Main Channel Distance (ft)

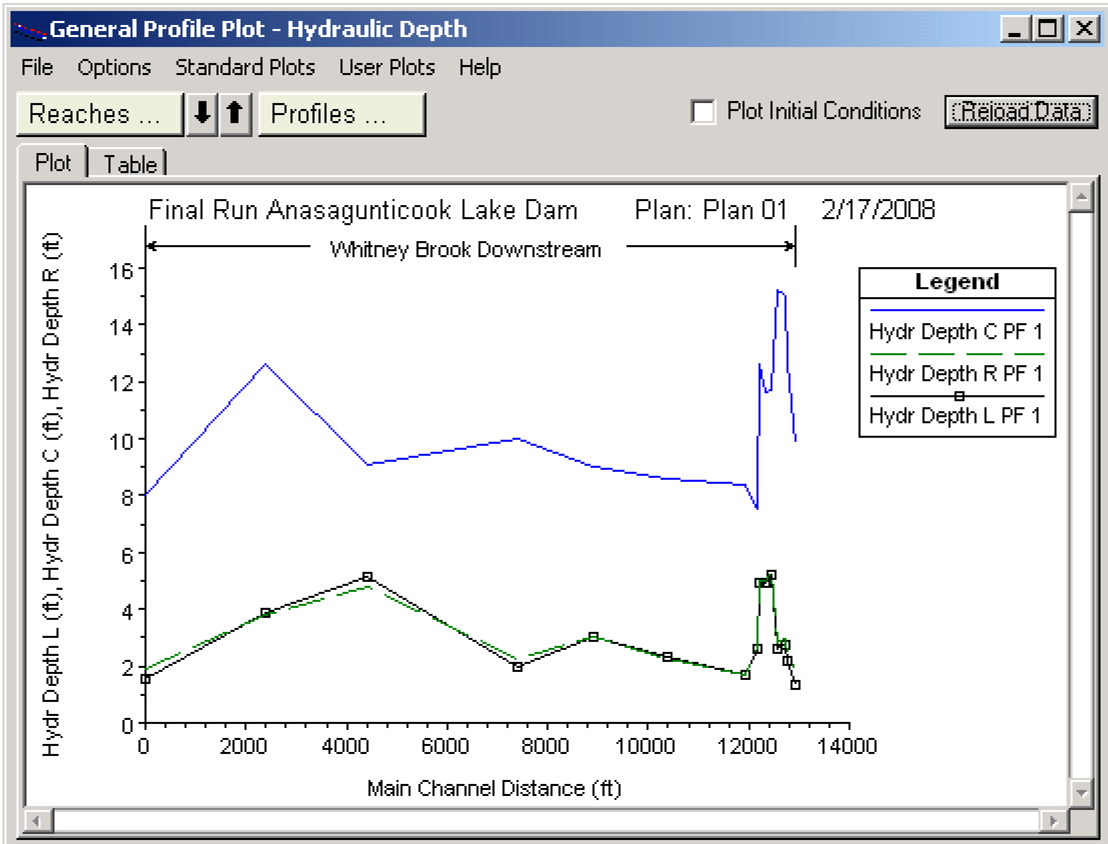
Gravity Dam- HEC-RAS Output 1/6 PMF and 500 Year Flood

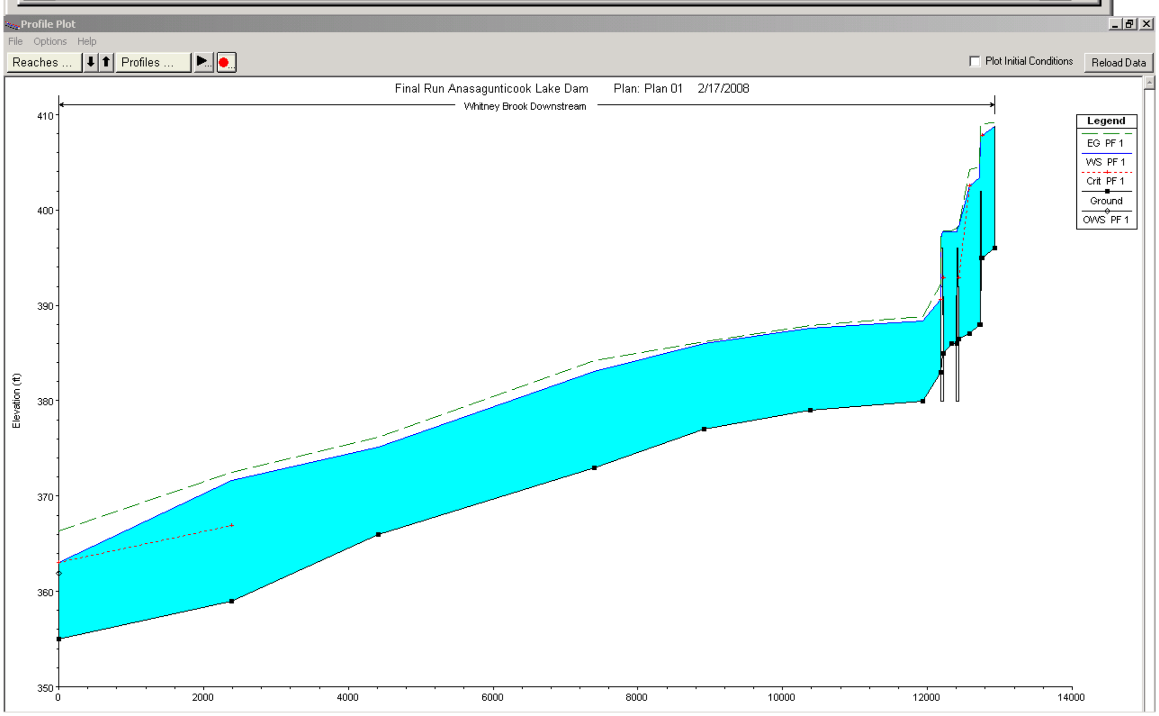
HEC-RAS Plan: Plan 01 River: Whitney Brook Reach: Downstream Profile: PF 1

Reach	River Sta	Profile	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chl
			(cfs)	(ft)	(ft)	(ft)	(ft)	(ft/ft)	(ft/s)	(sq ft)	(ft)	
Downstream	11860	PF 1	9500.00	396.00	408.75		409.16	0.000239	5.23	1941.40	276.75	0.29
Downstream	11690	PF 1	9500.00	395.00	407.84	407.84	409.01	0.001534	12.88	1738.10	671.30	0.64
Downstream	11670		Inl Struct									
Downstream	11640	PF 1	9500.00	388.00	403.36		404.52	0.001349	12.07	1573.51	460.68	0.55
Downstream	11510	PF 1	9500.00	387.00	402.57	402.57	404.25	0.001736	13.82	1328.24	389.38	0.62
Downstream	11399	PF 1	9500.00	386.50	398.30	392.88	398.35	0.000070	2.95	5825.39	1046.06	0.15
Downstream	11379		Inl Struct									
Downstream	11359	PF 1	9500.00	386.00	397.73		397.80	0.000090	3.30	5268.47	996.92	0.17
Downstream	11250	PF 1	9500.00	385.00	397.72	392.89	397.78	0.000088	3.34	5291.29	995.42	0.17
Downstream	11230		Inl Struct									
Downstream	11210	PF 1	9500.00	383.00	390.61	390.61	392.26	0.002556	13.92	1207.44	377.33	0.89
Downstream	10966	PF 1	9500.00	380.00	388.38		388.87	0.000490	6.79	2662.25	1134.46	0.41
Downstream	9604	PF 1	9500.00	379.00	387.57		387.93	0.001293	6.39	4447.75	1675.65	0.38
Downstream	8099	PF 1	9500.00	377.00	386.01		386.22	0.000889	5.48	5437.27	1641.41	0.32
Downstream	7397	PF 1	9500.00	373.00	383.04		384.24	0.002545	9.97	2187.97	780.88	0.55
Downstream	4414	PF 1	9500.00	366.00	375.09		376.21	0.004247	12.05	1832.43	334.31	0.70
Downstream	2393	PF 1	9500.00	359.00	371.68	366.89	372.49	0.001251	7.82	1962.93	326.02	0.39
Downstream	0	PF 1	9500.00	355.00	363.06	363.06	366.39	0.007598	14.87	751.85	157.84	0.92


Cross Section
File Options Help
River: Whitney Brook
Reach: Downstream
River Sta.: 11670 IS
Reload Data
Final Run Anasagunticook Lake Dam Plan: Plan 01 2/17/2008
.025
.025
Legend
EG PF 1
WS PF 1
Ground
Bank Sta
Elevation (ft)
430
425
420
415
410
405
400
395
0
500
1000
1500
2000
2500


X-Y-Z Perspective Plot
File Options
Upstream RS: 11860
Downstream RS: 0
Rotation Angle -15
Azimuth Angle 30
Final Run Anasagunticook Lake Dam Plan: Plan 01 2/17/2008
Legend
WS PF 1
Ground
Bank Sta
11860
11359
10966
9604
8099
7397
4414
2393
0

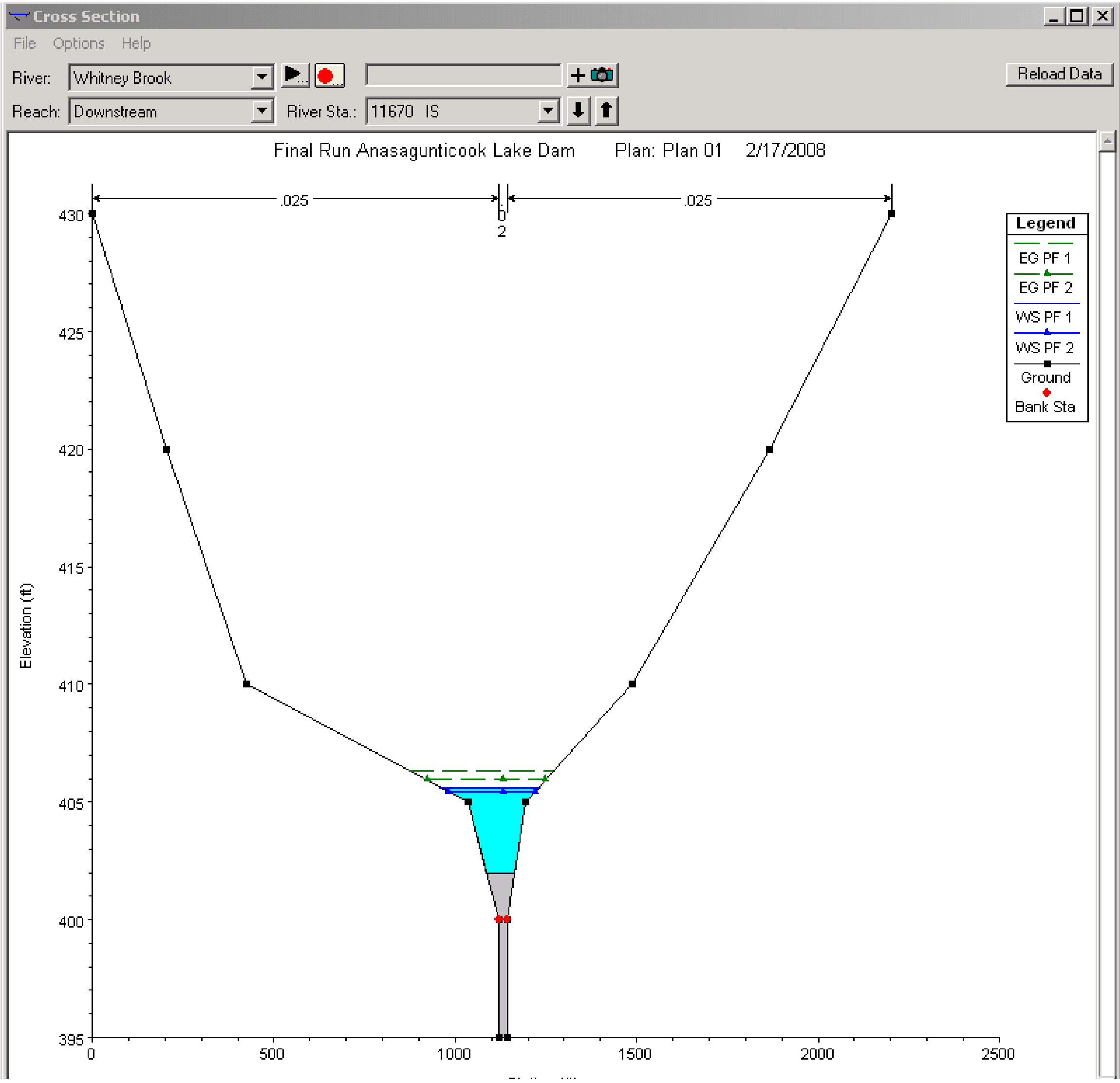

General Profile Plot - Hydraulic Depth
File
Options
Standard Plots
User Plots
Help
Reaches ...
Profiles ...
Plot Initial Conditions
Reload Data
Plot
Table
Final Run Anasagunticook Lake Dam Plan: Plan 01 2/17/2008
Whitney Brook Downstream
Legend
Hydr Depth C PF 1
Hydr Depth R PF 1
Hydr Depth L PF 1
Hydr Depth L (ft), Hydr Depth C (ft), Hydr Depth R (ft)
0
2
4
6
8
10
12
14
16
0
2000
4000
6000
8000
10000
12000
14000
Main Channel Distance (ft)

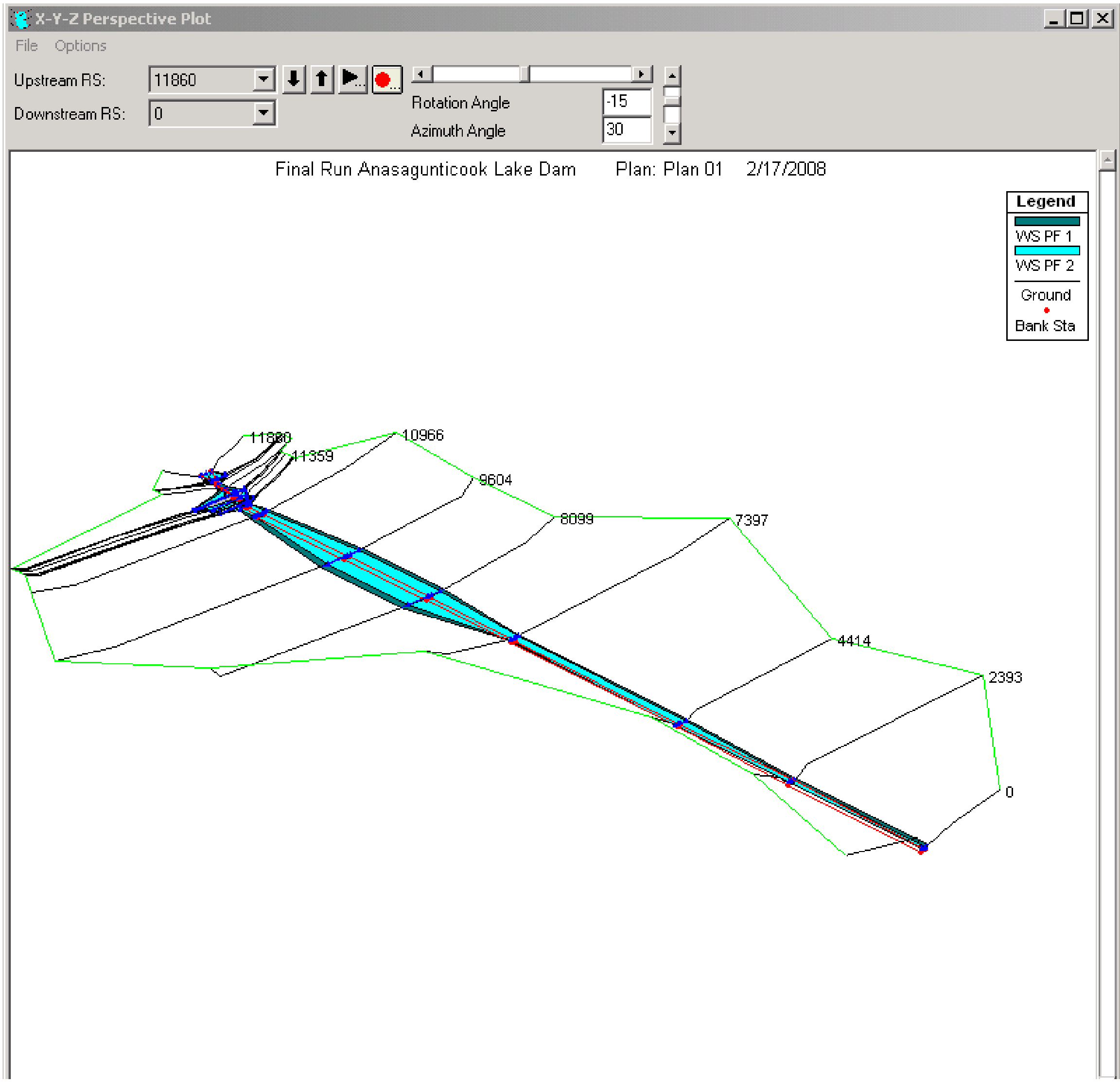

Profile Plot
File Options Help
Reaches ...
Profiles ...
Plot Initial Conditions
Reload Data
Final Run Anasagunticook Lake Dam Plan: Plan 01 2/17/2008
Whitney Brook Downstream
Legend
EG PF 1
WS PF 1
Crit PF 1
Ground
OWS PF 1
Elevation (ft)
350
360
370
380
390
400
410
0
2000
4000
6000
8000
10000
12000
14000

Gravity Dam- HEC-RAS Output 1/2 PMF

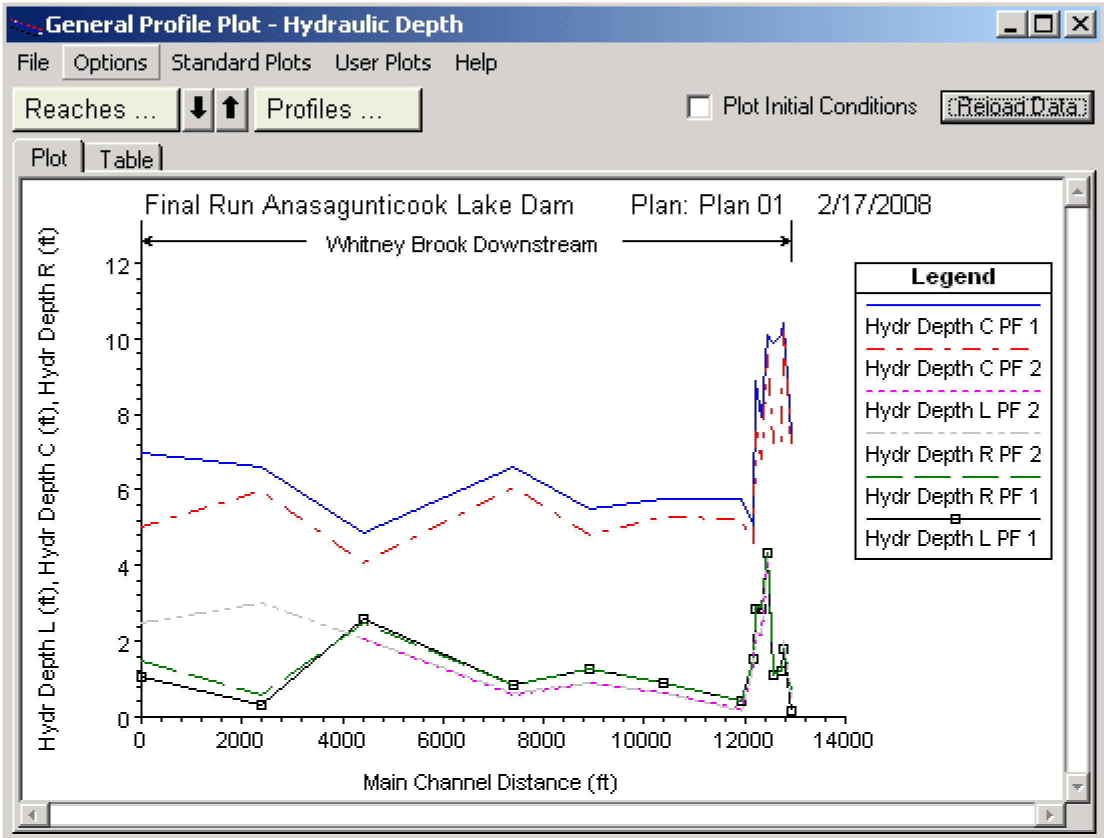
HEC-RAS Plan: Plan 01 River: Whitney Brook Reach: Downstream

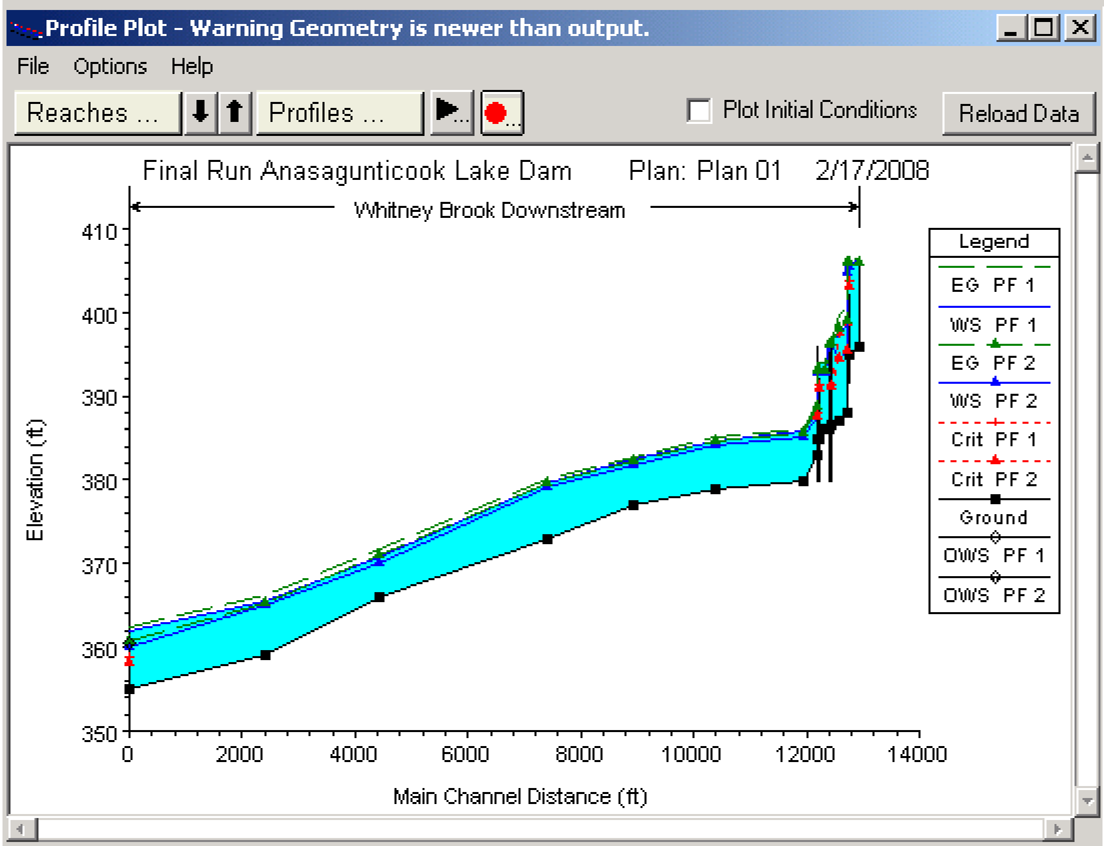
Reach	River Sta	Profile	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	F
			(cfs)	(ft)	(ft)	(ft)	(ft)	(ft/ft)	(ft/s)	
Downstream	11860	PF 1	3160.00	396.00	406.35		406.44	0.000070	2.35	
Downstream	11860	PF 2	2500.00	396.00	405.95		406.01	0.000053	1.97	
Downstream	11690	PF 1	3160.00	395.00	405.63	403.81	406.35	0.000826	8.31	
Downstream	11690	PF 2	2500.00	395.00	405.46	403.02	405.94	0.000562	6.78	
Downstream	11670		Inl Struct							
Downstream	11640	PF 1	3160.00	388.00	398.42	398.42	400.53	0.002362	12.24	
Downstream	11640	PF 2	2500.00	388.00	395.49	395.49	399.09	0.005569	15.22	
Downstream	11510	PF 1	3160.00	387.00	397.23	397.23	399.66	0.002700	12.92	
Downstream	11510	PF 2	2500.00	387.00	394.48	394.48	398.09	0.005581	15.24	
Downstream	11399	PF 1	3160.00	386.50	396.68	391.43	396.69	0.000018	1.35	
Downstream	11399	PF 2	2500.00	386.50	396.26	391.19	396.27	0.000014	1.18	
Downstream	11379		Inl Struct							
Downstream	11359	PF 1	3160.00	386.00	394.03		394.08	0.000103	2.73	
Downstream	11359	PF 2	2500.00	386.00	392.89		392.97	0.000179	3.24	
Downstream	11250	PF 1	3160.00	385.00	394.01	391.28	394.06	0.000095	2.75	
Downstream	11250	PF 2	2500.00	385.00	392.86	390.97	392.94	0.000159	3.24	
Downstream	11230		Inl Struct							
Downstream	11210	PF 1	3160.00	383.00	388.11	388.11	389.26	0.002326	10.16	
Downstream	11210	PF 2	2500.00	383.00	387.64	387.64	388.74	0.002371	9.62	
Downstream	10966	PF 1	3160.00	380.00	385.78		386.14	0.000414	4.87	
Downstream	10966	PF 2	2500.00	380.00	385.25		385.54	0.000370	4.32	
Downstream	9604	PF 1	3160.00	379.00	384.75		385.10	0.001381	5.06	
Downstream	9604	PF 2	2500.00	379.00	384.27		384.58	0.001292	4.62	
Downstream	8099	PF 1	3160.00	377.00	382.48		382.83	0.001673	5.40	
Downstream	8099	PF 2	2500.00	377.00	381.80		382.20	0.002023	5.43	
Downstream	7397	PF 1	3160.00	373.00	379.61		380.23	0.001831	6.40	
Downstream	7397	PF 2	2500.00	373.00	379.05		379.54	0.001578	5.60	
Downstream	4414	PF 1	3160.00	366.00	370.86		371.78	0.006476	9.80	
Downstream	4414	PF 2	2500.00	366.00	370.10		371.10	0.008296	9.90	
Downstream	2393	PF 1	3160.00	359.00	365.63		366.17	0.001700	5.91	
Downstream	2393	PF 2	2500.00	359.00	364.99		365.41	0.001497	5.18	
Downstream	0	PF 1	3160.00	355.00	362.00	358.73	362.51	0.001387	5.79	
Downstream	0	PF 2	2500.00	355.00	360.00	358.19	360.65	0.002730	6.49	


Cross Section
File Options Help
River: Whitney Brook
Reload Data
Reach: Downstream
River Sta.: 11670 IS
Final Run Anasagunticook Lake Dam Plan: Plan 01 2/17/2008
.025
.025
0
2
Legend
EG PF 1
EG PF 2
WS PF 1
WS PF 2
Ground
Bank Sta
Elevation (ft)
430
425
420
415
410
405
400
395
0
500
1000
1500
2000
2500



X-Y-Z Perspective Plot
File Options
Upstream RS: 11860
Downstream RS: 0
Rotation Angle -15
Azimuth Angle 30
Final Run Anasagunticook Lake Dam Plan: Plan 01 2/17/2008
Legend
WS PF 1
WS PF 2
Ground
Bank Sta
11860
11359
10966
9604
8099
7397
4414
2393
0


General Profile Plot - Hydraulic Depth
File Options Standard Plots User Plots Help
Reaches ... Profiles ...
Plot Initial Conditions
Reload Data
Plot Table
Final Run Anasagunticook Lake Dam Plan: Plan 01 2/17/2008
Whitney Brook Downstream
Hydr Depth L (ft), Hydr Depth C (ft), Hydr Depth R (ft)
Main Channel Distance (ft)
Legend
Hydr Depth C PF 1
Hydr Depth C PF 2
Hydr Depth L PF 2
Hydr Depth R PF 2
Hydr Depth R PF 1
Hydr Depth L PF 1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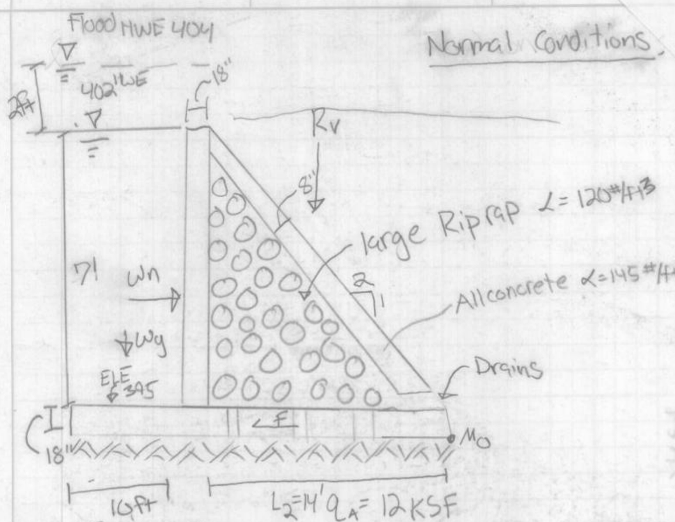
File Options Help
Reaches ... Profiles ...
Plot Initial Conditions
Reload Data
Final Run Anasagunticook Lake Dam Plan: Plan 01 2/17/2008
Whitney Brook Downstream
Elevation (ft)
Main Channel Distance (ft)
Legend
EG PF 1
WS PF 1
EG PF 2
WS PF 2
Crit PF 1
Crit PF 2
Ground
OWS PF 1
OWS PF 2

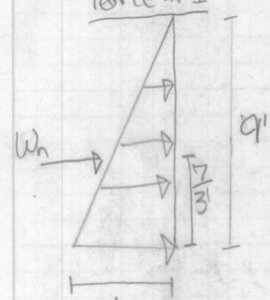
Appendix D – Design calculations

ROCK FILL GRAVITY DAM

Normal Conditions



Force #1



$L = \alpha * h$
$= 62.4*9$

Force 2

$W_y = L_1 * \alpha * h$
$= L_1 * 62.4*7$
$W_y = 436.8\ L_1$ #ft

23260
22024.6

$W_h = \frac{1}{2} b \cdot h = \frac{1}{2}(62.4*9)(9)$
$= 1530$ #/ft

$R_v = \frac{1}{2}(b*h)*\alpha = \frac{1}{2}(7*14)\ 120$ #/ft2

$R_v = 5880$ #/ft

$R_{VR} = \rho * L * H = 62.4$ #/ft3 * 10ft * 9ft
$= 5616$ #/ft

$P_{vc} = (24ft)(18")(1ft)(145\#/ft^3)$

$= 5220\#/ft$

$R_v = \frac{1}{2}(14ft * 7ft * 120\#/ft^3)$

$= 5880\#/ft$

$P_{cr} = (7ft * 18in) * 145\#/ft^3 + (\sqrt{14^2+7^2})(18")(145\#/ft^3)$

$= 4026\#/ft$

$P_U = \frac{1}{2} \gamma * H * L$

$= \frac{1}{2} 62.4\#/ft^3 * 9ft * 24ft$

$= 6739.2\#/ft$

No uplift pressures, drainage system will be installed

Earthquake forces:

$P_e = C \lambda w h$

$$C = \frac{C_m}{2}\left[\frac{y}{h}\left(2-\frac{y}{h}\right) + \sqrt{\frac{y}{h}\left(2-\frac{y}{h}\right)}\right]$$

λ = Eq intensity = $\frac{\text{earthquake acceleration}}{\text{acceleration of gravity}}$ = average value .1g

$= \frac{.1g}{1g} = .1 = \lambda$

$w = \gamma = 62.4 \#/ft^3$

h = resevoir depht $y = 9ft$

y = verticle distance from WSE to area studied $y = 9ft$

C_m = value from chart p.237 design small dams f-164

$= .7$

$$C = \frac{.7}{2}\left[\frac{9'}{9'}\left(2-\frac{9'}{9'}\right) + \sqrt{\frac{9'}{9'}\left(2-\frac{9'}{9'}\right)}\right] = .7$$

$\lambda = \frac{.1g}{1g} = .1$

$\therefore P_e = .7 * .1 * 62.4\#/ft^3 * 9ft$

$= 34.3 \#/ft^2$

$V_e = .726\ P_e * y = .726 * 39.3\#/ft^2 * 3.5ft$

$= 99.86\#/ft$

$I = .5K/ft$

Overturning:

$\Sigma M = (P_h * 3.5ft) + (P_{vr} * 14ft) + (R_v * 7ft) + (R_{vc} * 12ft)$
$+ (P_{cr} * 7ft) - (P_v * 16ft) - (I * 7ft) - (E * 3.5ft)$

$\Sigma M_r = 244.986 ft\text{-}K$

$\Sigma M_o = 117.025 ft\text{-}K$

$\frac{\Sigma M_R}{2} \geq \Sigma M_o \Rightarrow \frac{244.986^{ft\text{-}K}}{2} > 117.025\ ft\text{-}K$

$122.493 ft\text{-}K \geq 117.025 ft\text{-}K$ ok

w/ $Sf = 2.09$

Sliding:

$\Sigma f_x = 0 = W_h + 1.5 - (N_f * \mu_v)$

$\Sigma W_h = 1530^{\#} + 500^{\#} + 99\#$
$= 2129\#$

$\Sigma N_f = 5616 + 5880 + 5220$
$+ 4926 - 6739$
$= 14903\#$

$N_f * \mu_v = 14903\# * .3$
$= 4470\#$

$$\frac{\Sigma F_{rest}}{1.5} \geq \Sigma F_{sliding} \Rightarrow \frac{4470\#}{1.5} \geq 2192\#$$

$$2980.6\# \geq 2192\#$$

so ok w/ s.f. of 2.1

Check Eccentricity:

$$\Sigma M_b = \Sigma M_O + \Sigma F_y * x + \Sigma M_R = 0$$

$$= 117.025 \text{ ft-K} + 14903\# * x - 244986 \text{ ft-K}$$

$$= 8.58 \text{ ft}$$

$$e = \frac{B}{2} - x = \frac{24 \text{ ft}}{2} - 8.58 \text{ ft}$$

$$= 3.42 \text{ ft}$$

$$\frac{B}{6} = \frac{24 \text{ ft}}{6} = 4 \text{ ft}$$

$e < \frac{B}{6}$ so ok moment is in middle third

Bearing Check:

$$B' = B - 2e = 24 \text{ ft} - 2(3.42 \text{ ft})$$

$$= 17.16 \text{ ft}$$

$$q_{equ} = \frac{P + W_F}{B' * L'} = \frac{21642\#/\text{ft}}{17.16 \text{ ft} * 1 \text{ ft}} = 1261\#/\text{ft}^2 < 5 \text{ K}/\text{ft}^2 \text{ ok}$$

(USACE EM 1110-1-2908)

Load Combinations:

$P_u = 1.4\,H = 1.4 * 1530 = 2142\#/ft$

$P_u = 1.4H + 1.6\,I = 1.4 * 1530 + 1.6 * 500 = 2942\ \#/ft$

$P_u = 1.4H + 1.0E = 1.4 * 1530 + 99 * 1.0 = 2231\ \#/ft$

$\therefore$ Use $2942\#/ft$

Max Moment:

$\Sigma M_o = M_{umax} * P_u * L = 0 = M_{umax} - (2.942K * 7ft)$

$M_{umax} = 20.59ft\text{-}K$

Shear max:

$\Sigma F_y = 0 = V_{max} - P$

$V_{max} = 2.942K$

Stem thickness:

Since no Stirups

Shear Capacity

$\frac{V_n}{\phi} = 2 b_w d \sqrt{f'c} = 2(12in)\,d\sqrt{3000psi} = 2942\#$

$d = 2.13in$

Use d min 15in (ACI)

$T \doteq d + d_b + \text{cover} = 15\text{in} + .5 + 3\text{in}$

$= 18.5''$

Check Shear Transfer to Footing:

$$\frac{A_{vf}}{b} = \frac{\frac{V_u}{b}}{\phi f_y \mu} = \frac{2942\,\text{lb/ft}}{.85(40000)(.6)} = 0.144\ \text{in}^2/\text{ft}$$

add to Flex steel

Flex Stem Steel:

$$\frac{A_s}{b} = \left(\frac{f'c\,b}{1.176 f_y}\right)\left(d - \sqrt{d^2 - \frac{2.353\frac{M_u}{b}}{\phi f'c\,b}}\right)$$

$$= \left(\frac{3000 * 12}{1.176(40000)}\right)\left(15'' - \sqrt{15^2 - \frac{2.353 * 20.59\,\text{ft-k} * 12 * 1000}{.9(40{,}000)(12)}}\right)$$

$= .516\text{in}^2/\text{ft}$

Check Steel Ratios

$$\rho = \frac{A_s}{bd} = \frac{.516\text{in}^2/\text{ft}}{(15\text{in})(12)} = .003869 \leq (\rho_b * .35 = .0173)$$

ok

$$A_{s\,min} \geq 0.0015 t_g = 0.0015(18.5\text{in}*)(12\text{in})$$

$$= 0.33\text{in}^2 < .516\text{in}^2/\text{ft}$$

ok

Add total steel A

$$A_f + A_v = .516 in^2 + .144 in^2$$
$$= .66 in^2/ft$$

choose rebar

as #6 bar w/ 7 1/2" spacing = .70 in²/ft

Check ϕM_n Moment cap.

$$a = \frac{\rho d f_y}{0.85 f'c} = \frac{.003864 * 15 * 40000}{.85 * 3000}$$

$$= 0.91''$$

$$\phi M_n = \phi A_s f_y (d - \frac{a}{2}) = .9 * 40000 (15 - \frac{.91}{2}) * .66 in^2$$

$$= 28.79 \text{ k-ft} > \text{actual} = 20.59 \text{ ft-k}$$

OK

Footing Design:

Required Development length

$$L_{dh} = \frac{1200\ d_b}{\sqrt{f'c}} = \frac{1200 * \frac{3}{4}}{\sqrt{3000}}$$

$$= 16.5 in$$

T min

$$T \geq l_{dh} + 3'' = 16.5 in + 3 in$$

$$= 19.5 in \therefore \text{use } 20 in$$

Heal Extension Shear

$$\phi \frac{V_n}{b} = 2 b_w d \sqrt{f'_c} = 2 * 12 d * \sqrt{3000 in^2}$$

$$d_{req} = 20.5 \text{ inches} > T_{min}$$

$$\therefore T_{actual} \rightarrow d_{req} = d + \frac{d_b}{} + 3in = 20.5in + .5in + 3in$$

$$= 24 \text{ inches}$$

Flexual Design

$$\frac{M_u}{b} = 1.4 * (24911 \#/ft)\left(\frac{24ft}{2}\right)(12in/ft)$$

$$= 6{,}030{,}058 \text{ in-}\#$$

$$\frac{A_s}{b} = \left(\frac{f'_c b}{1.176 f_y}\right)\left(d - \sqrt{\frac{2.353 M_u/b}{\phi f'_c * b}}\right)$$

$$= \frac{(3000*12)}{1.176*40000} * \left(20.5 - \sqrt{\frac{2{,}353(6{,}030{,}058)}{(.9*3000*12)}}\right)$$

$$= 3.261 in^2/ft$$

$$\text{Min } A_s/b = 0.0018 A_g = .0018 * 24 * 12in$$

$$= .5184 in^2 < 3.26 in^2/ft$$

so use $3.261 in^2/ft$

$\therefore$ use 6 #17 bar w/ $A = 3.60 in^2$

Check Temp Steel

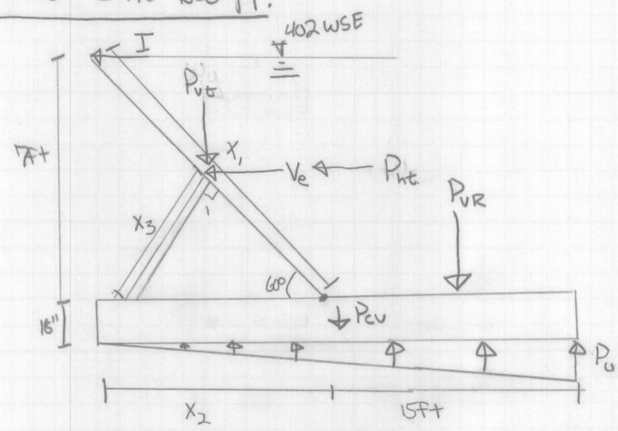
$A_s = 0.0018 A_g = .0018 * 24 * 24 * 12$

$= 5.386 \text{ in}^2/\text{ft}$

$\therefore$ use 6 #8 bars w/ $A = 5.53 \text{ in}^2$
per foot

CREST GATE DESIGN

New Crest Gate Design:



X_1: $\sin\theta = \frac{O}{H}$

$$X_1 = \frac{7'}{\sin 60°} = 8.1\text{ft}$$

X_2: $\tan\theta = \frac{O}{A}$

$$X_2 = \frac{7'}{\tan 60°} = 4.05\text{ft}$$

X_3: Choose 60in for cylinder

Use 1ft Hypothetical Cross section:

$$P_{Ht} = \frac{1}{2}(\gamma h L) = \frac{1}{2}(4.05\text{ft} * 7\text{ft} * 62.4\#/\text{ft}^3)$$

$$= 884.5\#/\text{ft}$$

$$P_{vt} = \frac{1}{2}(\gamma h^2) = \frac{1}{2}(7\text{ft}^2 * 62\#/\text{ft}^3)$$

$$= 3057.6\#/\text{ft}$$

Sum Moments

$$+\circlearrowright \Sigma M_0 = 0 = \left(P_{ht} * \frac{7}{3} ft\right) - \left(P_{vt} * 7.5 ft\right) - \left(P_{vh} * 7.5 ft\right) - \left(P_{vR} * 9.025\right)$$

$$+ \left(P_e * 3.5 ft\right) + \left(P_v * 5 ft\right) + \left(5K * 7 ft\right) + M_R$$

$M_R = 129{,}638$ ft-lbs

$M_O = 43{,}927$ ft-lbs

$$\frac{\Sigma M_{resisting}}{2} \geq \Sigma M_{overturning}$$

$$\frac{129{,}638 \text{ ft-lbs}}{2} \geq 43{,}927 \text{ ft-lbs}$$

$= 64{,}819$ ft-lbs $\geq 43{,}927$ ft-lbs

$\therefore$ ok w/ $\boxed{sf = 2.95}$

Sliding

$\Sigma W_h = 884.5\# + 37.14\# + 500\# = 1421.64\#$

$\Sigma N_f = 3057\# + 6552\# + 4286.25\# - 4160\# = 9735\#$

$\Sigma F_x = 0 = W_h * 1.5 - (N_f * \mu_v)$

$$\frac{\Sigma F_{rest}}{1.5} \geq \Sigma F_{sliding} \Rightarrow \frac{(9735\# * .3)}{1.5} \geq 1421.64\#$$

$1947\# \geq 1421\#$

ok

w/ $\boxed{sf = 2.05}$

Check eccentricity:

$$\Sigma M/_b = \Sigma M_o + \Sigma F_y * X + \Sigma M_R = 0$$

$$= 43927\text{ ft-lb} + (13895\# * X) - 129638\text{ ft-lbs}$$

$$= 6.16\text{ ft}$$

$$e = \frac{B}{2} - X = \frac{19.05\text{ ft}}{2} - 6.16$$

$$= 3.162\text{ ft}$$

$$\frac{B}{6} = \frac{19.05}{6} = 3.175\text{ ft}$$

$$\therefore e < B/6 \quad \text{OK}$$

Bearing check

$$B' = B - 2e = 19.05 - 2(3.162\text{ ft})$$

$$= 12.726$$

$$q_{equ} = \frac{P + W_F}{B' * L'} = \frac{13895\#/\text{ft}}{12.726\text{ ft} * 1\text{ ft}} = 1091.87\#/\text{ft}^2$$

$$\therefore 1091.87 < q_{All} = 5{,}000\#/\text{ft}^2$$ (USACE EM 1110-1-2908)

Load Combinations:

$P_u = 1.4H + 1.2D + 1.6C = (1.4 * 3194.5\#) = 5516\#$

$P_u = 1.2D + 1.4H + 1.6\,C{+}I{+}M = (1.4 * 3194.5\#) + 1.6(500) = 6318.1\#$

$P_u = 1.2(D) + 1.4H + 1.0E = (1.4 * 3194.5\#) + 1.0(37.14\#) = 5555\#$

$\therefore$ use $6318.1\#/ft$

Hydraulic cylinders:

$P_{max} = H_p \dfrac{D^2}{2}\pi = 5000 * \dfrac{5in}{2}\pi = 39.26K$

$D_{max} = \dfrac{P_{max}/b}{P_u} = \dfrac{39.26K/ft}{6.318K} = 6.21ft = 6ft$

Moment:

$\Sigma M_0 = 0 = M_{umax} + (P_u * L * D_{max})$

$M_{umax} = 6318.1\#/ft * 3.5ft * 6ft$

$= 132.68K\text{-}ft$

$F_b = .66F_y = .66 * 50000psi$

$= 33.3\ ksi$

$S_{req} = \dfrac{M_u}{F_b} = \dfrac{132.68K\text{-}ft * 12in/ft}{33.33KSI} = 47.81in^3$

$S_{req} = 47.81in^3 \geq S = 48.8in^3$ is a $W21 * 112\#/ft$

(AISC steel manual)

OK

Shear:

$$\Sigma F_y = 0 = V_{nmax} - P_u = V_{nmax} - (6.38K * 6ft)$$

$$V_{nmax} = 38.28K$$

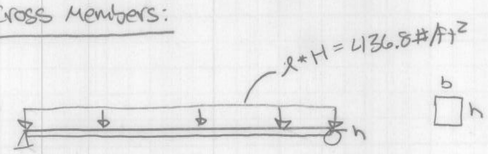
$$F_v = .4\ F_y = .4 * 50KSI$$
$$= 20KSI$$

$$A_{req} = \frac{V_u}{F_v} = \frac{38.28K}{20KSI} = 1.94in^2$$

$$A_{req} \geq A = 33in^2 \text{ for a } W21 * 112\#/ft$$

$\therefore$ OK

Wood Cross Members:



Assume b = 6in

$\therefore$ $P_d = \ell * h * b = 62.4\#/ft^3 * 7ft * 6in * \frac{1ft}{12in} = 218.4\#/ft$

For Simply Supported beam:

$$M_{max} = \frac{WL^2}{8} = \frac{.437K/ft * 6ft^2}{8} = 1.966ft\text{-}K$$

$$V_{max} = \frac{WL}{2} = \frac{.437 * 6ft}{2} = 1.31K$$

Moments:

$M' \geq M$

$M' = F'_b S$

$F'_b = F_b C_m C_d C_1 C_t C_{fu} C_i C_f C_r$

Use Douglas-fir #1

$F_b = 1200 PSI$

$F_v = 170\#/in^2$

$C_m = .85$ = M.C. > 19% (wet)

$C_d = .85$ = impact loading

$C_i = 1.0$ = Not insciced

$C_1 = 1.0$ = Not braced

$C_t = 1.0$ = No temp

$C_{fu} = 1.0$ = No flat use

$C_f = 1.3$ = Beam size

$C_r = 1.0$ = No repeating members

$F'_b = 1200 PSI * .85 * .85 * 1.3$
$= 1127.1 PSI$

$$S_{req} = \frac{M}{F'_b} = \frac{1.966 Ft\text{-}K * 12 in/Ft}{1127.1 psi} = 20.93 in^3$$

Use 6"x6" Beam w/S = 27.73in³ > 20.93in³ ok

Shear:

$$V' \geq V$$

$$V' = \frac{2}{3} F_v' A$$

$$F_v' = F_v C_D C_M C_t C_i$$

$$= 170\,psi * .85 * .85 * 1.0 * 1.0$$

$$= 122.82\,PSI$$

$$A_{req} = \frac{V}{\frac{2}{3} F'_v} = \frac{1.31K}{\frac{2}{3} * .122KSI}$$

$$= 16.1\,in^2 < A = 30.25 \text{ for } 6'' * 6'' \text{ Standard beam}$$

$\therefore$ OK

Footing Design:

Heal extension Shear:

$\frac{V_n}{b} = (P_{ft} + V_e + I) = 15316.84\#/ft$

$\frac{V_u}{b} = V * 1.4 = 21.44K/ft$

$\phi \frac{V_n}{b} = 2 b_w d \sqrt{f'_c} = 2 * 12in\ d \sqrt{3000\#/in^2}$

$d_{req} = 14.19\ in \geq 18in$ (used in design)

$T > d = d_b/2 + 3 = 19.19in + .5in + 3in$

$= 22.69in$

∴ use 24" Slab

Flexual Design:

$\frac{M_u}{b} = 1.4 * (15316.84\#/ft)\left(\frac{79.05ft}{2}\right)(12in/ft)$

$= 1,610,886\ in\text{-}lb/ft$

$\frac{A_s}{b} = \left(\frac{f'_c b}{1.176 f_y}\right)\left(d - \sqrt{\frac{2.353 M_u/b}{\phi f'_c * b}}\right)$

$= \left(\frac{(3000)(12)}{(1.176\ 60,000)}\right)\left(19.19 - \sqrt{\frac{2.353 * 1,610,886}{.9(3000)(12)}}\right)$

$= 1.126 in^2/ft$

$\min A_s/b = 0.0018 A_g = .0018 * (24in *)(12in)$

$= .5184 in^2$

$\frac{A_s}{b} \geq \min A_s b \therefore$ ok use

$1.126 in^2/ft$

$\therefore$ use #6 bar at 4 in bar spacing

which is $1.32 in^2/ft$

Check Temperture Steel

$A_s = 0.0018 A_g = (0.0018)(24 in)(19.05 ft)(12 in/ft)$

$= 4.26 in^2/ft$

Use 6 #8 bars with $A_{req} = 4.74 in^2$

Appendix E – Cost Estimate

Concrete Gravity Dam					
Division	Cost/Unit	# of units	Unit		Total
Division 1 - General requirements					
Subdivision:	The 25% O&P is already included in the RS means				
1310 Project Management/Coordination	cost estimating data				
620 Overhead and Profit (25% of total)	25%				
700 Field Personel	995	10	week		9950
1500 Temporary facilities and controls					
50 Generator	1250	2	each		2500
1590 Equipment rental					
100 concrete pump	704	5	day		3520
100 vibrators	11.3	5	each	5 days	113
100 Concrete batch truck	43.05		day	5 day	86.1
400 Restrooms	23.75	80	day		1900
1560 Barriers and Encolsures					
250 temporary fencing	6.75	440	linear foot		2970
1580 Project signs					
700 Signs	18	10	each		180
Division 2 - Site construction					
Subdivision:					
2220 Site demolition					
310 Demolition					10000
120 Disposal of material	$9.00	40	cy		1000
2240 Dewatering					
500 Dewatering	7.3		cy		5000
2260 Excavation support/protection					
200 Coffer dams	35	300	sf		10500
2310 Grading					
100 Finsih grading	$2.37	1500	sy		3555
2315 Excavation & Fill					
110 Channel excavation	$6.15		cy		3000
Division 3 - Concrete:					
Subdivisions:					
3060 Basic Concrete materials					
200 Cement	$75	3657	cy		275000
3200 Concrete Reinforcement					
100 #6 bar	$0.45	1728	lb		12000
100 #7 bar	$0.45	34560	lb		330000
100 #8 bar	$0.45	1440	lb		18000
Total					$690,000

Fix Existing Dam

Division	Cost/Unit	# of units	Unit	Total	
Division 1 - General requirements					
Subdivision:	The 25% O&P is already included in the RS means				
1310 Project Management/Coordination	cost estimating data				
620 Overhead and Profit (25% of total)	25%				
700 Field Personel	995	10	week		9950
1500 Temporary facilities and controls					
50 Generator	1250	2	each		2500
1590 Equipment rental					
100 vibrators	11.3	1	each	1 days	113
400 Restrooms	23.75	50	day		1000
1560 Barriers and Encolsures					
250 temporary fencing	6.75	440	linear foot		2970
1580 Project signs					
700 Signs	18	10	each		180
Division 2 - Site construction					
Subdivision:					
2240 Dewatering					
500 Dewatering	7.3		cy		5000
2260 Excavation support/protection					
200 Coffer dams	35	300	sf		10500
2310 Grading					
100 Finsih grading	$2.37	1500	sy		3555
2315 Excavation & Fill					
110 Embankment excavation	$6.15	1291	cy		11000
200 Build new embankment	$2.50	1069	cy		3000
350 Line E. Spillway with riprap	$118		sy		160000
Division 3 - Concrete:					
Subdivisions:					
superficial concrete					
Fixing existing structure					20000
Division - Metals					
Subdivisions:					
Fix gates with metal	$8,000		each		32000
Total					$261,768

Crest Gate Design

Division	Cost/Unit	# of units	Unit	Time	Total	notes
Division 1 - General requirements						
Subdivision:	The 25% O&P is already included in the RS means					
1310 Project Management/Coordination	cost estimating data					
620 Overhead and Profit (25% of total)	25%					
700 Field Personel	995	10	week		9950	
1500 Temporary facilities and controls						
50 Generator	1250	2	each		2500	
50 Welder	920	1	each		920	
1590 Equipment rental						
100 concrete pump	704	2	day		1408	
100 vibrators	11.3	5	each	2 days	113	
100 Concrete batch truck	43.05		day	2 day	86.1	
400 Restrooms	23.75	80	day		1900	
1560 Barriers and Encolsures						
250 temporary fencing	6.75	440	linear foot		2970	
1580 Project signs						
700 Signs	18	10	each		180	
Division 2 - Site construction						
Subdivision:						
2220 Site demolition						
310 Demolition					10000	
120 Disposal of material	$9.00	40	cy		1000	
2240 Dewatering						
500 Dewatering	7.3		cy		5000	
2260 Excavation support/protection						
200 Coffer dams	35	300	sf		10500	
2310 Grading						
100 Finsih grading	$2.37	1500	sy		3555	
2315 Excavation & Fill						
110 Channel excavation	$6.15		cy		3000	
Division 3 - Concrete:						
Subdivisions:						
3060 Basic Concrete materials						
200 Cement	$75	169	cy		12700	
3200 Concrete Reinforcement						
100 #6 bar	$0.45	7011	lb		4900	
100 #8 bar	$0.45	720	lb		900	
Division 5 - Metals						
Subdivisions:						
5120 Structural Steel						
640 structural steel members (I beams)	$120		Lf		17000	
Division 6 - Wood and Plastic						
Subdivisions:						
6100 Wood						
555 Beams	$2,100	1.68	1000 feet		12600	
Division 15 - Mechanical						
Hydraulic cylinders	$450	20	each		9000	
Total					$111,000	

Appendix F – Case study of Crest Gates

Since the most appropriate and most economical design, for the Lake Anastigunticook Dam Replacement Project, is a hinged, crest, gate (Bascule Gate) it is appropriate to investigate a similar design already constructed. Such a dam is the Collins Hydroelectric Project Dam (FERC L.P. No. P-6544-MA), owned by Swift River Company, located on the Chicopee River, in Wilbraham, MA. Swift River Company has generously allowed the use of both photographs and drawings of the dam and of its recent rehabilitation.

Figure One: Collins Hydroelectric Project. Panoramic view of dam taken from beneath the highway bridge.

The original dam was a timber crib structure, built by the Collins Paper Company, in 1872. The dam spanned the Chicopee River and conveyed water, through a power canal, to waterwheels, located in the basement of the brick mill. As the timbers reached their useful life, the structure weakened. On March 7th, 1979, a freshet that peaked at 10,500 cfs, caused a catastrophic failure of the timbers and the dam breached.

In 1982 Swift River Company, filed an ownership exemption, with the Federal Energy Regulatory Commission, to rebuild the dam and install a hydrogenation plant. The redevelopment scheme included abandoning both the power canal and the original timber crib dam design. The proposed project included a power plant, constructed integral with a Bascule style spillway. The former power canal was filled in. The river bed, downstream of the dam, was dredged to allow the head, at the exit of the old tailrace, to be brought upstream to the outlet of the new turbines. Two ESAC, 650 KW, pit bulb, turbines were installed in the powerhouse.

In plan view, the Collins Dam is a two section, dogleg. The primary length of the dog leg consists of a short Bascule gated section that extends from the north river bank, 56 feet to the powerhouse forebay wall, the powerhouse, and a much longer, twin, 64 feet long, Bascule gated spillway that extends 128 feet to the abutment of the former power canal. The dam then swings 90 degrees and a lengthy canal spillway, runs parallel to the river's edge, to the state highway bridge. Beyond this point, the former power canal has been completely filled in. The combined hydraulic capacities, of the fully depressed Bascule gates and the fixed canal spillway, with the headwater at the top of the abutments is 12,000 cfs.

In section, the dam has a rock filled base. The base is capped with heavily reinforced concrete. At 16 foot intervals, a heavy, steel, hinged, I-beam, needle, operated by a massive hydraulic cylinder, is attached to the concrete cap. The adjacent needles have southern yellow pine timbers stacked in their grooves. These timbers create movable panels that are raised up and down by the hydraulic cylinders. This allows the headwater elevation to remain constant with varying river flows. In between each four panel spillway is a robust concrete pier. Rubber seals are used to reduce leakage between the movable panels and the concrete piers. A heavy, steel, sheet pile, cutoff wall was driven 30 feet down into the river bottom upstream of the rock fill. The space between the top of the sheet piling and the rock fill was filled with air entrained concrete fill. Heavy riprap was placed on the downstream slope. A concrete pump was utilized to fill the interstices between the riprap with air entrained concrete. Automatic, unmanned operation was originally controlled by direct current, ice cube relays. These were replaced in 1989 with an Allen Bradley PLC (programmable logic controller). Fail safe operation was achieved by incorporating internal relief valves in the hydraulic manifold. In the event that the PLC and/or hydraulic system failed, during a flood, the relief valves are adjusted to open with a predetermined water surface elevation over the boards. The relief valve for each panel is set slightly higher then its successor panel. This allows the boards to fail in a controlled, progressive, cascade. Although this system was tested, it has never been used. The PLC, with its backup battery supply, has performed flawlessly.

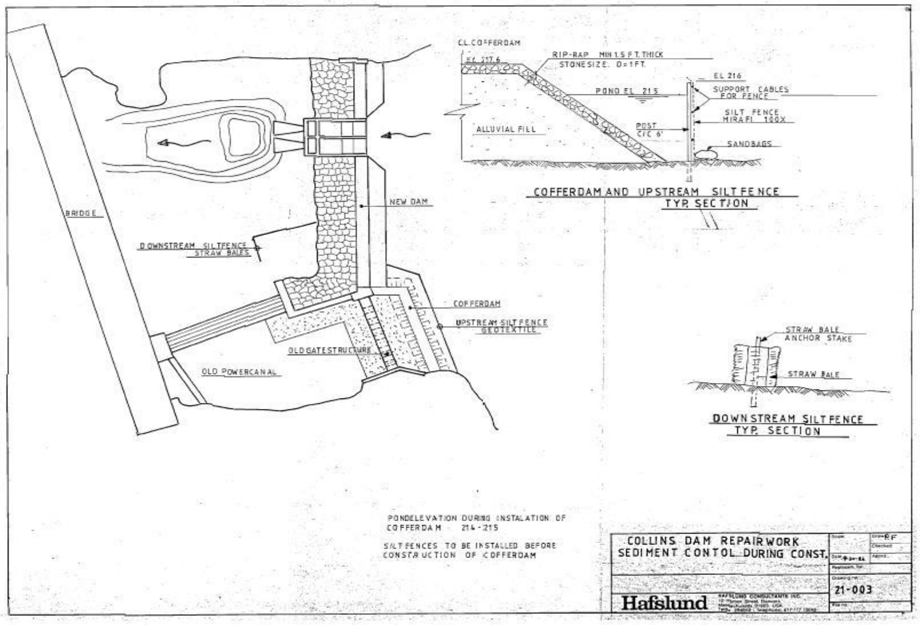


Figure Two: Plan View of Collins HEP Project.

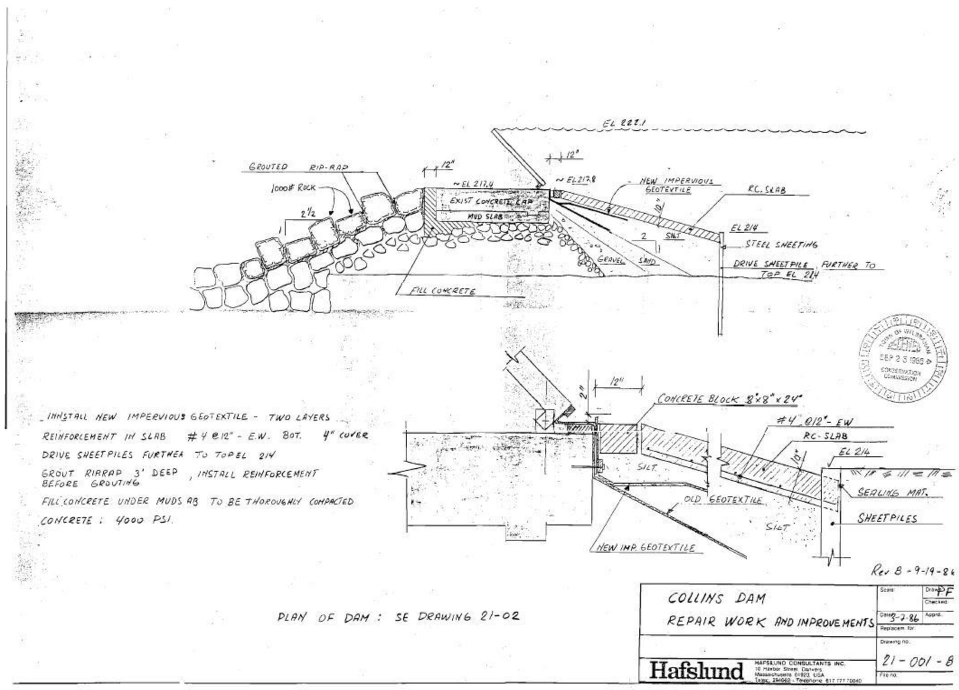


Figure Three: Sectional view of Collins HEP Spillway.

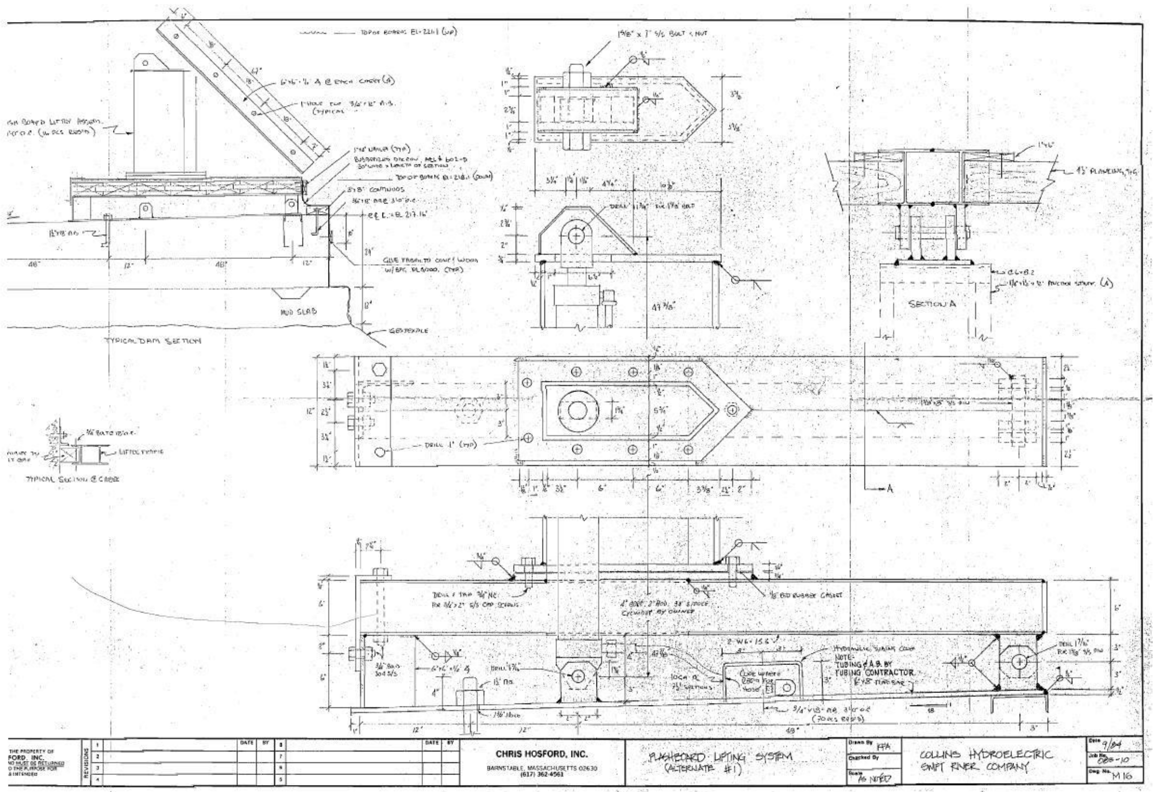


Figure Four: Detail drawing of Collins Bascule Gates

The project was constructed in 1984. In the summer of 2006, after 33 years of continuous operation, the Bascule sections were rehabilitated. The old wood was removed, the needles were sandblasted and painted. The hinge pins were replaced. The hydraulic cylinders were rebuilt. The cylinders needed new seals, new cylinder tie rods and new pins. They were sandblasted and painted. The hydraulic lines were replaced with all new stainless steel lines. The hydraulic control manifold and hydraulic powerpack were replaced. An Allen Bradley, SLC 500 programmable logic controller was installed to replace the ice cube relay automatic pond, level, control loop. The cost of the rebuild was $ 260,000, including labor.


Figure Five: Summer of 2006 rehabilitation. Note all the wooden panels have been removed for replacement

Figure Six: Summer of 2006 rehabilitation. Note all the wooden panels have been replaced. The blue barrels in the background are the boater safety buoys.

In conclusion, the Collins Dam serves as a model for the proposed Lake Anastigunticook Dam Replacement. It is a simple, inexpensive structure. It has a moderate life span. The life span can be easily prolonged with simple maintenance. It has flawlessly functioned for 33 years.

The construction of a Collins type dam, at the proposed dam site, at Lake Anastigunticock is simplified. This is because the leveling slab can be poured directly on the underlying ledge. This eliminates the rock filled section. The use of a PLC based control system allows the lake level to remain constant. This is achieved by lowering the timber panels with the hydraulic system as flood flows increase. Once the panels are fully depressed, the hydraulic profile reverts to channel control exerted by the historic channel walls. The hydraulic system is charged with water soluble, environmentally friendly oil.

The following photographs depict the summer of 2006 rehabilitation:

Figure Seven: Hydraulically controlled, I-Beam, Needles. Note the rectangular caps that the cylinders thrust on. The caps allow the boards to fully depress. They also protect the cylinder from debris flowing over the dam crest.

Figure Eight: A close up view of a hydraulically controlled, I-Beam, Needles.

Figure Nine: Note the laminated panels being held together with stainless steel, threaded rod. This is a simple, durable method of construction


Figure Ten: The downstream riprap being stabilized with air entrained concrete.
Up lift on the finished riprap was prevented with subsurface drains.

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Appendix H – Project proposal

ANASAGUNTICOOK LAKE DAM MQP PROPOSAL

William Fay

Celeste Fay

September '07

1.0 PROBLEM STATEMENT/GOALS

Dams have been molding societies for thousands of years. They provide drinking water and irrigation to areas that would otherwise perish and flood control to previously untamed rivers. Additionally, the roots of modern industry are based on the power captured by dams. For all of their glory however, there is a continuing battle for the perseverance of dam safety. All dams large and small are held accountable to a certain standard of dam safety depending on the jurisdiction that they fall under. The main goal of our project is to determine if the Anasagunticook Lake Dam is currently satisfying Maine Emergency Management Agency (MEMA) dam safety regulation. If it is not we will redesign the dam such that it meets both safety/environmental regulations and is constructible and economically feasible. Furthermore a secondary goal of this project is to provide an adequate Capstone design experience as a WPI graduation requirement.

2.0 CAPSTONE DESIGN REQUIREMENTS Accreditation Board for Engineering and Technology (ABET) requirement

The following requirements are per ABET and were e-mailed to all '08 Seniors form Tahir El-Korchi, Dept. Head, WPI on September 19, 2007.

1. At the start of an MQP, the faculty advisor discusses the need for a capstone design experience and the elements of capstone design.

2. In the MQP Proposal, a section on capstone design will be included which:

a. Presents a description of the design problem (about one paragraph text, may include a sketch if desired).

b. Outlines how the design problem is to be approached (a general description of the iterative process, or the range of parameters that will be investigated).
c. Discusses how most of the eight realistic constraints listed in the ASCE commentary are to be addressed. (Constraints: economic; environmental; sustainability; manufacturability; ethical; health and safety; social; and political.)

3. The proposal is to be included in the appendix section of the final report.

4. The final MQP Report is to include a one-page statement (at a minimum) or a chapter that informs the reader as to how the project satisfies capstone design.

3.0 BACKGROUND OF DAM

3.1 HISTORY

Canton, ME was first settled between 1790 and 1792. Originally, Anasagunticook Lake was Whitey pond named from a hunter who had been wounded by Indians. The first dam was built on Whitey brook around 1849. In 1886, Canton mills, powered by water consisted of a saw-mill, shook and stave mill and a grist mill[1]. The dam has been washed out and rebuild at least once in the early twentieth century. The industry and mills of Canton failed in the early 1970's and in 1996, Ray Fortier, formerly the dam operator purchased the dam. Even with the area changing, the pond is the primary water supply in the Town of Canton, making the questionable condition of the dam even more critical as it is currently drained approximately 6 feet because of the dam safety situation.

3.2 PERTINANT STRUCTURES

At this point in the project, we have not had an opportunity to visit the site. The following information about the pertinent structures has been collected from the multitude of reports and correspondence relative to Anasagunticook Lake Dam and where applicable the reference is noted. Future portions of this project will include verifying all of the numbers and assessments noted below.

[1] Varney, 1886

Anasagunticook Lake Dam is located in Oxford County ME at latitude 44.44 longitude 70.31667, approximately 330 feet south-west of the intersection of Main St and Turner St. The 100 year old dam is built on the outlet of a pond with approximately 580 acres and approximately 9800 acres of drainage basin. The earthen dam is approximately 175 feet long, 11 feet high and has a 25 foot concrete spillway structure with 4 overflow sluice gates. Additionally, there is a power intake blocked by another gate. The gates are powered by a single manual chain fall attached to the steel overhead gantry frame.

The left embankment is a non-homogeneous mixture of riprap and boulders with a fill of silty fine sand and a rock block foundation wall. The owner has placed a 3-4 foot layer of gravely sand with cobbles and boulders on top of the abutment which was then covered with hay. The core is approximately 12-15 feet thick and it is believed by Wright and Pierce that both the rock block wall and core were constructed on bedrock. [2] In the MEMA June 2006 dam safety order to Ray Fortier which references the original dam safety order given May 5, 2004, the upstream left embankment was described as having "settlement of embankment along the spillway retaining wall" and " settlement of embankment along the concrete retaining approach wall."[3] The downstream left embankment was described as having a " Sinkhole and surrounding settlement in embankment along outside stone retaining wall" ,"60' rut along embankment about 5'-10' in from outside stone retaining wall" and " 15' section of collapsed stone retaining wall 90' upstream of spillway."[4]

The right embankment described is by Wright and Pierce as extending 150 feet upstream from the dam from an elevation of 398' (stream el.) to between 404' to 406'. The existing ground surface is relatively clear from the stream to about ½ way to the railroad bridge with the other half overgrown with small bushes and trees. The ground surface approximately 40 feet from the stream has a covering of cobbles and boulders. The steep slope directly adjacent to the stream is covered in "spotty" riprap (the thickness undeterminable due to non-uniformity). The fill at the top of the slope is topsoil over approximately 6 feet of gravely sand which appears to be non-homogeneous fill. [5] The MEMA dam safety order described the right embankment upstream as being deficient due to settlement of the embankment at the spillway concrete retaining wall. The downstream right embankment was described as having seepage from the toe area of the right dike about 60 feet from the spillway and uncontrolled leakage of approximately 50-100 gpm.[6]

[2] Wright and Pierce, 8/07
[3] '04 Dam safety order
[4] '04 Dam safety order
[5] Wright and Pierce, '07
[6] '04 Dam safety order

The gravity spillway is constructed of concrete with 4 overflow gates. Three of the four gates are constructed of wooden leaves and stems while the fourth is of stainless steel. All of the wooden gates have been reinforced for strength however, one of the three is still in poor condition. The stainless steel gate is in good condition. The gate guides only extend approximately 1 foot to 2 feet above the spillway deck meaning that the gates have to be either 1-2 feet open or taken completely out. [7] There does appear to be minor spalling in the concrete that should be taken care of however overall, the concrete spillway structure appears to be in good condition. The MEMA order described that the spillway was deficient due to gate overflow restrictions and leaks in the guides however it is structurally sound. Also it is questionable if the spillway could pass the USACE design flood inflow.

3.3 CURRENT ISSUES/ORDERS

Anasigunticook Pond Dam has left one of the largest paper trail in the MEMA dam safety office. There are several issues that are enraging people and many orders issued for repairs to the dam. In December '06, MEMA gave Ray Fortier a dam safety order which updated a similar order dated May 5, 2004 (which was not complied to at all) It included the following:

- Engage a licensed professional engineer, specializing in dam construction to assist in preparing a remedial action plan
- Develop a remedial action plan with the assistance of the PE to restore the integrity and structural stability of the dam and to assure that it functions and operates in a manner that will protect public safety, including at a minimum
    - Evaluation of causes and extent of seepage, settlement and erosion of both earthen embankments and a plan for restoring the integrity and safety of the abutments
    - A plan for removing all new fill material along the left embankment or if the PE determines that the fill is not compromising any structural integrity, a plan for stabilizing and incorporating the fill into the embankment
    - A plan for repairing and resting the four spillway gates such that they are functional and can be completely raised in a timely manner
    - Develop an emergency operational procedure for the spillway gates during a flooding situation

[7] Wright and Pierce, '07

  - Develop a plan for reducing the height of all four spillway gates to increase the flow capacity of the spillway
  - Schedule for completing all elements by Dec. 31, 2007
- Complete all work in accordance with local and state permitting rules

In a letter dated May 8, 2007, from MEMA to the owner, it was discussed that none of the previously issued orders had been complied with. As a result, it was decided that the current state of the dam poses a potential but real and impending danger to life, limb or property because of flooding or potential and imminent flooding pursuant to 37-B M.R.S.A., Section 1114(2). Because of this danger, Ray Fortier was ordered to maintain a lower water level and keep the spillway gates open until the remedial actions have been met. The entire situation becomes more complicated because much of the town is dependent on the lake as a source of water. On September 13, 1978 the State Soil and Water Conservation Commission (enforced by the DEP) issued a water level order to try to regulate the lake water level. Operation of the dam is also regulated by the Anasagunticook Lake Water Level Management Plan issued by DEP which describes the specific steps necessary to carry out the water level order. The plan describes closing two of the gates on or about April 15 every year and to close the remaining two gates on or about May 1st with the goal of achieving a target water level of Mark 23 2/3 for the summer. In addition to the dam safety problem, another real issue at this site is that with the four gates opened all summer, the water level is approximately 6 feet below the target level.

3.4 REGULATION

1) State of Maine

a. Maine Emergency Management Agency (MEMA) is responsible for dam safety in Maine. Title 37-B, Chapter 24 of the Maine State Statues gives the authority to the State Dam Safety program and describes how it is set up, regulated, and administered. The full content of the statute can be found at http://janus.state.me.us/legis/statutes/37-b/title37-bch24sec0.html . For regulations and specifications related to dam safety the statute refers to the United States Army Corps of Engineers' standards.

b. The Maine Department of Environmental Protection (Maine DEP) is responsible for the protection of environmental quality in the State of Maine. More research needs to be done regarding jurisdiction of Maine DEP in regards to the Anasagunticook Lake Dam. However 06-096 Chapter 450 and 04-061 Chapter 11 of the Maine DEP's Administrative Regulations describe the regulation of hydroelectric projects and dams. This can be found at http://www.maine.gov/dep/blwq/docstand/hydropage.htm . Also the Maine DEP is responsible for water level orders and enforcing them.

2)Local Regulation

a. Town of Canton

The town of Canton has a direct regulatory position in the project resulting from the ruling of Superior Court Docket CV-97-55. The court's ruling mandated that the Town review and approve of any applications for local permits required to rehabilitate the dam. The ruling and orders can be found at: http://www.cantonmaine.com/canton/ad20.htm .

b. Canton Water District

We have tried to contact Robert Doucette of the Canton Water District to request a copy of their charter. However a copy has not yet been secured. The water district supplies approximately 330 customers with water from Lake Anasagunticook. The supply is threatened by the lowered water levels, so the Water District has a direct interest in regulating what happens ay the dam site.

3)United States Army Corps of Engineers (USACE)

a. Water Quality

The USACE regulates any dredging or filling of materials in waterways of the United States. This comes from section 404 of the Clean Water Act, a copy of it is at http://www.usace.army.mil/cw/cecwo/reg/sec404.htm .

b. Dam Safety Regulations

The USACE is referred by MEMA as the source of engineering regulations for dam safety. The USACE has over 120 sets of engineering regulations on civil works alone. The pertinent regulations for this project are as followed:

i. ER 1110-1-8100 deals with regulations regarding laboratory investigations and testing.

ii. ER 1110-2-101 deals with the regulations surrounding the reporting of distress in civil works.

iii. ER 1110-2-110 deals with regulations regarding the evaluation of civil works projects.

iv. ER 1110-20112 describes regulations dealing with the required visits to construction sites by design personnel.

v. ER 1110-2-1150 describes the regulations for the engineering and design of civil Works Projects.

vi. ER 1110-2-1156 explains the engineering regulation to dam safety organization, responsibilities, and activities.

vii. ER 110-2-1302 discribes the engineering regulation of civil works cost engineering.

viii. ER 1110-2-1450 talks about the engineering regulation of hydrologic frequency investigations.

ix. ER 1110-2-1464 deals with the regulation for hydrologic analisis of watershed runoff.

x. ER 1110-2-1806 talks about earthquake design and evaluation of civil works regulation.

xi. ER 1110-8-2(FR) describes the engineering regulation for the inflow design floods for dams and reservoirs.

ER in the document title stands for engineering regulation. These documents can be downloaded from http://www.usace.army.mil/publications/eng-regs/cecw.htm .

4.0 METHODOLOGY

This section describes how we are going to go about our research and Project. It is a broad overview but it includes everything that we might need to research and analyze to complete our project. As the project progresses tasks and items will be removed or added as needed to supplement the project.

1) Regulation and Current Issues

    a) Regulation

        i) Governing Bodies

        ii) Authority of Governing Bodies

        iii) State of Maine Dam Safety Regulations

        iv) United Sates Army Corps of Engineers (USACE)

            (1) USACE Regulations

            (2) USACE Engineering Manuals

            (3) USACE Computer Programs

        v) Maine Department of Environmental Protection

        vi) Applicable Town Laws and Ordinances (Including water supply)

    b) Current Issues

        i) Current MEMA Administrative Orders

        ii) Water Level Order

        iii) Correspondence between the Town of Canton, Ray Fortier (Owner), and MEMA

        iv) Engineering Reports

        v) Town of Canton Water Supply

2) Hydrology and Hydraulics

        i) Basic hydrologic and meteorological data

            (1) Gathering Stream Flow Data (Historic)

            (2) Compiling Peak Discharge Data

            (3) Available Rainfall Records

ii) Field Reconnaissance of Drainage Basin

(1) Drainage Network

(2) Soil and Geologic Conditions

(3) Slope

(4) Land Use

(5) Significant Basins

(6) Vegetative Cover

iii) Development of Probable Maximum Storms

(1) Hydro-Meteorological Reports 51 & 52

(2) USACE HMR-52

iv) Flood Run-Off

(1) Unit Hydrograph Lag Time

(2) Development of Unit Hydrograph

(3) Base Flow and Interflow

(4) Design Flood Hydrograph

v) Estimates of Flood Frequency

vi) Inflow Design Flood

vii) Comparison to the FEMA Flood Maps

viii) Size and Estimate Spillway outflow at Overtopping

ix) Compare to Design Flood with USACE Specifications

x) Analysis of Downstream Channel

(1) Profile

(2) Convergence and Divergence

(3) Channel Freeboard

(4) Inundation Analysis

3) Site Visit

a) General Site Inspection

b) Photos

c) Appurtenant Structure Survey

d) Spillway Dimensions

e) Soil Samples (Dyke and/or Foundations)

f) Structural Deficiencies

g) Inspection and Survey of Downstream Channel

h) Impoundment Survey (Visual)

i) Drainage Area Survey (Vegetation, slope, soil type, development, ect.)

j) Owner Interview

k) Sand Bar inspection and Survey

4) Embankments

a) Soils Analysis

b) Current Integrity Analysis

c) Pore Water Pressure

d) Seepage Through Embankments

e) Stability Analysis

f) Slope Analysis

g) Seismic Threat Analysis

h) Seismic Design

i) Crest Width Design

j) Freeboard Calculations

k) Waves

i) Maximum Wave Height Analysis

ii) Upstream Slope Protection Design

l) Downstream Slope Protection

m) Interior Drainage Design

n) Exterior Drainage Design

o) Vegetation

p) Construction Materials

5) Spillway

a) Current Integrity Analysis

b) Analysis of Spillway Size and Type (In H&H but more detail)

c) Tail Water Curve

d) Analysis of Downstream Basin

e) Forces Acting on Dam

i) External Water Pressure

ii) Internal Water Pressure

iii) Dead Load

iv) Ice

v) Silt Pressure

vi) Earthquake

vii) Load Combinations

f) Stress and Stability Analysis

i) Safety Factors

ii) Sliding Stability

iii) Internal Stresses (Uncracked)

iv) Internal Stresses and Sliding Stability (Cracked)

g) Spillway gate Design

h) Emergency Spillway Design (Fuse-plug type?)

i) Fuse Plug Design

ii) Channel Design

iii) Backwater

i) Construction Materials

6) Foundations

a) Determine Foundation Type

b) Rock Foundation

i) Rock Type

ii) Rock Strength

iii) Internal Water Pressures

iv) Dam Foundation Interface

c) Earth Foundation

i) Soil Type

ii) Soil Strength

iii) Seepage/Permeability

iv) Internal Pressures

d) Foundation Configuration

7) Laboratory Tests

a) Soils

i) Gradation

ii) Moisture Content

iii) Atterburg Limits

iv) Specific Gravity

v) Laboratory Compaction

vi) Relative Density

b) Rip-Rap and Concrete Aggregate

i) Specific Gravity and Absorption

ii) Abrasion

iii) Soundness

iv) Density

v) Hardness

8) Report

a) Introduction

b) Background

c) Methodology

d) Engineering Analysis

e) Assessment of Current Conditions

f) Best Course of Action with regards to economic feasibility, design considerations, and engineering feasibility.

g) Conclusions

h) Appendices

   i) List of Terms

   ii) Pertinent Regulations

   iii) List of References

   iv) Capstone Design Assessment

   v) Drawings, surveys, and site plans

vi) Engineering Calculations

5.0 PROJECT TIMELINE

1. Regulation and Current Issues

   8/23-9-24

2. Project Proposal

   8/23-10/11

3. Hydrology and Hydraulics

   9/04-10/26

4. Site Visit

   Before 11/01

5. Existing Structure Evaluation

   11/01-12/20

6. Redesign Existing Dam or New Design

   12/20-02/01

7. Report Rough Draft

   11/01-02/01

8. Report Final Draft

   02/01-02/28

WPI 2007-2008 Undergraduate Calendar

	S	M	T	W	R	F	S		S	M	T	W	R	F	S
	29	30	31	1	2	3	4		10	11	12	13	14	15	16
	5	6	7	8	9	10	11	FEB	17	18	19	20	21	22	23
AUG	12	13	14	15	16	17	18		24	25	26	27	28	29	1
	19	20	21	22	23	24	25		2	3	4	5	6	7	8
	26	27	28	29	30	31	1		9	10	11	12	13	14	15
	2	3	4	5	6	7	8	MAR	16	17	18	19	20	21	22
	9	10	11	12	13	14	15		23	24	25	26	27	28	29
SEPT	16	17	18	19	20	21	22		30	31	1	2	3	4	5
	23	24	25	26	27	28	29		6	7	8	9	10	11	12
	30	1	2	3	4	5	6	APR	13	14	15	16	17	18	19
	7	8	9	10	11	12	13		20	21	22	23	24	25	26
OCT	14	15	16	17	18	19	20		27	28	29	30	1	2	3
	21	22	23	24	25	26	27		4	5	6	7	8	9	10
	28	29	30	31	1	2	3	MAY	11	12	13	14	15	16	17
	4	5	6	7	8	9	10		18	19	20	21	22	23	24
NOV	11	12	13	14	15	16	17		25	26	27	28	29	30	31
	18	19	20	21	22	23	24		1	2	3	4	5	6	7
	25	26	27	28	29	30	1		8	9	10	11	12	13	14
	2	3	4	5	6	7	8	JUNE	15	16	17	18	19	20	21
	9	10	11	12	13	14	15		22	23	24	25	26	27	28
DEC	16	17	18	19	20	21	22		29	30	1	2	3	4	5
	23	24	25	26	27	28	29		6	7	8	9	10	11	12
	30	31	1	2	3	4	5	JULY	13	14	15	16	17	18	19
	6	7	8	9	10	11	12		20	21	22	23	24	25	26
JAN	13	14	15	16	17	18	19		27	28	29	30	31	1	2
	20	21	22	23	24	25	26		3	4	5	6	7	8	9
	27	28	29	30	31	1	2	AUG	10	11	12	13	14	15	16
FEB	3	4	5	6	7	8	9		17	18	19	20	21	22	23